UNION SANITARY DISTRICT

Directors
Manny Fernandez
Tom Handley
Pat Kite
Anjali Lathi
Jennifer Toy

Officers
Paul R. Eldredge
General Manager/
District Engineer

Karen W. Murphy
Attorney

BOARD MEETING AGENDA
Monday, May 23, 2016
Regular Meeting - 7:00 P.M.

Union Sanitary District
Administration Building
5072 Benson Road
Union City, CA 94587

1. Call to Order.

2. Pledge of Allegiance.

3. Roll Call.

Motion 4. Approve Minutes of the Special Meeting of April 27, 2016.

Motion 5. Approve Minutes of the Meeting of May 9, 2016.

6. Monthly Operations Report *(to be reviewed by the Budget & Finance Committee).*
   a. April Monthly Odor Report & Financial Reports.
   b. Third Quarter FY 16 District-Wide Balanced Scorecard Measures.
   c. Balanced Scorecard Report for the Technical Services Workgroup.

7. Written Communications.

8. Oral Communications.

The public may provide oral comments at regular and special Board meetings; however, whenever possible, written statements are preferred ***(to be received at the Union Sanitary District office at least one working day prior to the meeting)****. This portion of the agenda is where a member of the public may address and ask questions of the Board relating to any matter within the Board's jurisdiction that is not on the agenda. If the subject relates to an agenda item, the speaker should address the Board at the time the item is considered. Oral comments are limited to three minutes per individuals, with a maximum of 30 minutes per subject. Speaker's cards will be available in the Boardroom and are to be completed prior to discussion.*

Motion 9. Schedule Public Hearing to Consider Collection of Sewer Service Charges on the Tax Roll for Fiscal Year 2017 *(to be reviewed by the Legal/Community Affairs Committee).*

Motion 10. Accept the Final Seismic Assessment Reports from Degenkolb Engineers (*to be reviewed by the Construction Committee)*.

Motion 11. Authorize the General Manager to execute Amendment No. 2 to Task Order No. 2 with West Yost Associates for the Plant Facilities Improvements Project *(to be reviewed by the Construction Committee)*.

Information	12.	Information Items:

a. Check Register.
b. Standard Specifications and Information Bulletin Update *(to be reviewed by the Construction Committee)*.

Information	13.	Committee Meeting Reports. *(No Board action is taken at Committee meetings):*

a. Construction Committee – scheduled for Wednesday, May 18, 2016, at 10:30 a.m.
b. Budget & Finance Committee – scheduled for Thursday, May 19, 2016, at 8:30 a.m.
c. Legal/Community Affairs Committee – scheduled for Friday, May 20, 2016, at 9:15 a.m.
d. Legislative Committee – will not meet.
e. Personnel Committee – will not meet.
f. Ad Hoc Subcommittee for Communications Strategy.

Information	14.	General Manager's Report. *(Information on recent issues of interest to the Board).*

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15. Other Business:
    a. Comments and questions. *Directors can share information relating to District business and are welcome to request information from staff.*
    b. Scheduling matters for future consideration.

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16. Adjournment – The Board will adjourn to the next Regular Meeting in the Boardroom on Monday, June 13, 2016, at 7:00 p.m.

The Public may provide oral comments at regular and special Board meetings; however, whenever possible, written statements are preferred (to be received at the Union Sanitary District at least one working day prior to the meeting).
If the subject relates to an agenda item, the speaker should address the Board at the time the item is considered. If the subject is within the Board's jurisdiction but not on the agenda, the speaker will be heard at the time "Oral Communications" is calendared. Oral comments are limited to three minutes per individual, with a maximum of 30 minutes per subject. Speaker's cards will be available in the Boardroom and are to be completed prior to discussion of the agenda item.

The facilities at the District Offices are wheelchair accessible. Any attendee requiring special accommodations at the meeting should contact the General Manager's office at (510) 477-7503 at least 24 hours in advance of the meeting.

THE PUBLIC IS INVITED TO ATTEND

NOTICE OF
COMMITTEE MEETING

All meetings will be held in
the General Manager's Office
5072 Benson Road, Union City, CA 94587

BOARD MEETING OF MAY 23, 2016

Committee Membership:

Budget and Finance	Directors Manny Fernandez and Pat Kite (Alt. – Jennifer Toy)
Construction Committee	Directors Tom Handley and Jennifer Toy (Alt. – Pat Kite)
Legal/Community Affairs	Directors Pat Kite and Anjali Lathi (Alt. – Tom Handley)
Legislative Committee	Directors Manny Fernandez and Tom Handley (Alt–Pat Kite)
Personnel Committee	Directors Manny Fernandez and Jennifer Toy (Alt. – Anjali Lathi)
Audit Committee	Directors Anjali Lathi and Jennifer Toy (Alt. Manny Fernandez)

Construction Committee, Wednesday, May 18, 2016, at 10:30 a.m.

10. Accept the Final Seismic Assessment Reports from Degenkolb Engineers.

11. Authorize the General Manager to execute Amendment No. 2 to Task Order No. 2 with West Yost Associates for the Plant Facilities Improvements Project.

12b. Standard Specifications and Information Bulletin Update.

Budget & Finance Committee, Thursday, May 19, 2016, at 8:30 a.m.

6. Monthly Operations Report*.*
   a. April Monthly Odor Report & Financial Reports.
   b. Third Quarter FY 16 District-Wide Balanced Scorecard Measures.
   c. Balanced Scorecard Report for the Technical Services Workgroup.

Legal/Community Affairs Committee, Friday, May 20, 2016, at 9:15 a.m.

9. Schedule Public Hearing to Consider Collection of Sewer Service Charges on the Tax Roll for Fiscal Year 2017.

Committee meetings may include teleconference participation by one or more Directors.
(Gov. Code Section 54953 (b))
Committee Meetings are open to the public. Only written comments will be considered. No action will be taken.

MINUTES OF THE SPECIAL MEETING OF THE
BOARD OF DIRECTORS OF
UNION SANITARY DISTRICT
April 27, 2016

CALL TO ORDER

President Toy called the special meeting to order at 5:40 p.m.

ROLL CALL

PRESENT: Jennifer Toy, President
Tom Handley, Vice President
Pat Kite, Secretary
Anjali Lathi, Director
Manny Fernandez, Director

STAFF: Paul Eldredge, General Manager
Karen Murphy, District Counsel
Armando Lopez, Treatment & Disposal Services Manager
James Schofield, Collection Services Manager
Robert Simonich, Fabrication, Maintenance, and Construction Manager
Sami Ghossain, Technical Services Manager
Pamela Arends-King, Business Services Manager/CFO
Maria Buckley, Principle Financial Analyst

PUBLIC COMMENT

There was no public comment.

BOARD WORKSHOP – FY17 OPERATING BUDGET

Staff provided a presentation on the FY17 Operating Budget and responded to Board questions.

ADJOURNMENT:

The special meeting was adjourned at approximately 8:00 p.m. to the next Regular Board Meeting in the Boardroom on Monday, May 9, 2016, at 7:00 p.m.

SUBMITTED: ATTEST:

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REGINA McEVOY
SECRETARY TO THE BOARD

____
PAT KITE
SECRETARY

APPROVED:

____
JENNIFER TOY
PRESIDENT

Adopted this 23rd day of May, 2016

MINUTES OF THE MEETING OF THE
BOARD OF DIRECTORS OF
UNION SANITARY DISTRICT
May 9, 2016

CALL TO ORDER

President Toy called the meeting to order at 7:00 p.m.

PLEDGE OF ALLEGIANCE

ROLL CALL

PRESENT: Jennifer Toy, President
Tom Handley, Vice President
Pat Kite, Secretary
Anjali Lathi, Director
Manny Fernandez, Director

STAFF: Paul Eldredge, General Manager
Karen Murphy, District Counsel
Leah Castella, Special Counsel
Armando Lopez, Treatment & Disposal Services Manager
Robert Simonich, Fabrication, Maintenance, and Construction Manager
Sami Ghossain, Technical Services Manager
Pamela Arends-King, Business Services Manager/CFO
Tim Grillo, Research and Support Team Coach
Michelle Powell, Communications and Intergovernmental Relations Coordinator
Mariela Espinosa, Customer Service Fee Analyst
Sol Cooper, Mechanic
Regina McEvoy, Assistant to the General Manager/Board Secretary

VISITOR: Alice Johnson, League of Women Voters
Marty Koller, Alameda County Water District Boardmember
Pranshu Chaturvedi and family
Shreya Ramachandran and family
Gabriele Estabrook, Mission San Jose High School
Fe Marie Bustos, The Stratford School

APPROVAL OF THE MINUTES OF THE SPECIAL MEETING OF APRIL 19, 2016

It was moved by Secretary Kite, seconded by Vice President Handley, to approve the Minutes of the Special Meeting of April 19, 2016. Motion carried unanimously.

APPROVAL OF THE MINUTES OF THE MEETING OF APRIL 25, 2016

It was moved by Director Lathi, seconded by Director Fernandez, to approve the Minutes of the Meeting of April 25, 2016. Motion carried unanimously.

WRITTEN COMMUNICATIONS

There were no written communications.

ORAL COMMUNICATIONS

There were no oral communications.

PRESENTATION OF ALAMEDA COUNTY SCIENCE AND ENGINEERING FAIR EXCELLENCE IN WATER RESEARCH AWARDS TO JUNIOR AND SENIOR DIVISION FIRST-PLACE WINNERS

Communications and Intergovernmental Relations Coordinator Powell stated the District is one of ten Alameda County water and wastewater agencies that collaborated to create and fund the annual Excellence in Water Research Awards for the annual Alameda County Science and Engineering Fair. The awards include cash prizes and are given to students whose projects are related to water or wastewater issues. A member of the District's laboratory staff has served as a judge for these awards since their inception four years ago. The Board presented awards to the following students and teachers:

Student	School	Teacher	Project Title
Pranshu Chaturvedi	Mission San Jose High School	Gabriele Estabrook	A Novel Technique for Water Desalination Using the Diamagnetic Properties of Water
Shreya Ramachandran	The Stratford School	Fe Marie Bustos	Effect of Soap Nut Greywater on Soil and Plants

President Toy recessed the meeting at 7:10 p.m. for a reception honoring the science fair winners and their projects.

President Toy reconvened the meeting at 7:30 p.m.

LEGISLATIVE UPDATE ON REGIONAL, STATE, AND NATIONAL ISSUES OF INTEREST TO THE BOARD

This item was reviewed by the Legislative Committee. General Manager Eldredge stated the informational report included in the Board meeting packet provided an overview of legislation which may impact the District or be of interest to the Board. General Manager Eldredge provided a brief overview of the following proposed legislation: SB 163 – (Hertzberg D) Wastewater Treatment: Recycled Water; and SB 1069 – (Wieckowski D)

An Act to Amend Sections 65582.1, 65583.1, 65589.1, 65852.150, 65852.2, and 66412.2 of the Government Code, Relating to Land Use. Staff responded to Boardmember questions.

Vice President Handley stated SB163 would require the District to discharge treated wastewater to the aquifer and asked who would take control of the aquifer.

General Manager Eldredge stated the aquifer is an adjudicated groundwater basin controlled by Alameda County Water District per State legislation.

Secretary Kite requested clarification regarding AB 2389 – (Ridley-Thomas D) Special Districts: District-Based Elections: Reapportionment. General Manager Eldredge stated staff would research the bill and provide more information to the Board.

Secretary Kite requested further information regarding AB 2511 – (Levine D) Fertilizing Materials: Auxiliary Soil and Plant Substances: Biochar. General Manager Eldredge stated biochar is produced by a certain biosolids treatment technology whereby biosolids are heated to a high temperature and biochar is the remaining material.

Director Lathi requested clarification regarding AB 2257 – (Mainenschein R) Local Agency Meetings: Agenda: Online Posting. General Manager Eldredge stated that while agendas have been posted on the Board of Directors page on the District website, the proposed bill would require a prominent direct link to the current agenda itself.

President Toy stated the format of the legislative update report was easy to follow.

Director Lathi requested future versions of the report indicate the current status of the proposed legislation.

REVIEW AND APPROVE PROPOSED CHANGES TO POLICY NO. 3060, COMMUNICATION BY MEMBERS OF THE BOARD OF DIRECTORS

This item was reviewed by the Personnel Committee. General Manager Eldredge stated the Board previously considered a revised version of Policy No. 3060 at the April 11, 2016, Board meeting. The Board discussed proposed revisions at the meeting, and directed staff to incorporate further edits to the Policy. Special Counsel Castella stated the revisions to Policy No. 3060 were designed to clarify that the Policy was not intended to in any way limit the freedom of individual Boardmembers to communicate on their own behalf with the public, media representatives, or other publicly elected officials. Staff recommended the Board either approve the Policy as drafted or approve the Policy with amendments.

The Board agreed by consensus to add the following text to Section 3.a of the Policy: When a communication is sent on behalf of the entire Board, it will be signed by the individual Boardmember with the language, “on behalf of the Union Sanitary District Board.”

It was moved by Vice President Handley, seconded by Secretary Kite, to Approve Proposed Changes to Policy No. 3060, Communication by Members of the Board of Directors as amended. Motion carried unanimously.

The following item was considered at this time: **Reclaimed Water Alternatives**. For information regarding said item, please see the Information Items section of the minutes.

REVIEW AND APPROVE PROPOSED CHANGES TO POLICY NO. 3030, BOARDMEMBER BUSINESS EXPENSE

This item was reviewed by the Personnel Committee. General Manager Eldredge stated the recommended revisions to the Policy included the following: changing the distance requirement for overnight lodging, altering the parking reimbursement requirement, adding information pertaining to car rentals, and providing clarification regarding unauthorized expenses. Staff recommended the Board adopt the proposed changes to Policy No. 3030, Boardmember Business and Travel Expense.

The Board reviewed proposed edits to the Policy, and discussed additional edits.

Sol Cooper requested the Board keep in mind public perception in regard to loosening restrictions contained within the Policy.

General Manager Eldredge stated staff would redraft proposed revisions to Policy 3030 to be presented at a future Board meeting.

REVIEW AND APPROVE PROPOSED CHANGES TO POLICY NO. 3045, BOARD EDUCATION AND TRAINING BUDGET

This item was reviewed by the Personnel Committee. General Manager Eldredge stated minor edits to the Policy were proposed to add clarity and ensure consistency with other Board policies. Staff recommended the Board adopt the proposed changes to Policy No. 3045, Board Education and Training Budget.

It was moved by Director Lathi, seconded by Secretary Kite, to Approve Changes to Policy No. 3045, Board Education and Training Budget. Motion carried unanimously.

SELECT BOARD MEMBERS TO REPRESENT USD ON EXTERNAL COMMITTEES FOR FY17

This item was reviewed by the Personnel Committee. General Manager Eldredge stated Policy No. 3070, Boardmember Officers and Committee Membership, calls for the Board to select representatives for four External Committees no later than the first meeting in May. A list of current external committee representatives and alternates was included in the Board meeting packet. The Board discussed external committee assignment preferences.

It was moved by Vice President Handley, seconded by Director Lathi, to accept the following external committee assignments for FY 2017:

Organization	Representative	Alternate
Alameda County Water District Financing Authority (ACWDFA)	Anjali Lathi	Pat Kite
East Bay Dischargers Authority (EBDA) Commission	Jennifer Toy	Tom Handley
Alameda County Special Districts Association (ACSDA)	Pat Kite	Manny Fernandez
Southern Alameda County Geographic Information System (SACGIS)	Manny Fernandez	Anjali Lathi

Motion carried unanimously.

CONSIDER A RESOLUTION TO AUTHORIZE STAFF TO SPECIFY HYDRO INTERNATIONAL AS A SOLE SOURCE EQUIPMENT MANUFACTURER FOR THE SLUDGE DEGRITTER SYSTEM PROJECT

This item was reviewed by the Construction Committee. Technical Services Manager Ghossain stated degritters remove grit from the primary sludge process to minimize deposits in the anaerobic digesters and reduce wear on pumps, centrifuges, and other equipment. Grit removal extends the life of Plant equipment and results in cost savings for the District. The Board authorized the General Manager to execute Task Order No. 1 with West Yost Associates in the amount of $180,629, to provide design services for the Sludge Degritter System Project. The District has been operating and maintaining the two existing degritters, manufactured by Hydro International, since 2001. Staff evaluated the new degritter pursuant to Policy No. 2760, Standardized Equipment Policy, and determined the equipment meets the “Match Existing Equipment” criteria which allows it to be sole sourced. Staff recommended the Board adopt a resolution authorizing staff to specify Hydro International as a sole source equipment manufacturer of the Eutek Slurry Cup grit separator and washing unit and the Eutek Grit Snail grit clarifier and dewatering escalator for the Sludge Degritter System Project.

Vice President Handley asked if staff explored alternate options before deciding to proceed with purchasing a new Hydro International degritter. Treatment & Disposal Services Manager Lopez stated that given the layout of the Plant, this type of technology is the most advantageous to the District.

It was moved by Director Fernandez, seconded by Director Lathi, to Adopt Resolution No. 2780, to Authorize Staff to Specify Hydro International as a Sole Source Equipment Manufacturer of the Eutek Slurry Cup Grit Separator and Washing Unit and the Eutek Grit Snail Clarifier and Dewatering Escalator for the Sludge Degritter System Project. Motion carried unanimously.

INFORMATION ITEMS:

Check Register

All questions were answered to the Board's satisfaction.

Reclaimed Water Alternatives

This item was reviewed by the Legal/Community Affairs Committee. Research and Support Team Coach Grillo stated the Board directed staff to develop alternatives, including approximate implementation and operation costs, for producing a small volume of reclaimed water and a corresponding residential fill station. Research and Support Team Coach Grillo provided an overview of the reclaimed water alternatives detailed in the Board meeting packet, and responded to Boardmember questions.

General Manager Eldredge stated the Alameda County Water District Board will consider a similar item at its meeting to be held May 26, 2016.

Status of Priority 1 Capital Improvement Program Projects

This item was reviewed by the Construction Committee. Technical Services Manager Ghossain stated the Executive Team quarterly reviews the status of Priority 1 Capital Improvement Program (CIP) Projects. For FY 2016, ten projects were ranked as Priority 1 projects. A summary of Priority 1 projects was included in the Board meeting packet.

Third Quarterly Report on the Capital Improvement Program

This item was reviewed by the Construction Committee. Technical Services Manager Ghossain stated total CIP expenditures up to March 31, 2016, were below projections for the third quarter. This was due in part to delays with the following projects: Thickener Control Building, Fremont & Paseo Padre Lift Station Internal Lift Pumps, and Equalization Storage at Alvarado. Information regarding delayed projects was included in the Board meeting packet.

Report on the East Bay Dischargers Authority (EBDA) Commission Meeting of April 21, 2016

Vice President Handley stated General Manager Eldredge will schedule a knowledge transfer meeting for Vice President Handley and President Toy before the July EBDA meeting.

COMMITTEE MEETING REPORTS:

The Construction, Legislative, Legal/Community Affairs, and Personnel Committees met.

GENERAL MANAGER'S REPORT:

General Manager Eldredge reported the following:

- Environmental Compliance staff recently participated in an Earth Day event and collected 184 pollution prevention pledges, distributed 200 USD tote bags, collected approximately 200 pounds of expired medication, and exchanged 14 mercury thermometers.
- The District's Annual Certificates of Merit Ceremony will be held in the Boardroom from 3:00 – 4:00 p.m. on Wednesday, May 25, 2016.

- The SinkMod Recognition BBQ will be held at the District beginning at 11:30 a.m. on Wednesday, June 8, 2016.
- The Executive Team will be out of the office May 10 & 11, 2016, for a scheduled retreat.

OTHER BUSINESS:

Secretary Kite expressed interest in visiting Oro Loma Sanitary District's levy project. General Manager Eldredge stated a tour would be arranged.

Director Lathi stated she recently attended the California Water Environment Association conference in Santa Clara.

ADJOURNMENT:

The meeting was adjourned at 9:28 p.m. to the next scheduled Regular Board Meeting to be held in the Boardroom on Monday, May 23, 2016, at 7:00 p.m.

SUBMITTED:	ATTEST:
______________________	______________________
REGINA McEVOY SECRETARY TO THE BOARD	PAT KITE SECRETARY

APPROVED:

____

JENNIFER TOY
PRESIDENT

Adopted this 23rd day of May, 2016

DATE: May 16, 2016

TO: Board of Directors - Union Sanitary District

FROM: Paul R. Eldredge, General Manager/District Engineer

SUBJECT: Agenda Item No. 6.a - Meeting of May 23, 2016
Information Item: **April Monthly Odor Report & Financial Reports**

Background

Attached are the Hours Worked and Leave Time by Work Group Reports, and Financial Reports. Staff is available to answer questions regarding information contained in the report.

Work Group Managers

General Manager/Administration	Paul Eldredge	GM
Business Services	Pamela Arends-King	BS/CFO
Collection Services	James Schofield	CS
Technical Support	Sami Ghossain	TS
Treatment and Disposal Services	Armando Lopez	T&D
Fabrication, Maintenance, and Construction	Robert Simonich	FMC

ODOR COMPLAINTS:
There were two odor complaints received for the Treatment Plant during the month of April. Details regarding the two complaints, received from the same Union City resident, are included in the attached Odor Report.

G.M. ACTIVITIES: For the month of April, the GM was involved in the following:

- Participated in the General Manager Check-in Closed Session.
- Attended the SB 1213 Committee Hearing in Sacramento.
- Participated in the New Developments in the Brown Act webinar offered by CSDA.
- Attended the groundbreaking ceremony for the Ohlone College Academic Core Buildings Project.
- Participated in the Newsletter Draft Layout and Content Review Board Workshop.
- Attended the Newark State of the City Luncheon.
- Attended the FY17 Operating Budget Board Workshop.

Attachments: Odor Report
Odor Report Map
Hours Worked and Leave Time by Work Group
Financial Reports

ODOR REPORT
April 2016

During the recording period from April 01, 2016 through April 30, 2016, there were two odor related service requests received by the District.

City: Union City

1. ***Complaint Details:***

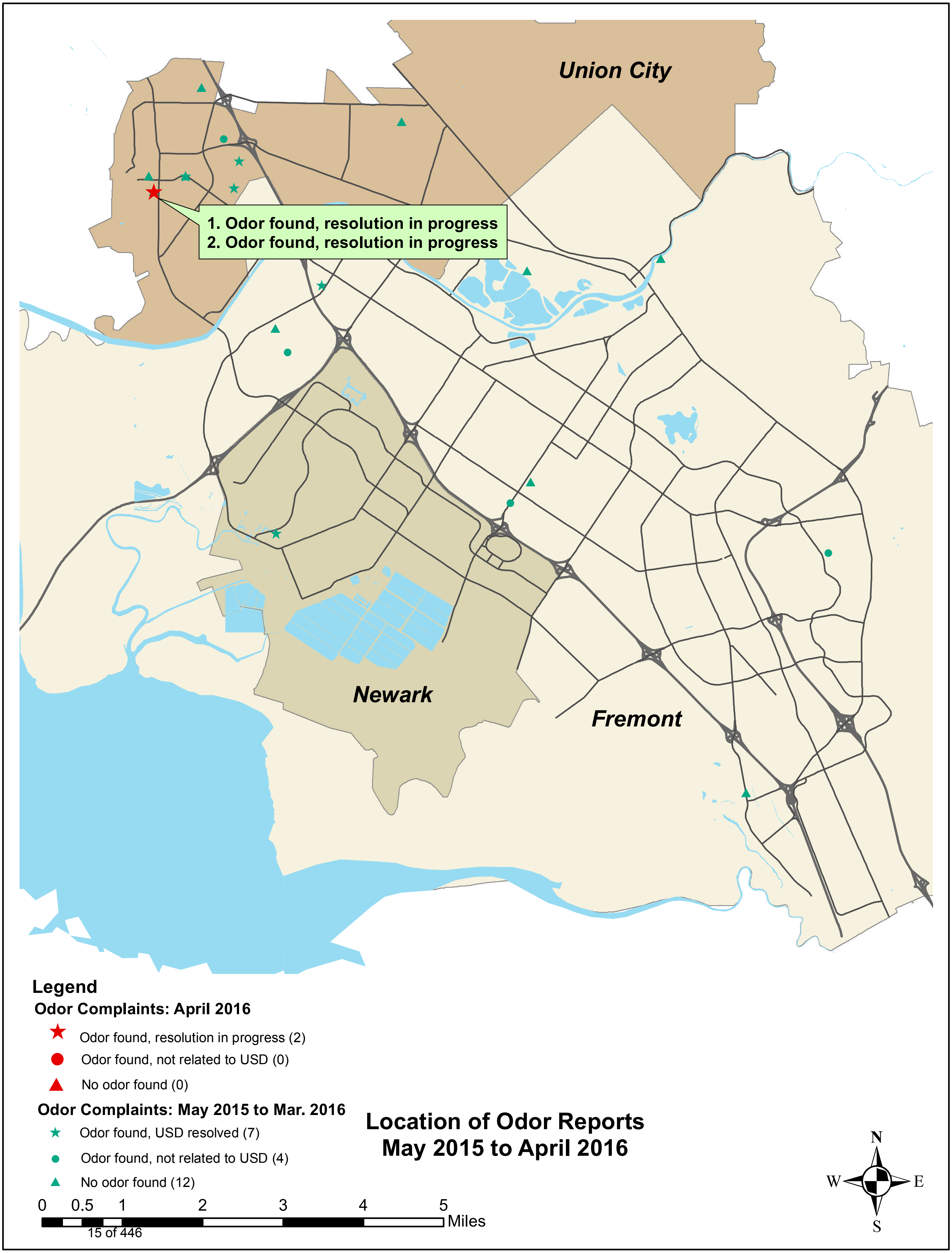
Date: 4/12/2016 — *Time:* 2:00 pm
Location: MACKINAW ST — *Reported By:* Sam Dua
Wind (from): Southwest — *Wind Speed:* 12 mph mph
Temperature: 58 Degrees F — *Weather:* Cloudy

Response and Follow-up:
An Operator and the T&D WGM investigated resident's area. While driving through the neighborhood, an odor was occasionally detected but could not discern what the odor was or where it was coming from. It was also trash day, and the smell of garbage was present. The manhole in front the residence was tested and no H2S was detected.

2. ***Complaint Details:***

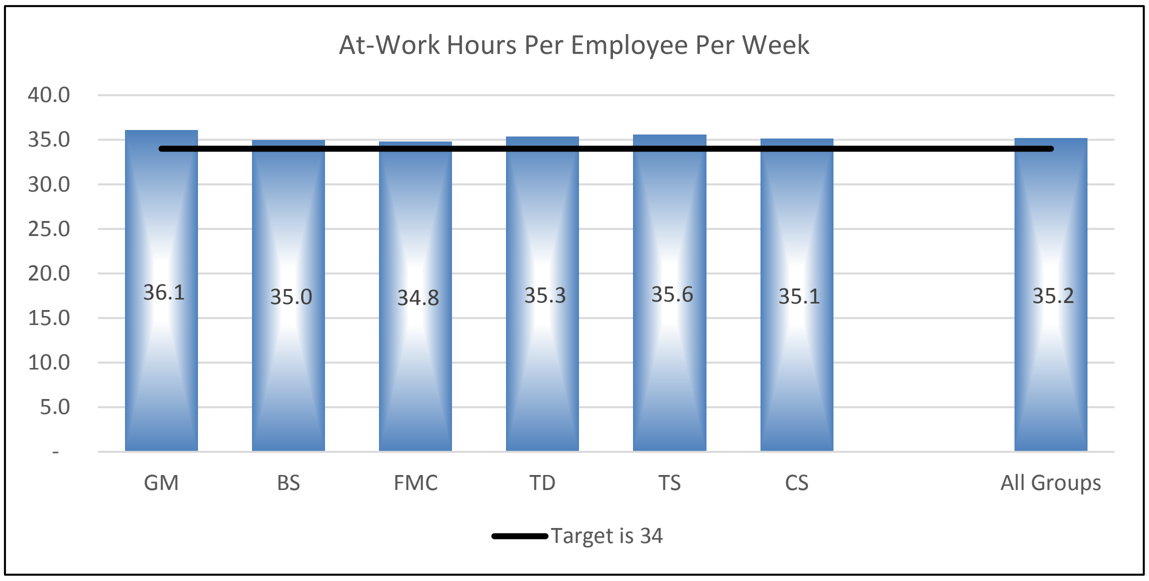
Date: 4/29/2016 — *Time:* 4:21 pm
Location: MACKINAW ST — *Reported By:* Sam Dua
Wind (from): Southwest — *Wind Speed:* 17.9 mph mph
Temperature: 75 Degrees F — *Weather:* Sunny

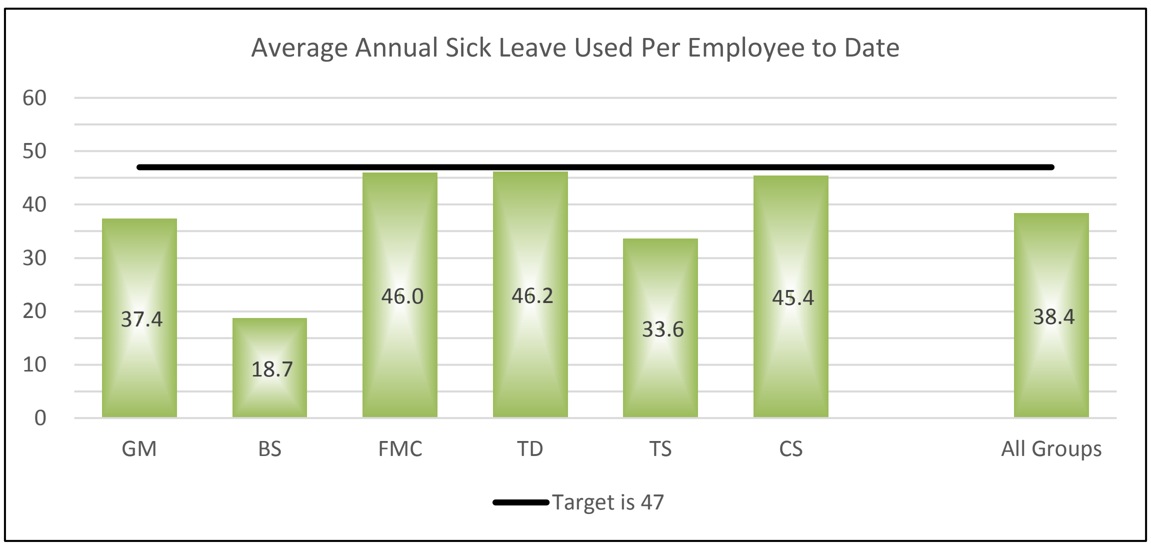
Response and Follow-up:
WGM and TPO Coach were left voicemail messages which were not received until Monday, May 2, 2016. Resident stated there has been an odor for several days. WGM returned phone but had to leave a voicemail. Resident has not returned the call as of the morning of May 5, 2016. R&S Coach stated there was a strong odor on Friday but identified it as a bay/fishy odor that was detected as far as 880/Alvarado Niles Rd.



Union City
1. Odor found, resolution in progress
2. Odor found, resolution in progress
Newark
Fremont
Legend
Odor Complaints: April 2016
Odor found, resolution in progress (2)
Odor found, not related to USD (0)
No odor found (0)
Odor Complaints: May 2015 to Mar. 2016
Odor found, USD resolved (7)
Odor found, not related to USD (4)
No odor found (12)
Location of Odor Reports
May 2015 to April 2016
N
W
E
S
0 0.5 1 2 3 4 5
Miles

HOURS WORKED AND LEAVE TIME BY WORK GROUP
July 2, 2015 through May 4, 2016
Weeks to Date: 44 out of 52 (84.6%)





NOTES

(1) Regular hours does not include hours worked by part-time or temporary employees.

(2) Overtime hours includes call outs.

(3) Discretionary Leave includes Vacation, HEC, Holiday, MAL, FLEX, Funeral, Jury Duty, Military, OT Banked Use, Paid Admin., SLIP, VRIP, Holiday Banked Use leaves.

(4) Sick Leave includes sick and catastrophic sick leaves as well as protected time off, of which the District has no discretion.

An employee using 15 vacation, 11 holiday, 2 HEC, and 5 sick days will work an average of **34.9** hours per week over the course of a year; with 20 vacation days, **34.2** hours per week.

HOURS WORKED AND LEAVE TIME BY WORK GROUP
July 2, 2015 through May 4, 2016
Weeks to Date: 44 out of 52 (84.6%)

Group	Average Number of Employees	AT-WORK HOURS		At-Work Hours Per Employee Per Week	LEAVE HOURS				Average Annual Sick Leave Used Per Employee To Date	FY15		
		Regular (1)	Overtime (2)		Discretionary (3)	Short Term Disability	Workers Comp	Sick (4)		Average Number of Employees	At-Work Hours Per Week Per Employee	Annual Sick Leave Used
GM	2	3,102.25	61.75	36.1	343.00	-	-	74.75	37.4	3	34.4	28.8
BS	22	33,413.77	318.19	35.0	4,859.26	-	-	411.81	18.7	22	35.3	30.2
FMC	22	32,945.25	610.69	34.8	4,274.50	260.37	-	1,011.88	46.0	23	34.2	52.4
TD	25	37,715.92	1,025.91	35.3	4,696.58	333.09	-	1,154.41	46.2	25	35.3	24.1
TS	31	48,011.17	366.81	35.6	5,869.78	18.67	-	1,042.18	33.6	30	35.0	28.1
CS	31	44,971.26	2,750.41	35.1	6,564.42	186.45	324.00	1,408.58	45.4	29	36.8	68.4
All Groups	**133**	**200,159.62**	**5,133.76**	**35.2**	**26,607.54**	**798.58**	**324.00**	**5,103.61**	**38.4**	**132**	**35.3**	**40.8**
SICK LEAVE INCENTIVE PROGRAM TARGETS				**≥34**					**≤47**			

The Sick Leave Incentive Program target goals are 47 or less hours of sick leave per employee annually, and 34 or more hours of at-work time per week per employee.

NOTES

(1) Regular hours does not include hours worked by part-time or temporary employees.

(2) Overtime hours includes call outs.

(3) Discretionary Leave includes Vacation, HEC, Holiday, MAL, FLEX, Funeral, Jury Duty, Military, OT Banked Use, Paid Admin., SLIP, VRIP, Holiday Banked Use leaves.

(4) Sick Leave includes sick and catastrophic sick leaves, as well as protected time off, of which the District has no discretion.

An employee using 15 vacation, 11 holiday, 2 HEC, and 5 sick days will work an average of **34.9** hours per week over the course of a year;
with 20 vacation days, **34.2** hours per week.

BUDGET AND FINANCE REPORT

FY 2016 | **Year-to-date as of 4/30/16** | 83% of year elapsed

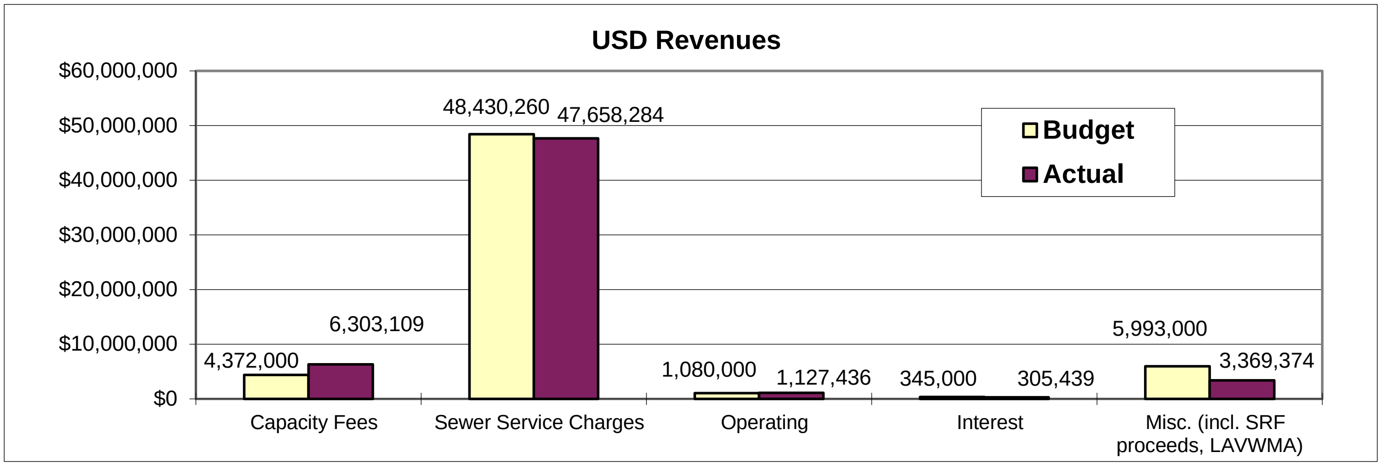
Revenues	Budget	Actual	% of Budget Rec'd	Audited Last Year Actuals 6/30/15
Capacity Fees	$4,372,000	$6,303,109	144%	$4,820,637
Sewer Service Charges	48,430,260	47,658,284	98%	48,379,254
Operating	1,080,000	1,127,436	104%	1,143,435
Interest	345,000	305,439	89%	309,600
Misc. (incl. LAVWMA pymnt, solar, Cogen rebates)	493,000	350,139	71%	2,127,594
Subtotal Revenues	$54,720,260	$55,744,407	102%	$56,780,521
SRF Loan Proceeds (Thickener)	5,500,000	3,019,235	55%	4,501,122
Total Revenues + SRF Proceeds	**$60,220,260**	**$58,763,642**	**98%**	**$61,281,643**

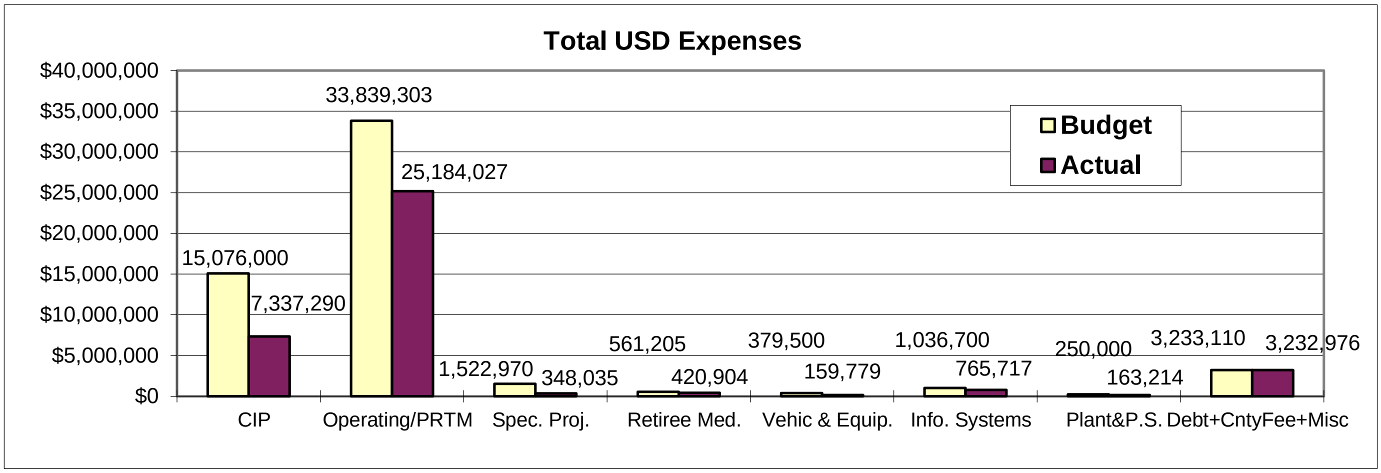
Expenses	Budget	Actual	% of Budget Used	Last Year Actuals
Capital Improvement Prog.				
Capacity Projects	$4,523,000	$2,139,009	47%	$3,755,472
Renewal & Repl. Projects	10,553,000	5,198,281	49%	12,194,927
Operating	33,827,303	25,159,720	74%	30,058,848
Special Projects	1,522,970	348,035	23%	1,065,653
Retiree Medical (Annual Required Contribution)	561,205	420,904	75%	543,540
Vehicle & Equipment	379,500	159,779	42%	787,159
Information Systems	1,036,700	765,717	74%	616,117
Plant & Pump Station R&R	250,000	163,214	65%	168,089
Pretreatment Fund	12,000	24,307	203%	109,499
County Fee for Sewer Service Charge Admin.	106,000	105,866	100%	105,559
Debt Servicing:				
SRF Loans (Irv.,Wilw,LHH,Cdr,NPS, Sub1,Boyc,Prim Cl)	3,127,110	3,127,110	100%	3,127,110
Total Expenses	**$55,898,788**	**$37,611,942**	**67%**	**$52,531,974**
Total Revenue & Proceeds less Expenses	***$4,321,472***	***$21,151,700***		***$8,749,669***

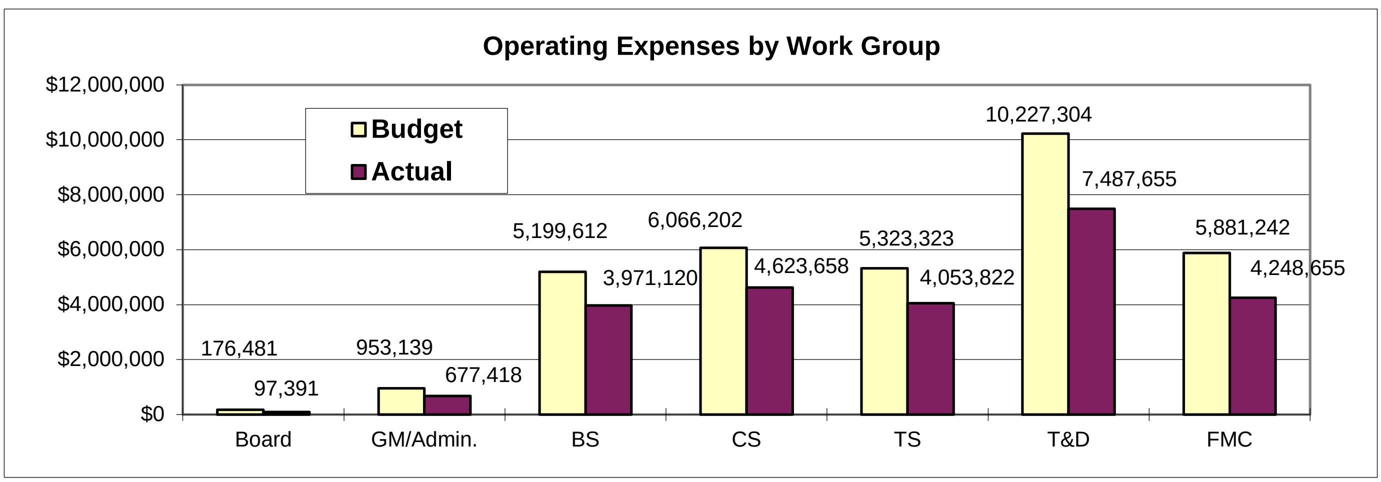
Gross Operating Expenses by Work Group	Budget	Actual	% of Budget Used	Last Year Actuals
Board of Directors	$176,481	$97,391	55%	$135,699
General Manager/Admin.	953,139	677,418	71%	987,502
Business Services	5,199,612	3,971,120	76%	4,460,485
Collection Services	6,066,202	4,623,658	76%	5,447,126
Technical Services	5,323,323	4,053,822	76%	4,693,517
Treatment & Disposal Services	10,227,304	7,487,655	73%	9,172,622
Fabrication, Maint. & Construction	5,881,242	4,248,655	72%	5,161,897
Total	$33,827,303	$25,159,720	74%	**$30,058,848**

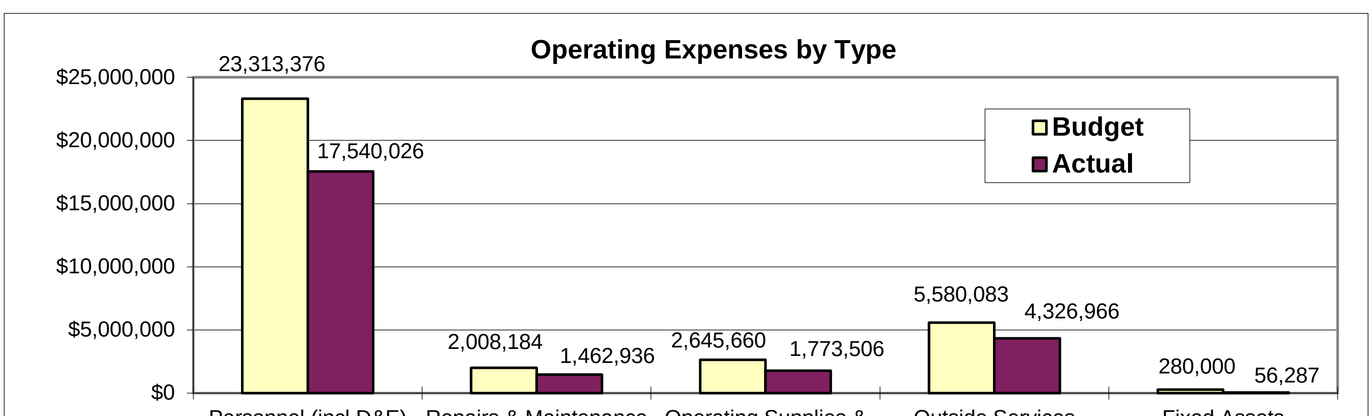
Operating Expenses by Type	Budget	Actual	% of Budget Used	Last Year Actuals
Personnel (incl D&E)	$23,313,376	$17,540,026	75% (81%)*	$20,901,890
Repairs & Maintenance	2,008,184	1,462,936	73%	1,772,819
Supplies & Matls (chemicals, small tools)	2,645,660	1,773,506	67%	2,285,558
Outside Services (utilities, biosolids, legal)	5,580,083	4,326,966	78%	4,961,560
Fixed Assets	280,000	56,287	20%	137,021
Total	$33,827,303	$25,159,720	74%	**$30,058,848**

* Personnel Budget Target

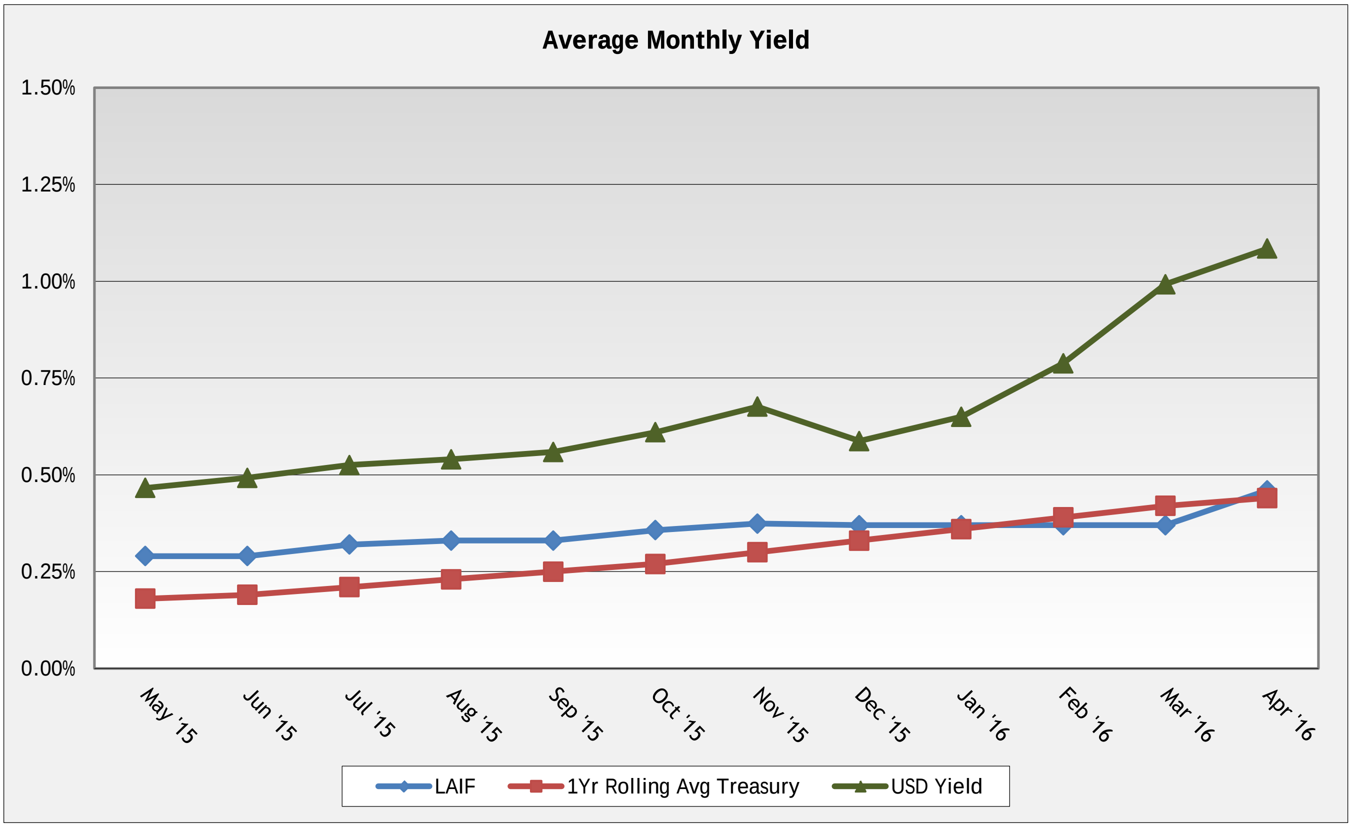

USD Revenues
$60,000,000
$50,000,000
$40,000,000
$30,000,000
$20,000,000
$10,000,000
$0
Budget
Actual
4,372,000
6,303,109
48,430,260
47,658,284
1,080,000
1,127,436
345,000
305,439
5,993,000
3,369,374
Capacity Fees
Sewer Service Charges
Operating
Interest
Misc. (incl. SRF proceeds, LAVWMA)

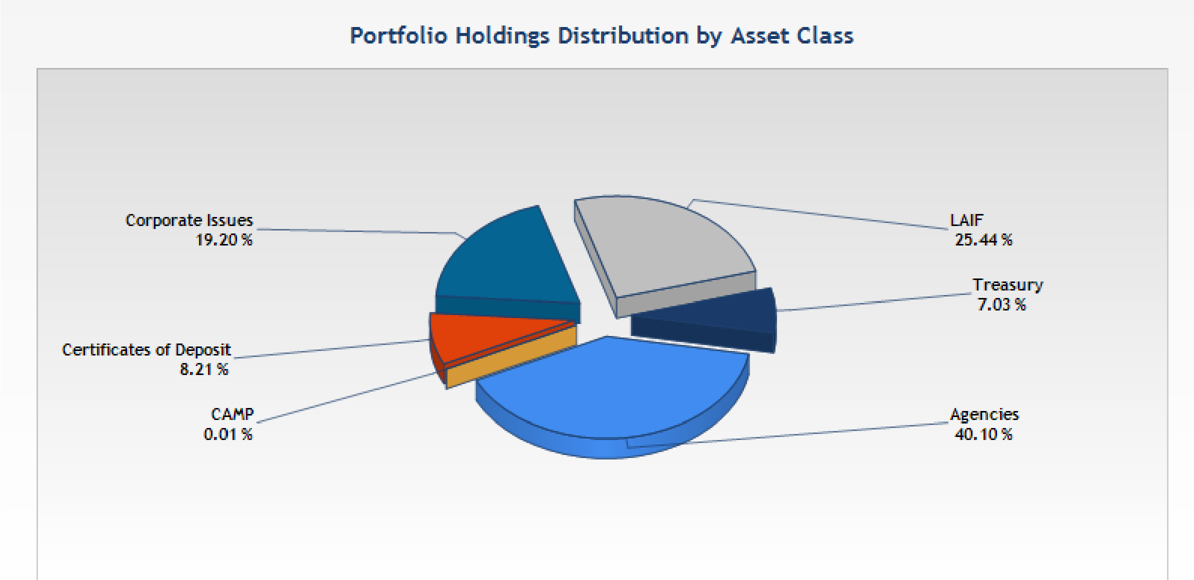

Total USD Expenses
$40,000,000
$35,000,000
$30,000,000
$25,000,000
$20,000,000
$15,000,000
$10,000,000
$5,000,000
$0
Budget
Actual
15,076,000
7,337,290
33,839,303
25,184,027
1,522,970
348,035
561,205
420,904
379,500
159,779
1,036,700
765,717
250,000
163,214
3,233,110
3,232,976
CIP
Operating/PRTM
Spec. Proj.
Retiree Med.
Vehic & Equip.
Info. Systems
Plant&P.S.
Debt+CntyFee+Misc

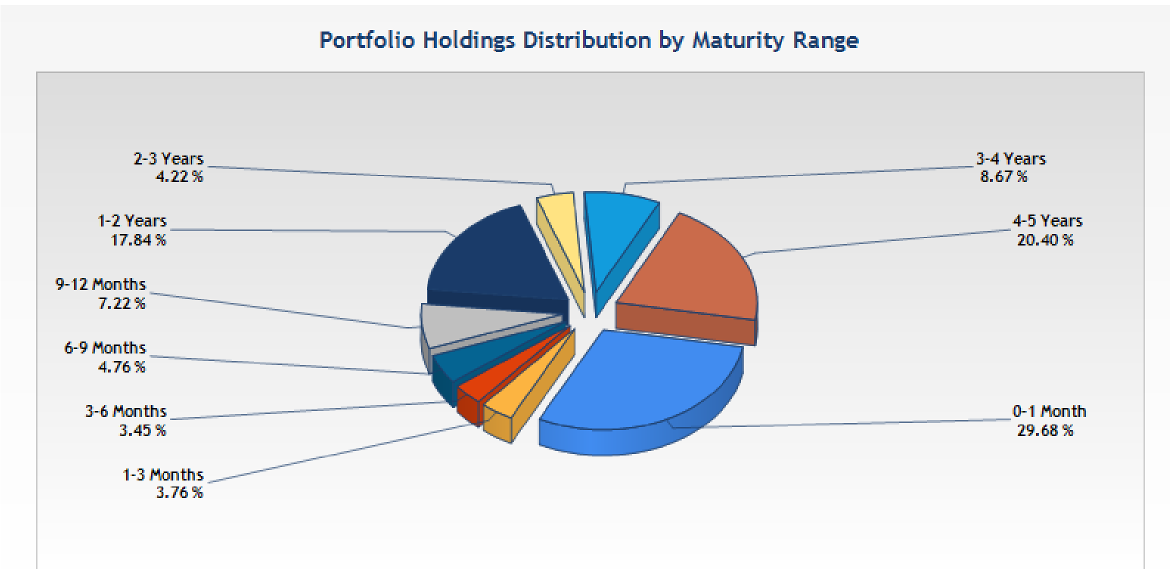

Operating Expenses by Work Group
$12,000,000
$10,000,000
$8,000,000
$6,000,000
$4,000,000
$2,000,000
$0
Budget
Actual
176,481
97,391
953,139
677,418
5,199,612
3,971,120
6,066,202
4,623,658
5,323,323
4,053,822
10,227,304
7,487,655
5,881,242
4,248,655
Board
GM/Admin.
BS
CS
TS
T&D
FMC


Operating Expenses by Type
$25,000,000
$20,000,000
$15,000,000
$10,000,000
$5,000,000
$0
Budget
Actual
23,313,376
17,540,026
2,008,184
1,462,936
2,645,660
1,773,506
5,580,083
4,326,966
280,000
56,287
Personnel (incl D&E)
Repairs & Maintenance
Operating Supplies & Matls
Outside Services
Fixed Assets

Performance Measures for the USD Investment Portfolio







Maturity Range	Face Amount/Shares	YTM @ Cost	Cost Value	Days To Maturity	% of Portfolio	Market Value	Book Value	Duration To Maturity
0-1 Month	21,093,452.87	0.476	21,080,058.43	3	29.68	21,093,292.31	21,093,096.68	0.01
1-3 Months	2,670,000.00	0.691	2,696,840.50	67	3.76	2,674,705.04	2,673,820.56	0.18
3-6 Months	2,455,000.00	0.784	2,454,356.25	150	3.45	2,456,903.11	2,454,693.07	0.41
6-9 Months	3,385,000.00	0.796	3,408,264.00	226	4.76	3,394,286.72	3,393,402.87	0.61
9-12 Months	5,134,000.00	0.891	5,226,325.35	300	7.22	5,180,659.74	5,172,179.74	0.82
1-2 Years	12,676,000.00	0.922	12,694,069.00	512	17.84	12,706,358.42	12,689,090.64	1.39
2-3 Years	3,000,000.00	1.240	3,007,440.00	944	4.22	3,016,570.00	3,007,282.48	2.54
3-4 Years	6,160,000.00	1.619	6,171,676.57	1329	8.67	6,202,218.40	6,171,113.92	3.46
4-5 Years	14,500,000.00	2.103	14,554,327.54	1772	20.40	14,555,515.00	14,553,322.44	4.67
Total / Average	**71,073,452.87**	**1.083**	**71,293,357.64**	**649**	**100**	**71,280,508.74**	**71,208,002.40**	**1.72**

Union Sanitary District
Board Report - Holdings
Report Format: By Transaction
Group By: Asset Class
Portfolio/Report Group: All Portfolios
As of 4/30/2016

Description	CUSIP/Ticker	Credit Rating 1	Settlement Date	Face Amount/Shares	Cost Value	Coupon Rate	Market Value	YTM @ Cost	Next Call Date	Maturity Date	% of Portfolio
Agencies											
FFCB 0.9 9/21/2017	3133EFN78	Moodys-Aaa	3/21/2016	1,000,000.00	1,000,000.00	0.900	1,001,160.00	0.900		9/21/2017	1.40
FFCB 0.93 11/17/2017	3133EFPH4	Moodys-Aaa	11/18/2015	1,000,000.00	999,700.00	0.930	1,001,490.00	0.945		11/17/2017	1.40
FFCB 1.4 4/13/2020-17	3133EF2L0	Moodys-Aaa	4/25/2016	1,000,000.00	998,500.00	1.400	999,280.00	1.439	4/13/2017	4/13/2020	1.40
FFCB 1.58 10/13/2020-16	3133EF2A4	Moodys-Aaa	4/13/2016	1,000,000.00	1,000,000.00	1.580	995,600.00	1.580	10/13/2016	10/13/2020	1.40
FFCB 1.59 3/23/2020-17	3133EFR25	Moodys-Aaa	3/23/2016	1,000,000.00	1,000,000.00	1.590	1,001,220.00	1.590	3/23/2017	3/23/2020	1.40
FFCB 1.68 4/5/2021-17	3133EFX36	Moodys-Aaa	4/29/2016	1,000,000.00	999,300.00	1.680	999,300.00	1.695	4/5/2017	4/5/2021	1.40
FHLB 0.625 11/23/2016	3130A3J70	Moodys-Aaa	12/16/2015	1,000,000.00	999,000.00	0.625	1,000,570.00	0.732		11/23/2016	1.40
FHLB 0.8 5/17/2017	3130A4Q54	Moodys-Aaa	3/27/2015	1,000,000.00	1,001,690.00	0.800	1,001,910.00	0.720		5/17/2017	1.41
FHLB 0.9 9/28/2017	3130A5KH1	Moodys-Aaa	7/22/2015	1,000,000.00	1,001,140.00	0.900	1,002,190.00	0.847		9/28/2017	1.40
FHLB 1 3/29/2018-17	3130A7MB8	Moodys-Aaa	3/29/2016	1,000,000.00	1,000,000.00	1.000	1,000,030.00	1.000	3/29/2017	3/29/2018	1.40
FHLB 1.2 12/20/2018	313383DY4	Moodys-Aaa	4/25/2016	1,000,000.00	1,004,950.00	1.200	1,006,190.00	1.010		12/20/2018	1.41
FHLB 2 4/29/2021-16	3130A7J55	Moodys-Aaa	4/29/2016	2,000,000.00	2,000,000.00	2.000	2,004,760.00	2.000	7/29/2016	4/29/2021	2.81
FHLB Step 2/26/2021-16	3130A76Q3	Moodys-Aaa	2/26/2016	2,000,000.00	2,000,000.00	0.750	2,000,060.00	2.138	5/26/2016	2/26/2021	2.81
FHLB Step 3/15/2021-16	3130A7EG6	Moodys-Aaa	3/15/2016	1,000,000.00	1,000,000.00	1.000	999,370.00	2.216	6/15/2016	3/15/2021	1.40

Description	CUSIP/Ticker	Credit Rating 1	Settlement Date	Face Amount/Shares	Cost Value	Coupon Rate	Market Value	YTM @ Cost	Next Call Date	Maturity Date	% of Portfolio
FHLB Step 4/28/2021-16	3130A7PR0	Moodys-Aaa	4/28/2016	1,000,000.00	1,000,000.00	1.000	998,490.00	2.114	10/28/2016	4/28/2021	1.40
FHLB Step 4/28/2021-16	3130A7QX6	Moodys-Aaa	4/28/2016	1,000,000.00	1,000,000.00	1.250	998,010.00	2.021	10/28/2016	4/28/2021	1.40
FHLMC 0.8 8/25/2017-16	3134G8L49	Moodys-Aaa	2/25/2016	1,000,000.00	1,000,000.00	0.800	1,000,040.00	0.800	5/25/2016	8/25/2017	1.40
FHLMC 1 7/25/2017	3134G3ZH6	Moodys-Aaa	6/24/2015	1,000,000.00	1,004,540.00	1.000	1,003,070.00	0.780		7/25/2017	1.41
FHLMC 1.25 10/28/2019-17	3134G8XQ7	Moodys-Aaa	4/28/2016	1,000,000.00	1,000,000.00	1.250	1,000,090.00	1.250	4/28/2017	10/28/2019	1.40
FHLMC 1.27 3/29/2019	3134G8QB8	Moodys-Aaa	3/29/2016	1,000,000.00	1,000,000.00	1.270	1,001,080.00	1.270		3/29/2019	1.40
FHLMC Step 3/30/2020-17	3134G8ST7	Moodys-Aaa	3/30/2016	1,000,000.00	1,000,000.00	1.000	1,001,130.00	1.744	3/30/2017	3/30/2020	1.40
FHLMC Step 4/28/2021-16	3134G8VZ9	Moodys-Aaa	4/28/2016	2,500,000.00	2,500,000.00	1.250	2,490,425.00	2.116	10/28/2016	4/28/2021	3.51
FHLMC Step 7/28/2020-16	3134G8X20	Moodys-Aaa	4/28/2016	1,000,000.00	1,000,000.00	1.300	997,920.00	1.709	10/28/2016	7/28/2020	1.40
FNMA 0.625 8/26/2016	3135G0YE7	Moodys-Aaa	12/16/2015	1,000,000.00	999,540.00	0.625	1,000,710.00	0.691		8/26/2016	1.40
FNMA 1.25 1/30/2017	3135G0GY3	Moodys-Aaa	12/16/2015	1,000,000.00	1,004,790.00	1.250	1,004,830.00	0.820		1/30/2017	1.41
Sub Total / Average				**28,500,000.00**	**28,513,150.00**	**1.157**	**28,508,925.00**	**1.454**			**39.99**
CAMP											
CAMP LGIP	LGIP4000	None	5/31/2011	9,816.69	9,816.69	0.480	9,816.69	0.480	N/A	N/A	0.01
Sub Total / Average				**9,816.69**	**9,816.69**	**0.480**	**9,816.69**	**0.480**			**0.01**
Certificates of Deposit											
1st Source Bank 0.6 9/15/2016	33646CGK4	None	12/18/2015	245,000.00	244,816.25	0.600	244,934.58	0.701		9/15/2016	0.34
Ally Bank 1 10/24/2016	02006LKM4	None	10/23/2014	240,000.00	240,000.00	1.000	240,388.80	1.000		10/24/2016	0.34

Description	CUSIP/Ticker	Credit Rating 1	Settlement Date	Face Amount/Shares	Cost Value	Coupon Rate	Market Value	YTM @ Cost	Next Call Date	Maturity Date	% of Portfolio
American Express Bank 1.1 10/24/2016	02587CBZ2	None	10/23/2014	240,000.00	240,000.00	1.100	240,400.08	1.100		10/24/2016	0.34
American Express Centurian 1.05 6/5/2017	02587DYJ1	None	6/5/2015	240,000.00	240,000.00	1.050	240,258.00	1.050		6/5/2017	0.34
Bank Hapoalim 0.85 2/17/2017	06251AL65	None	2/18/2016	248,000.00	248,000.00	0.850	248,205.84	0.850		2/17/2017	0.35
Bank of Baroda Ny 0.65 10/27/2016	06062QCS1	None	10/27/2015	245,000.00	245,000.00	0.650	245,057.82	0.650		10/27/2016	0.34
Bank of India NY 0.65 10/26/2016	06279HBX0	None	10/30/2015	245,000.00	245,000.00	0.650	245,023.03	0.650		10/26/2016	0.34
BankUnited NA 0.9 5/24/2017	066519BE8	None	11/24/2015	240,000.00	240,000.00	0.900	240,309.12	0.900		5/24/2017	0.34
Bar Harbor Bank 0.7 1/30/2017	066851TT3	None	6/30/2015	240,000.00	240,000.00	0.700	240,174.24	0.700		1/30/2017	0.34
Capital One Bank 1 10/24/2016	140420QG8	None	10/22/2014	240,000.00	240,000.00	1.000	240,388.80	1.000		10/24/2016	0.34
Capital One National Asso Bank 1.25 8/28/2017	14042E6B1	None	8/26/2015	245,000.00	245,000.00	1.250	246,016.50	1.250		8/28/2017	0.34
Compass Bank 0.95 6/5/2017	20451PLE4	None	6/5/2015	240,000.00	240,000.00	0.950	240,258.24	0.950		6/5/2017	0.34
Discover Bank 0.75 1/3/2017	254672QZ4	None	7/1/2015	240,000.00	240,000.00	0.750	240,175.92	0.750		1/3/2017	0.34
First Niagara Bank 1.1 10/30/2017	33583CSV2	None	10/30/2015	245,000.00	245,000.00	1.100	245,866.81	1.100		10/30/2017	0.34
Goldman Sachs Bank 1 10/16/2017	38148JQX2	None	4/27/2015	240,000.00	239,520.00	1.000	240,844.32	1.069		10/16/2017	0.34
Great Midwest Bank 0.75 7/27/2016	39083PCK6	None	10/27/2014	240,000.00	240,000.00	0.750	240,139.68	0.750		7/27/2016	0.34

Description	CUSIP/Ticker	Credit Rating 1	Settlement Date	Face Amount/Shares	Cost Value	Coupon Rate	Market Value	YTM @ Cost	Next Call Date	Maturity Date	% of Portfolio
Marlin Business Bank 0.85 8/24/2017	57116ALG1	None	2/24/2016	248,000.00	248,000.00	0.850	248,554.03	0.850		8/24/2017	0.35
Medallion Bank 1.15 10/30/2017	58403B2L9	None	10/28/2015	245,000.00	245,000.00	1.150	245,864.12	1.150		10/30/2017	0.34
Merrick Bank 0.9 5/19/2017	59013JLK3	None	11/19/2015	240,000.00	240,000.00	0.900	240,294.48	0.900		5/19/2017	0.34
Patriot Bank 0.65 6/30/2016	70337MAH1	None	12/30/2015	240,000.00	240,000.00	0.650	239,961.36	0.650		6/30/2016	0.34
Safra National Bank 0.7 11/29/2016	78658QSF1	None	11/30/2015	245,000.00	245,000.00	0.700	245,009.80	0.700		11/29/2016	0.34
Santander Bank 0.8 2/17/2017	80280JLS8	None	2/17/2016	248,000.00	248,000.00	0.800	248,206.34	0.800		2/17/2017	0.35
TCF National Bank 0.85 8/17/2017	872278SH0	None	2/17/2016	248,000.00	248,000.00	0.850	248,538.66	0.850		8/17/2017	0.35
Wex Bank 0.85 5/19/2017	92937CDE5	None	11/20/2015	245,000.00	245,000.00	0.850	245,324.14	0.850		5/19/2017	0.34
Sub Total / Average				**5,832,000.00**	**5,831,336.25**	**0.877**	**5,840,194.71**	**0.884**			**8.18**
Corporate Issues											
Caterpillar Financial 1 3/3/2017	14912L5Z0	Moodys-A2	12/23/2014	1,313,000.00	1,307,603.57	1.000	1,315,744.17	1.190		3/3/2017	1.83
Chevron Corp 2.193 11/15/2019	166764AN0	Moodys-Aa2	2/26/2016	1,160,000.00	1,167,806.57	2.193	1,187,828.40	2.004		11/15/2019	1.64
General Electric Capital Corp 5.4 2/15/2017	36962G2G8	Moodys-A1	3/2/2015	1,085,000.00	1,179,514.35	5.400	1,124,049.15	0.890		2/15/2017	1.65
HSBC Holdings 3.4 3/8/2021	404280AV1	Moodys-A1	3/28/2016	2,000,000.00	2,055,027.54	3.400	2,071,580.00	2.800		3/8/2021	2.88
IBM Corp 1.8 5/17/2019	459200JE2	Moodys-Aa3	3/18/2016	1,000,000.00	1,005,370.00	1.800	1,012,670.00	1.624		5/17/2019	1.41
	459200HL8		11/26/2013	1,000,000.00	996,840.00	0.450	1,000,000.00	0.580		5/6/2016	1.40

Description	CUSIP/Ticker	Credit Rating 1	Settlement Date	Face Amount/Shares	Cost Value	Coupon Rate	Market Value	YTM @ Cost	Next Call Date	Maturity Date	% of Portfolio
Internaltional Business Machs 0.45 5/6/2016		Moodys-Aa3									
JP Morgan Chase & Co 2 8/15/2017	48126EAA5	Moodys-A3	2/16/2016	1,000,000.00	1,008,859.00	2.000	1,009,650.00	1.400		8/15/2017	1.42
JP Morgan Securities 0 5/13/2016	46640PED1	Moodys-P1	8/19/2015	1,000,000.00	995,235.56	0.000	999,809.44	0.653		5/13/2016	1.40
Royal Bank of Canada 1.2 1/23/2017	78010UNX1	Moodys-Aa3	10/2/2015	1,000,000.00	1,003,960.00	1.200	1,002,420.00	0.895		1/23/2017	1.41
Royal Bank of Canada 2.3 7/20/2016	78008TLB8	Moodys-Aa3	12/23/2014	1,190,000.00	1,217,310.50	2.300	1,194,284.00	0.830		7/20/2016	1.71
Toyota Motor Credit 1.55 7/13/2018	89236TCP8	Moodys-Aa3	3/16/2016	1,000,000.00	1,002,490.00	1.550	1,009,300.00	1.440		7/13/2018	1.41
US Bankcorp 2.2 11/15/2016	91159HHB9	Moodys-A1	3/31/2015	900,000.00	920,304.00	2.200	906,111.00	0.797		11/15/2016	1.29
Sub Total / Average				**13,648,000.00**	**13,860,321.09**	**2.098**	**13,833,446.16**	**1.375**			**19.44**
LAIF											
LAIF LGIP	LGIP1002	None	4/30/2011	18,083,636.18	18,083,636.18	0.460	18,083,636.18	0.460	N/A	N/A	25.37
Sub Total / Average				**18,083,636.18**	**18,083,636.18**	**0.460**	**18,083,636.18**	**0.460**			**25.37**
Treasury											
T-Bond 0.25 5/16/2016	912828VC1	Moodys-Aaa	1/24/2014	1,000,000.00	994,530.00	0.250	1,000,030.00	0.488		5/16/2016	1.39
T-Bond 0.5 3/31/2017	912828J92	Moodys-Aaa	3/9/2016	1,000,000.00	998,417.43	0.500	999,450.00	0.650		3/31/2017	1.40
T-Note 0.5 6/15/2016	912828VG2	Moodys-Aaa	3/27/2014	1,000,000.00	999,530.00	0.500	1,000,320.00	0.521		6/15/2016	1.40
T-Note 0.875 1/15/2018	912828H37	Moodys-Aaa	6/1/2015	1,000,000.00	1,001,560.00	0.875	1,002,230.00	0.815		1/15/2018	1.40

Description	CUSIP/Ticker	Credit Rating 1	Settlement Date	Face Amount/Shares	Cost Value	Coupon Rate	Market Value	YTM @ Cost	Next Call Date	Maturity Date	% of Portfolio
T-Note 0.875 11/15/2017	912828G20	Moodys-Aaa	6/24/2015	1,000,000.00	1,001,060.00	0.875	1,002,460.00	0.830		11/15/2017	1.40
Sub Total / Average				**5,000,000.00**	**4,995,097.43**	**0.601**	**5,004,490.00**	**0.661**			**7.01**
Total / Average				**71,073,452.87**	**71,293,357.64**	**1.101**	**71,280,508.74**	**1.084**			**100**

All investment actions executed since the last report have been made in full compliance with the District's Investment Policy.
The District will meet its expenditure obligations for the next six months.
Market value sources are the LAIF, CAMP, and BNY Mellon monthly statements.
Broker/Dealers: BOSC, Inc.; Cantella & Co.; First Empire Securities; Ladenburg, Thalman & Co, Inc.; UBS Financial Services; Wells Fargo Securities.

Union Sanitary District
Board Report - Activity
Portfolio/Report Group: All Portfolios
From 4/1/2016 To 4/30/2016

Description	CUSIP/Ticker	Face Amount/Shares	Principal	Interest/Dividends	Coupon Rate	YTM @ Cost	Settlement Date	Total
BUY								
FFCB 1.4 4/13/2020-17	3133EF2L0	1,000,000.00	998,500.00	466.67	1.400	1.439	4/25/2016	998,966.67
FFCB 1.58 10/13/2020-16	3133EF2A4	1,000,000.00	1,000,000.00	0.00	1.580	1.580	4/13/2016	1,000,000.00
FFCB 1.68 4/5/2021-17	3133EFX36	1,000,000.00	999,300.00	1,120.00	1.680	1.695	4/29/2016	1,000,420.00
FHLB 1.2 12/20/2018	313383DY4	1,000,000.00	1,004,950.00	4,166.67	1.200	1.010	4/25/2016	1,009,116.67
FHLB 2 4/29/2021-16	3130A7J55	2,000,000.00	2,000,000.00	0.00	2.000	2.000	4/29/2016	2,000,000.00
FHLB Step 4/28/2021-16	3130A7QX6	1,000,000.00	1,000,000.00	0.00	1.250	2.021	4/28/2016	1,000,000.00
FHLB Step 4/28/2021-16	3130A7PR0	1,000,000.00	1,000,000.00	0.00	1.000	2.114	4/28/2016	1,000,000.00
FHLMC 1.25 10/28/2019-17	3134G8XQ7	1,000,000.00	1,000,000.00	0.00	1.250	1.250	4/28/2016	1,000,000.00
FHLMC Step 4/28/2021-16	3134G8VZ9	2,500,000.00	2,500,000.00	0.00	1.250	2.116	4/28/2016	2,500,000.00
FHLMC Step 7/28/2020-16	3134G8X20	1,000,000.00	1,000,000.00	0.00	1.300	1.709	4/28/2016	1,000,000.00
Sub Total / Average		**12,500,000.00**	**12,502,750.00**	**5,753.34**				**12,508,503.34**
CALLED								
FHLB 0.75 7/28/2017-16	3130A4ZV7	1,000,000.00	1,000,000.00	1,875.00	0.750	0.000	4/28/2016	1,001,875.00
Sub Total / Average		**1,000,000.00**	**1,000,000.00**	**1,875.00**				**1,001,875.00**
DEPOSIT								
CAMP LGIP	LGIP4000	3.85	3.85	0.00		0.000	4/29/2016	3.85
LAIF LGIP	LGIP1002	25,233.78	25,233.78	0.00		0.000	4/15/2016	25,233.78
LAIF LGIP	LGIP1002	20,000,000.00	20,000,000.00	0.00		0.000	4/15/2016	20,000,000.00
Sub Total / Average		**20,025,237.63**	**20,025,237.63**	**0.00**				**20,025,237.63**
INTEREST								
Ally Bank 1 10/24/2016	02006LKM4	0.00	0.00	1,203.29	1.000	0.000	4/23/2016	1,203.29
	02587CBZ2	0.00	0.00	1,323.62	1.100	0.000	4/23/2016	1,323.62

Description	CUSIP/Ticker	Face Amount/Shares	Principal	Interest/Dividends	Coupon Rate	YTM @ Cost	Settlement Date	Total
American Express Bank 1.1 10/24/2016								
CAMP LGIP	LGIP4000	0.00	0.00	3.85		0.000	4/29/2016	3.85
Capital One Bank 1 10/24/2016	140420QG8	0.00	0.00	1,203.29	1.000	0.000	4/22/2016	1,203.29
Goldman Sachs Bank 1 10/16/2017	38148JQX2	0.00	0.00	1,203.29	1.000	0.000	4/15/2016	1,203.29
Great Midwest Bank 0.75 7/27/2016	39083PCK6	0.00	0.00	152.88	0.750	0.000	4/27/2016	152.88
LAIF LGIP	LGIP1002	0.00	0.00	25,233.78		0.000	4/15/2016	25,233.78
Medallion Bank 1.15 10/30/2017	58403B2L9	0.00	0.00	239.29	1.150	0.000	4/28/2016	239.29
Merrick Bank 0.9 5/19/2017	59013JLK3	0.00	0.00	183.45	0.900	0.000	4/19/2016	183.45
Sub Total / Average		**0.00**	**0.00**	**30,746.74**				**30,746.74**
WITHDRAW								
LAIF LGIP	LGIP1002	1,000,000.00	1,000,000.00	0.00		0.000	4/6/2016	1,000,000.00
LAIF LGIP	LGIP1002	2,000,000.00	2,000,000.00	0.00		0.000	4/25/2016	2,000,000.00
LAIF LGIP	LGIP1002	6,000,000.00	6,000,000.00	0.00		0.000	4/27/2016	6,000,000.00
LAIF LGIP	LGIP1002	3,000,000.00	3,000,000.00	0.00		0.000	4/28/2016	3,000,000.00
Sub Total / Average		**12,000,000.00**	**12,000,000.00**	**0.00**				**12,000,000.00**

UNION SANITARY DISTRICT

DATE: **5/23/16**

MEMO TO: Board of Directors - Union Sanitary District

FROM: **Paul R. Eldredge, General Manager/District Engineer**
Pamela Arends-King, Business Services Manager
Sheila Tolbert, HR Manager
Laurie Brenner, Organizational Performance Program Manager

SUBJECT: Agenda Item No. 6b - Meeting of May 23, 2016
Information Item: **Third Quarter FY 16 District-Wide Balanced Scorecard Measures**

Recommendation:
Information Only.

Background:
This report summarizes progress meeting the District's strategic objectives for the third quarter of fiscal year 2015-16 (January 1 through March 31, 2016).

Safety
The District experienced challenges in meeting published safety measures in the third quarter of FY16. Both "Total accidents with lost days" had one additional incident and there were two "Incidents of vehicle or equipment accidents/damage" in Q3. Unfortunately, neither of these measures can now meet their annual targets. "Average FTE lost time" was 0.57 against the goal of <0.5 in Q3, but the measure is only at 0.193 YTD, and not a concern overall.

No safety trainings were offered in Q3, therefore, percent targeted employees receiving that training was NA; however, corrective actions are currently underway to minimize performance shortfalls against established goals by the end of the year in both of these measures. The Training and Emergency Response Programs Manager (TERPM) vacancy continues to negatively impact performance in this area; however, we are in the final stages of hiring the top ranked candidate for the position.

See Table 1: Safety Objectives and Measures, for District performance against all safety measures in Q1.

Operational Excellence

A few measures did not meet published targets in the Operational Excellence scorecard in the third quarter of FY16. “Priority CIP Project milestones” came in at 50% against the target of 85% in Q3, contributing to the 67% YTD value. The District cannot attain the 85% target for the year at this point. Unexpected work delays, priority re-evaluation, and lack of market responsiveness to posted District project bids resulted in this shortcoming.

With only four recorded assessments completed, the “# Competency assessments…” measure in Collections Services (CS) remains substantially behind target at the end of Q3. CS management reports that there are 63 planned assessments in Q4 and states that the annual target will be achieved.

See Table 2: Operational Excellence Objectives and Measures, for District performance against all operational measures in Q1.

Table 1: **Safety Objectives and Measures**

Measures	Q3 FY16	FY16 Target	FY15	FY14	Comments
Total accidents with lost days	1	0	3	1	Two YTD
Other OSHA reportable accidents	0	<4	0	0	
# Incidents of vehicle or equipment accidents/damage	2	<2	3	4	2 in Q3, but at three YTD; exceeds annual target; all plant vehicles with minor damage
Cost associated with vehicle/equipment accidents	$0	<$5000	$444	$7,265	At $540 YTD; well below target
Ave FTE lost time	0.57	<0.5	0.4875	0.05	Still below annual target; 0.193 YTD
"Total Costs: Lost time wages only	$9,645.93	<$46,883	$48,903.84	$4,897	At $9,882.79 YTD
Ave FTE limited duty time	0.32	<0.5	0.53	0	At 0.16 YTD
"Total costs: Limited duty/Other ½ wages	$2,987.01	<$23,441	$26,545.28	0	At $4,775 YTD
X-Mod	1.01	<1.0	1.16	0.95	Improved over last year; now known that next year is 0.72- the lowest in District history
# Facility inspections completed (SIT)	0	4	4	4	Will make up with two inspections in Q4
% of areas of concern identified during internal facility inspections that are resolved within 45 days of report	0	>90%	92%	93%	Since no inspection was done in Q3
# work site inspections completed	81	275	300	323	248 inspections completed at end of Q3
# site visits (for potential BMPS)	0	2	2	2	PepsiCo being scheduled for Q4
# GM communications on status of safety program and performance	2	7	8	7	Q3- Safety survey announcement and "Days Without Injuries" update
# of major safety training events offered	0	7	8	7	Technical Training Program Manager vacancy impacting this measure negatively; final stages of hiring
Ave. % of targeted employees trained	77.8%	>90%	80%	91.8	

Legend for Table 1 and Table 2:

Green: meeting or exceeding target or projected to meet target by the end of the fiscal year
Yellow: Will not meet target if trend continues, and/or not meeting target by <10%- needs attention
Red: Will not meet FY target by >10%- corrective action needed

Table 2: **Operational Excellence Objectives and Measures**

Measures	Q3 FY16	FY16 Target	FY15	FY14	Comments
Progress implementing outreach plan milestones: % planned events completed	19.72%	>90%	94%	98%	Cumulative value now at 67.61% YTD; not a concern against planned activities
Response time to calls for service: % under 1 hour	96.90%	>95%	97.7%	97.1%	Q3- 62/64
"New: Response time to contact USD inquiries:	100%	>90%	96.4%	95%	Q3= 32/32
# Total adverse impacts on customers	0	<10	5	12	None in Q3; at three YTD
# Emergency preparedness events (drills, training, debriefs, etc.)	0	3	5	3	No events planned or held in Q3; can catch up to annual goal with two activities in Q4
Residential SSC compared to surrounding areas	11.50%	<33rd percentile	15.3%	11.50%	
# regional projects/initiatives with financial benefit	3	>3	3	2	
# Critical asset failures w/o negative impacts	0	<2	0	1	
# critical asset failures with negative impacts	0	0	2	0	Despite no issues in Q3, Alvarado sinkhole impacted this measure for the year
Priority CIP Project milestones met vs. planned	50%	85%	92%	100%	5/10 on track in Q3; 67% against the annual target; cannot achieve annual goal
# adverse impacts on environment	0	0	2	1	Despite no issues in Q3, Alvarado sinkhole impacted this measure for the year
# regional projects/initiatives with environmental benefit	3	>3	3	2	
Category 2/3 SSOs	1	< 10	4	4	Q3- 350 gallons spilled at Witherly/Mission Blvd.; 300 gallons recovered *86%). Roots indicated as causal factor
% Training System Milestones Completed (accumulative total)	67%	100%	100%	66%	YTD- FMC= 100%; TPO= 87.5%; CS= 12.5%
# competency assessments completed	4	65	60	22	63 planned for Q4; team indicates they will complete annual target

DATE: May 16, 2016

MEMO TO: Board of Directors – Union Sanitary District

FROM: Paul R. Eldredge, General Manager/District Engineer
Sami E. Ghossain, Manager of Technical Services

SUBJECT: Agenda Item No. 6.c - Meeting of May 23, 2016
Information Item: **Balanced Scorecard Report for the Technical Services Work Group**

Recommendation:

Information only

Background:

In the past two quarters, the Board has received reports from the Treatment and Disposal Work Group and the Collection Services Work Group on the status of their Balanced Scorecards. These two reports were based on the 'process scorecards' developed by the Operating Work Groups.

The Balanced Scorecard for the Technical Services (TS) Work Group has a different look than the Operating Groups Scorecard. Each of the three teams in the TS Workgroup has a different focus, therefore, we have not developed a process scorecard. Instead, each team has developed its strategic objectives represented in their unique scorecards. Each team has selected three to four of its performance measures to present which are most representative of the objectives of that team. The complete scorecard (typically 6-10 measures) is available for the Board's review. A summary of measures is attached, with those presented being highlighted.

The TS Workgroup consists of three teams. Each team is briefly described below:

Capital Improvements Projects Team (CIP Team): The CIP Team is responsible for the implementation of the 10-year Capital Improvement Program. Some of the team's responsibilities are to: develop the projects scopes, develop contracts for consulting services; coordinate input from the operating groups; review plans, specifications and reports; hold public information meetings; resolve disputes during construction; monitor contract status; review, negotiate and approve change orders; and ensure customer satisfaction at the completion of the projects. The team is also responsible for preparing in-house design and for providing construction management services for small projects.

The team measures are focused on internal and external customer satisfaction, management of District funds and successful quality control of capital projects.

Customer Service Team (CST Team): This team has a diverse group of responsibilities focused on meeting the needs of the District's commercial, residential and internal customers. The team's responsibilities include: conducting plan reviews, issuing permits to individuals and developers; construction inspection of new or repair of existing sewers on private property; administering the Sewer Service Charge billing program; collecting Capacity Fees from new developments; reviewing and responding to tri-city environmental planning documents; and responding to customer inquiries related to these responsibilities.

In addition, the CST Team is responsible for the reception area, mail distribution, maintenance of the workroom equipment, dispatching trouble calls received from customers; and assisting other work groups in providing public information via newsletters and press releases.

The team measures are focused on timely completion of plan reviews, dispatching trouble calls and collection of fees, providing quality construction inspection of sewer facilities and providing high-quality customer service to both external and internal customers.

Environmental Compliance Team (EC Team): This team is responsible for the implementation of the District's Industrial Pretreatment, Pollution Prevention, and Public Outreach Programs required as a part of our NPDES Permit, as well as for the $318,000 contract with the City of Fremont for the Clean Water Program. Day-to-day duties of the team include semi-annual site inspections of the 81 permitted Class I and Class II industries; sampling of industrial discharges for compliance with user permit conditions; review of permit applications of new industries; education and training on industrial production and treatment processes; issuing groundwater discharge permits for site clean-up operations; enforcement of Ordinance 36 and other regulations; collection and preparation of information for capacity and sewer service charge fees; inspection of non-industrial commercial businesses; the restaurant FOG program; and a school outreach program.

The EC Team's measures are related to the protection of District workers, facilities and plant from potentially harmful discharges, compliance with Local, State, and Federal regulations and requirements, and developing constructive and professional relationships with our Industrial and Commercial customers.

Attached is an organizational chart of the TS Workgroup.

The balanced scorecards are presented in a format that shows each team's mission statement and three to four objectives, measures and conclusions.

PRE/SEG:ks

Attachments: TS Organizational Chart
TS Teams' BSC Graphs
TS Teams' BSC Measures

Technical Services

Work Group Manager
Sami Ghossain
33

Capital Improvement Projects Team
Raymond Chau
Coach
8

Andrew Baile
Curtis Bosick
Derek Chiu
Kevin Chun
Chris Elliott
Mohammad Ghoury
Thomas Lam
Kristina Silva

Customer Service Team
Rollie Arbolante
Coach
11

Rica Agbuya
Al Bunyi
Lilly DeMelo
Andrew Dupler
Tiffany Douglas (Casual EE)
Mariela Espinosa
Glen Ginochio
John Hwang
Regina McEvoy
Michelle Powell
Nancy Walker

Environmental Compliance Team
Michael Dunning
Coach
11

Doug Dattawalker
Marian Gonzalez
Edda Marasigan
Joe Mendoza
Victor Padilla
Alex Paredes
Aaron Robles
Jose Soto
Ariel Teixeira
Audrey Villanueva
Jason Yeates

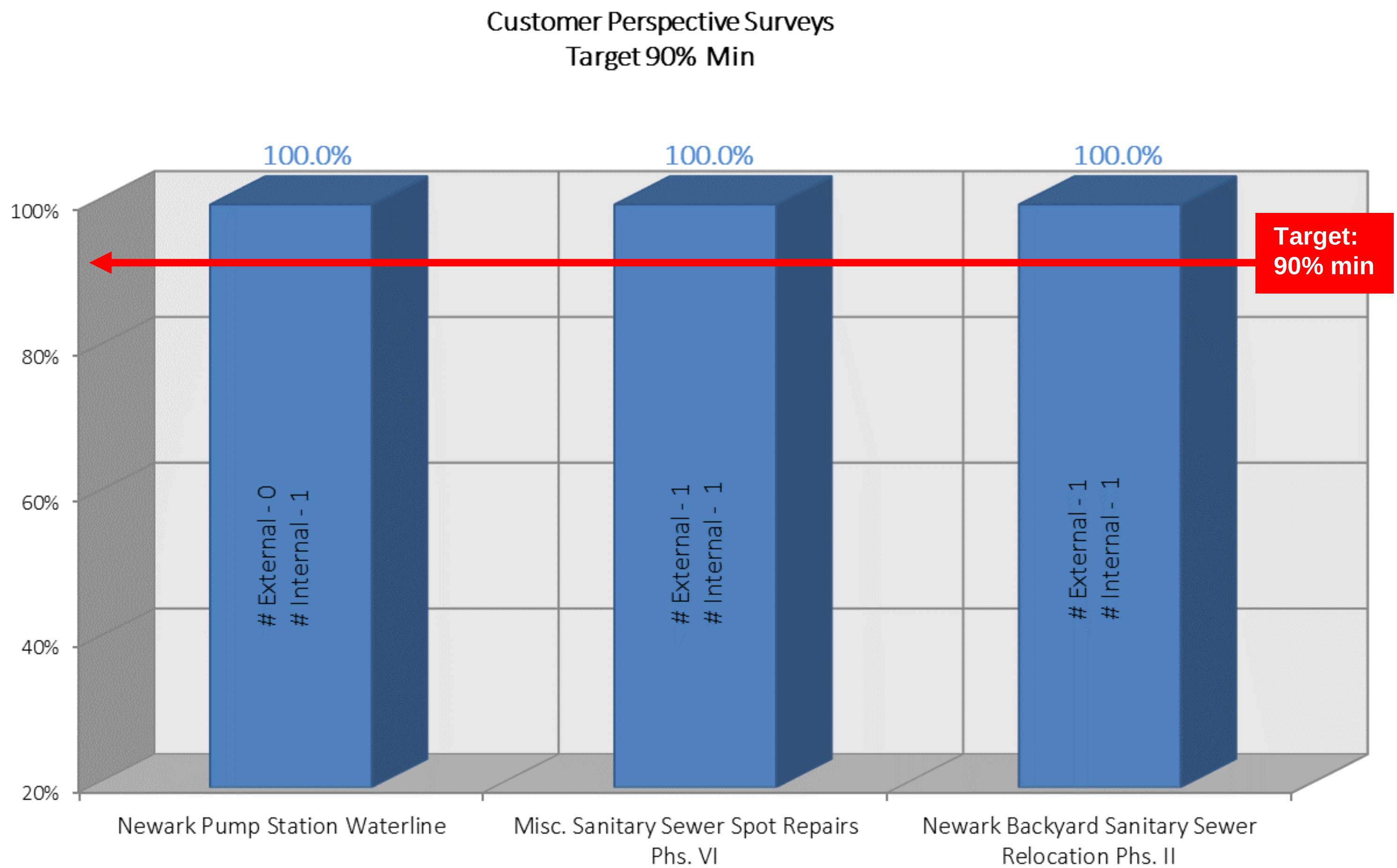
05/02/2016

Capital Improvement Projects Team
Balanced Scorecard—FY 16
May 2016

Mission Statement: The Capital Projects Team is committed to providing effective project management, engineering services, and administrative support for CIP projects and to our customers.

Objective: Deliver quality engineering projects by maximizing customer satisfaction on CIP projects

Measure: Individual project customer survey (operating groups and agencies) regarding communication and responsiveness of project managers (all projects)



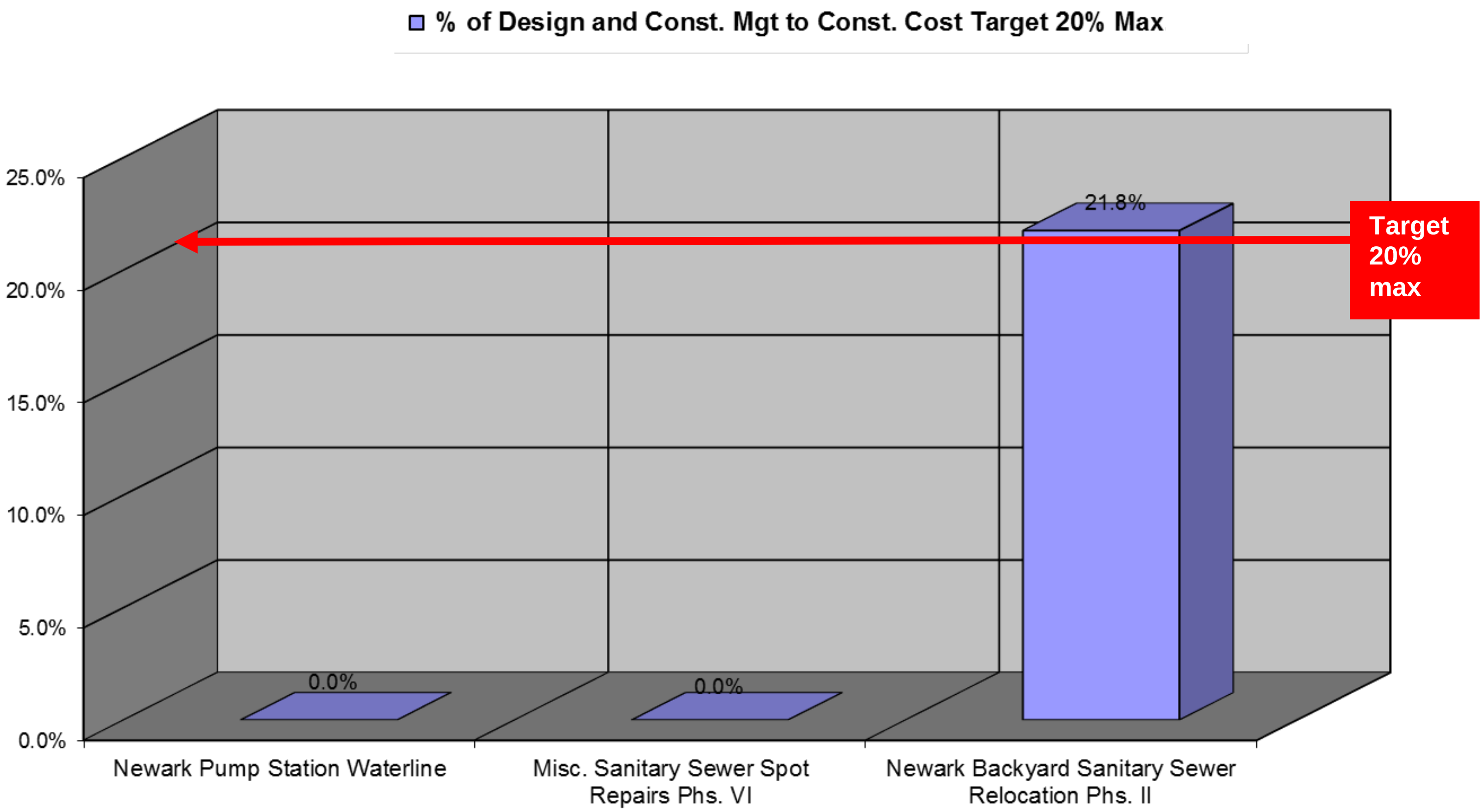
Conclusion: This feedback assists the team in understanding and meeting the expectations of both its internal and external customers.

Capital Improvement Projects Team
Balanced Scorecard—FY 16
May 2016

Mission Statement: The Capital Projects Team is committed to providing effective project management, engineering services, and administrative support for CIP projects and our customers.

Objective: Control cost through effective management of consultants and construction projects

Measure: % of design and construction management costs (final amounts) to construction cost (base bid amount plus change orders and claims)



Notes:

<u>Newark Backyard SS Relocation Ph 2</u> – Due to late completion by contractor, additional compensation to the construction management consultant contract was needed.

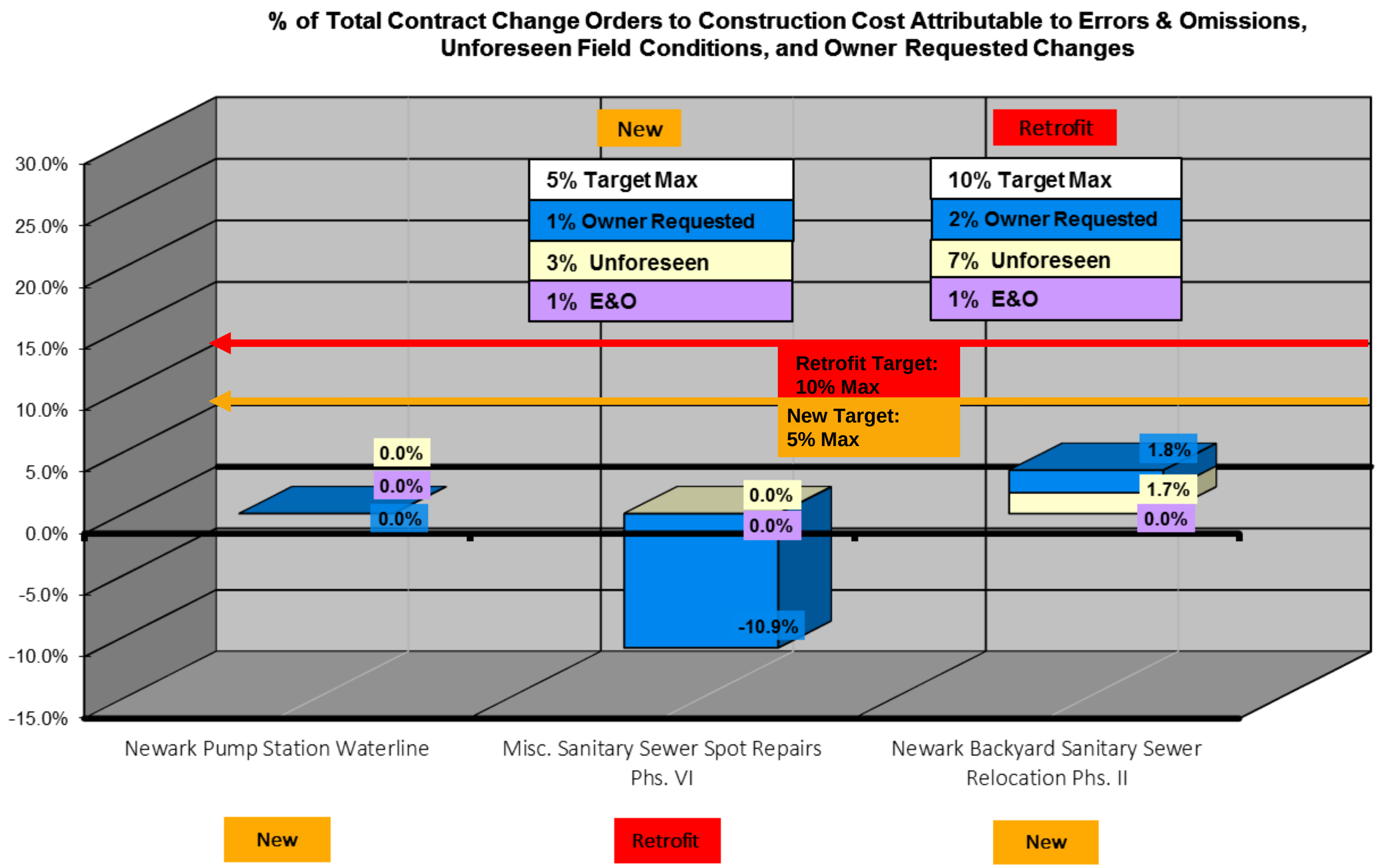
Conclusion: This data will help project managers better understand and control the effort required by consultants to design and manage the construction of CIP projects.

Capital Improvement Projects Team
Balanced Scorecard—FY 16
May 2016

Mission Statement: The Capital Projects Team is committed to providing effective project management, engineering services, and administrative support for CIP projects and to our customers.

Objective: Control cost through effective management of consultants and construction projects

Measure: % of total contract change order amounts (Target is 5% max. for new const., 10% for retrofit) to construction cost (base bid amount)



Notes:

Misc. Sanitary Sewer Spot Repairs Phs. VI – The negative percentage for the Owner Requested CO% is mainly due to the deletion of one project site. The site was deleted as it required ACWD to relocate an existing asbestos concrete water main that was in close proximity of the sewer main repair. ACWD provided an estimate to relocate a portion of that staff found to be too cost prohibitive.

Newark Backyard Sanitary Sewer Relocation Phs. II – The Owner Requested CO% exceeded the 1% target by 0.8%. To safeguard the pipe sag repair, the District required expanded excavation, lightweight fill, settlement monitoring, etc. for the repair.

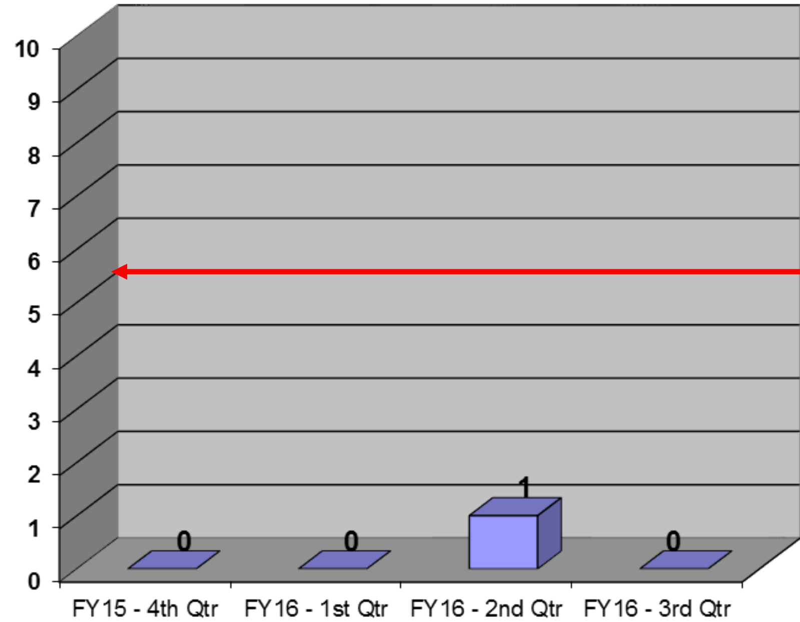
Conclusion: This data will assist staff in minimizing errors and omissions costs by improving quality control and by identifying potential problems during design.

Customer Service Team
Balanced Scorecard—FY 16
May 2016

Mission Statement: To provide high quality service to customers in a courteous and efficient manner; to enforce the District's ordinances and specifications for sewer construction and repairs; to process sewer service charges for properties served by the District; and to provide reception, communication and resource services.

Objective: Timely and accurate collection of fees (SSC, Capacity, and Permit Fees)

Measure: Number and amount of refunds and invoices needed due to administrative oversight

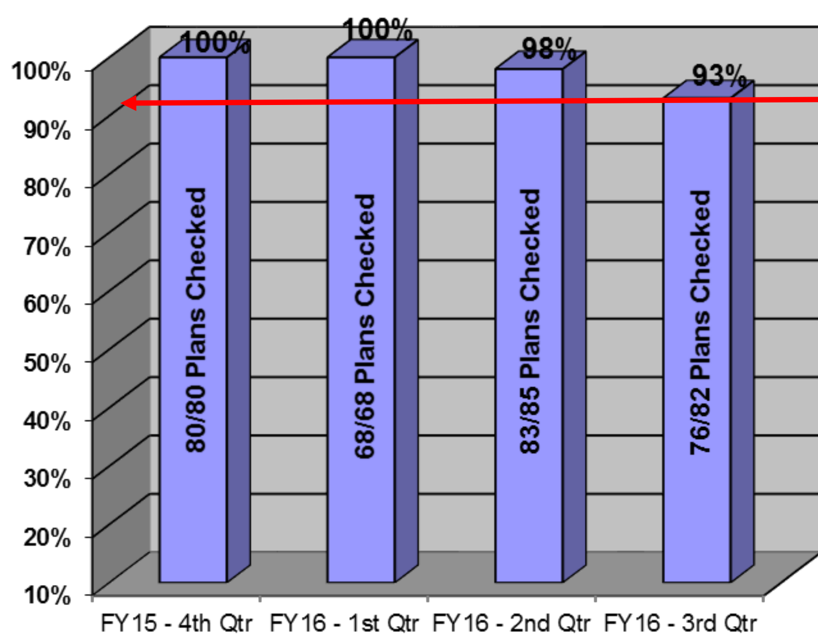


TARGET: Maximum of 5 refunds per year.

Conclusion: The Sewer Service Charge (SSC) database continues to be updated as information is received. Team members are meeting and exceeding the goal which is maximum of five refunds per year.

Objective: Timely plan checking

Measure: % plans checked within 10 working days



TARGET: Minimum 90% of Plans Checked within 10 Working Days

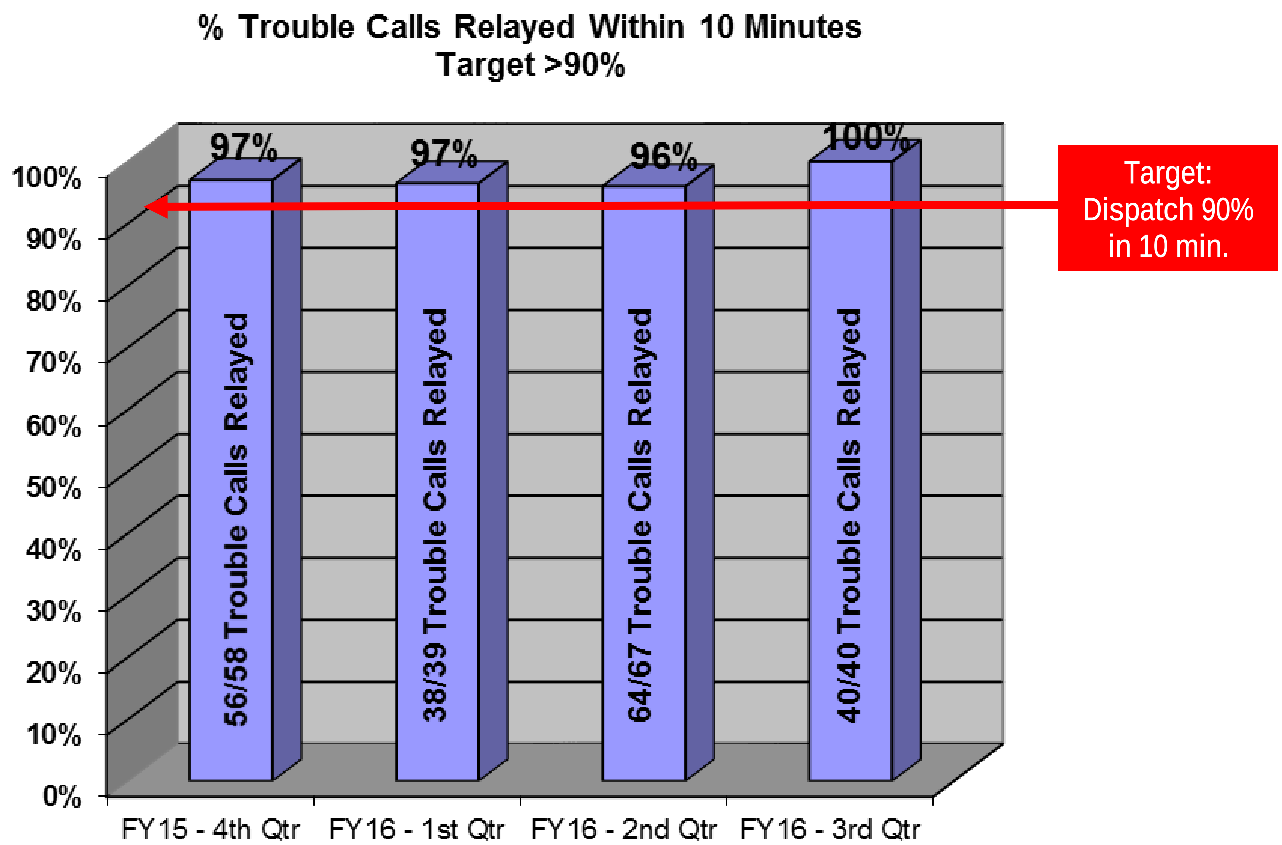
Conclusion: Team members met and exceeded the plan checking goals

Customer Service Team
Balanced Scorecard—FY 16
May 2016

Mission Statement: To provide high quality service to customers in a courteous and efficient manner. To enforce the District's ordinances and specifications for sewer construction and repairs. To process sewer service charges for properties served by the District, and to provide reception, communication and resource services.

Objective: Timely dispatch of trouble calls and relay service requests

Measure: % of calls relayed within 10 minutes



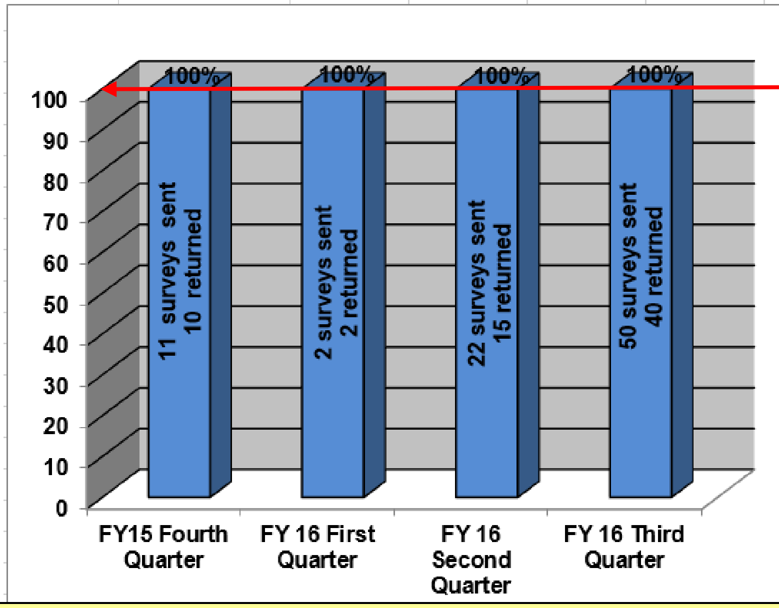
Conclusion: Front desk staff is continuing to dispatch trouble calls within 10 minutes of receiving a call.

Environmental Compliance Team
Balanced Scorecard – FY16
May 2016

Mission Statement: To effectively and efficiently implement environmental protection and compliance programs; to protect District personnel and facilities, public safety, and the environment from deleterious discharges; to preserve resources for beneficial use and reuse; to be responsive to the needs of the District, business community, and the general public; and to provide innovation and leadership in the areas of pollution prevention and industrial and commercial environmental compliance.

Objective: Deliver quality public outreach programs

Measure: % positive responses from teacher surveys

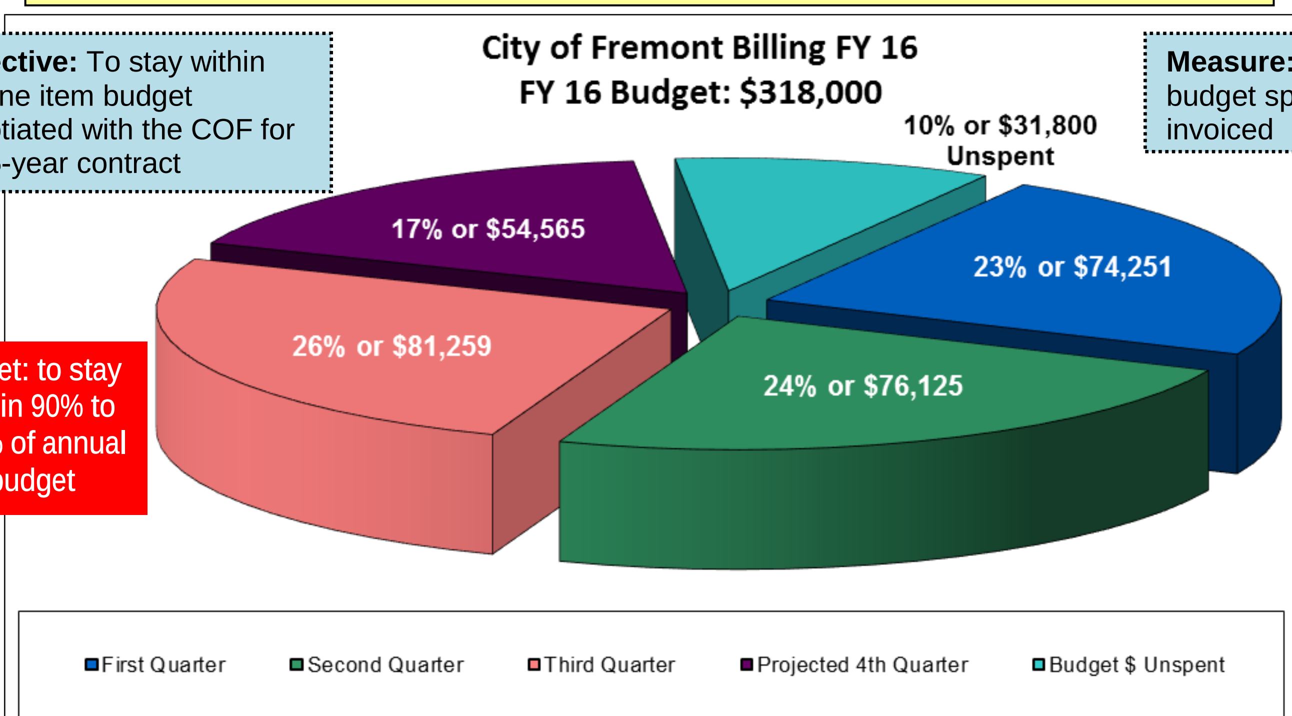


Conclusion: The team continues to provide quality public outreach programs in a professional manner with 100% positive feedback.

Objective: To stay within the line item budget negotiated with the COF for the 5-year contract

Measure: % of budget spent and invoiced

Target: to stay within 90% to 100% of annual budget



Conclusion: Projecting 90 % of annual budget to be used in FY 16

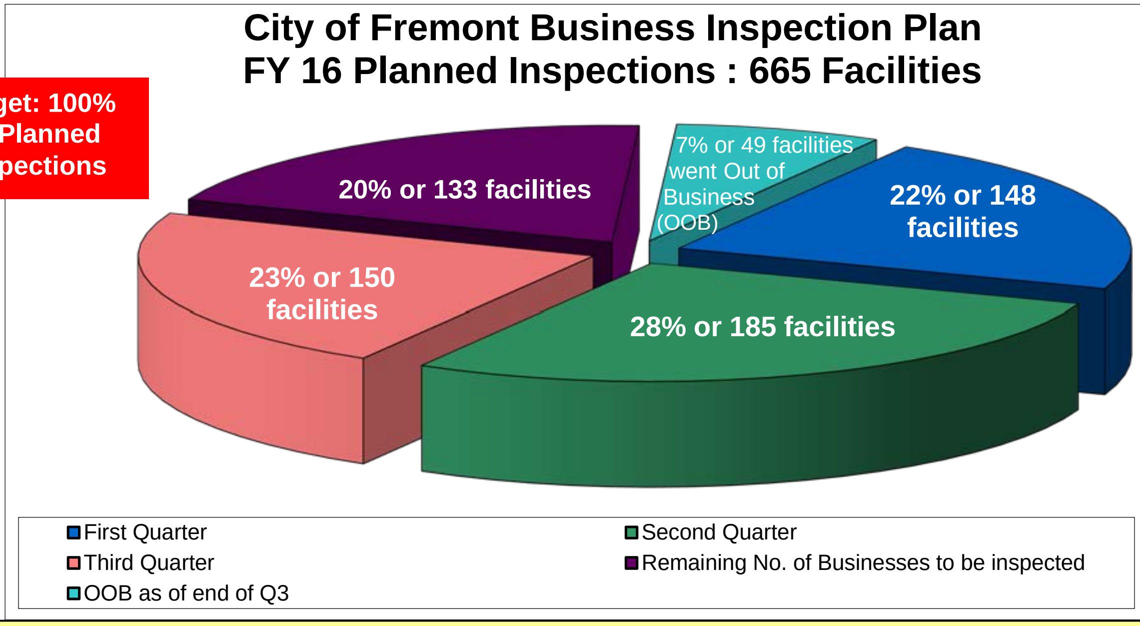
Environmental Compliance Team
Balanced Scorecard—FY 16
May 2016

Mission Statement: To effectively and efficiently implement environmental protection and compliance programs; to protect District personnel and facilities, public safety, and the environment from deleterious discharges; to preserve resources for beneficial use and reuse; to be responsive to the needs of the District, business community, and the general public; and to provide innovation and leadership in the areas of pollution prevention and industrial and commercial environmental compliance.

Objective: Monitor compliance of Commercial / Industrial Businesses

Measure: % of COF business plan facilities inspected

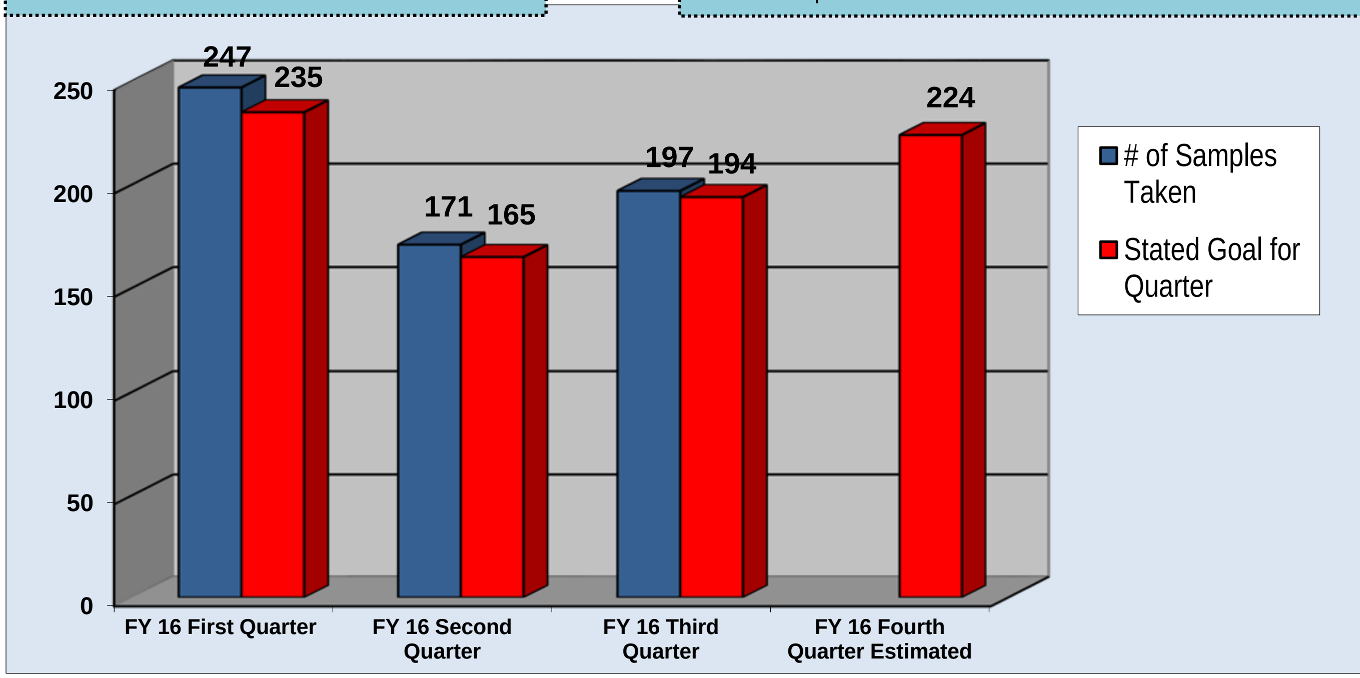
Target: 100% of Planned Inspections



Conclusion: Team is projected to meet the targeted goal for COF contract.

Objective: Monitor compliance of Industrial Businesses

Measure: Sampling events completed based on sampling plan



Conclusion: The team will continue to exceed goals for conducting sampling of industrial sites.

Technical Services Work Group
Team Performance Measures Summary
Fiscal Year 2016

Note Shaded measures are shown in graphs*

TEAM	OBJECTIVE	MEASURE
CIP	**Customer Perspective** Deliver quality engineering projects by maximizing customer satisfaction on CIP Projects	Individual project customer survey (operating groups and agencies) regarding communication and responsiveness of project managers (all projects). **Target: 90% min.**
		Track number and nature of complaints from our external customers, track response time of complaints directed to USD.
	Financial Perspective Control cost through effective management of consultants and construction projects.	% of design and construction management costs (final amounts) to construction cost (base bid amount plus change orders and claims). **Target: 20%**
		% of total contract change order amounts **Target is 5% max. for new const.,** **10% for retrofit** to construction cost (base bid amount). Percentage of Change Orders shall be separated by the following three categories: Errors and omissions – Target 1% max. for new const. and retrofit, Unforeseen field conditions – Target 3% max. for new const.7% for retrofit, Owner requested changes – Target 1% max. for new const.2% for retrofit.
	Internal Processes Maintain communication and education so that there are clearer and more realistic project expectations between Operating Groups and CIP	Internal customers survey (operating groups) regarding communication and responsiveness of project managers (all projects) **Target: 90% min.**
	Quality review and coordination of studies, master plans, and construction documents	Percentage of construction cost (base bid amount) attributable to Contract Change Order amounts due to errors and omissions **Target 1%**
	Employee Growth and Development Perspective Be aware of industry trends to implement efficient and cost effective technologies	# of ideas (training, informational, educational, technological) shared at team meetings – **Target 6 ideas shared/year.**

Technical Services Work Group
Team Performance Measures Summary
Fiscal Year 2016

Note Shaded measures are shown in graphs*

TEAM	OBJECTIVE	MEASURE
Customer Service	**Customer Perspective** Provide professional, courteous and timely services to internal and external customers	% positive responses on customer feedback surveys
	Financial Perspective Timely and accurate collection of fees (SSC, Capacity, and Permit Fees)	Number and amount of refunds and invoices issued due to administrative oversight
	Internal Processes Timely Plan Checking	% plans checked within 10 working days
	Accurate Plan checking and inspection	# of problems reported within one year of approval
	Timely dispatch of trouble calls	% calls relayed within 10 minutes (SLA)
	Employee Growth and Development Perspective Enhance employee skills (computer, new technology, updated regulations, cross-training, etc.)	Number of team members who have attended at least one outside training event (not including mandatory training)

Technical Services Work Group
Team Performance Measures Summary
Fiscal Year 2016

Note Shaded measures are shown in graphs*

TEAM	OBJECTIVE	MEASURE
Environmental Compliance	**<u>Customer Perspective</u>** Provide services in a professional manner with appropriate level of policy enforcement balanced by providing technical information, advice and regulatory requirements.	% of comments from customers during annual evaluation process that indicate fair and professional behavior and responsiveness % positive responses to customer service survey
	Deliver quality Public Outreach Programs	Achieve the P2 Report Goal (40% of 119 classrooms = 48 presentations)
		% of positive comments from teachers
	<u>Financial Perspective</u> Invoice appropriate fees for recovery of cost from enforcement actions.	% of violating industrial users invoiced
	Stay within City of Fremont contract line item budget	% of budget spent and invoiced
	<u>Internal Processes</u> Ensure Industrial and Commercial violations are appropriately addressed	% of violations addressed with corrective measures to achieve compliance with all ordinances.
	Monitor compliance of industrial and commercial businesses	% of COF business plan facilities inspected Sampling events completed based on sampling plan
	<u>Employee Growth and Development Perspective</u> Complete mandatory training	Average percentage of training completed
	Transfer knowledge from external committees and conferences	% of info shared based on number of committees and conferences (info, materials)

UNION SANITARY DISTRICT

DATE: May 16, 2016

TO: Board of Directors - Union Sanitary District

FROM: Paul R. Eldredge, General Manager/District Engineer
Pamela Arends-King, Business Services Manager/CFO
Maria Buckley, Principal Financial Analyst

SUBJECT: Agenda Item No. 9 - Meeting of May 23, 2016
Scheduling Public Hearing to Consider Collection of Sewer Service Charges on the Tax Roll for Fiscal Year 2017

Recommendation:

Set the time for holding the public hearing to consider collection of sewer service charges on the tax roll for fiscal year 2017, at 7:00 p.m. or as soon thereafter as the matter may be heard, on June 23, 2016, in the Boardroom at 5072 Benson Road, Union City, California.

Background:

On January 25, 2016, the Board approved sewer service charge rates for fiscal years 2017 through 2021. The collection of the sewer service charges on the tax roll requires an annual hearing and consideration of the Board. The District may authorize the sewer service charges for fiscal year 2017 to be collected on the tax rolls, consistent with past practices, by 1) creating a report setting forth the amount of the sewer service charges to be assessed on each parcel in the District; 2) filing the report with the Secretary of the Board; 3) scheduling a public hearing for the Board to hear all objections and protests (if any); 4) and authorizing the collection of the sewer service charges on the tax rolls, if there is no majority protest.

If the Board would like to consider placing the sewer service charges for fiscal year 2017 on the tax rolls, it should set the date for the public hearing to consider authorizing the collection. After the hearing is set by the Board, staff will prepare the report to be considered at the public hearing and will publish the attached Notice of the time and place of the hearing in the Argus newspaper on June 3, 2016, and June 10, 2016, and in the Tri-City Voice on June 7, 2016 and June 14, 2016.

UNION SANITARY DISTRICT

NOTICE OF FILING REPORT AND PUBLIC HEARING IN CONNECTION WITH THE COLLECTION OF FISCAL YEAR 2017 SEWER SERVICE CHARGES ON THE PROPERTY TAX ROLL

NOTICE IS HEREBY GIVEN that pursuant to Sections 5471 and 5473, et seq. of the Health and Safety Code of the State of California and Union Sanitary District Ordinance No. 31, the Board of Directors of Union Sanitary District will consider whether to collect its charges for sewer services for fiscal year 2017 on the tax roll, in the same manner as general taxes, consistent with past practices.

The District has filed a written report with the Secretary of the Board of Directors describing each parcel of real property subject to the charges and the amount of the charges against that parcel for fiscal year 2017. The District's report is on file and available for public inspection at the District Offices.

For reference, the charges for a single family home owner (the majority of USD's customers) are based on the adopted rate of $380.05 for Fiscal Year 2017. All other rates for individual customers can be found by contacting the District at (510) 477-7500 or on the Districts website www.unionsanitary.ca.gov/sewerservice.htm

NOTICE IS FURTHER GIVEN that on Monday, the 23rd day of June 2016, at the hour of 7:00 p.m.or as soon thereafter as the matter may be heard, at the Union Sanitary District Boardroom, 5072 Benson Road, Union City, California, in said District, the Board will hold a hearing to consider the report and whether to collect the sewer service charges for fiscal year 2017 on the property tax roll. At the hearing, the Board of Directors will hear and consider all objections or protests, if any, to the District's report. Any questions regarding the charges may be directed to Business Services Manager/CFO Arends-King.

Publish dates: June 3, 2016 – Argus
June 10, 2016 – Argus
June 7, 2016 – Tri-City Voice
June 14, 2016 – Tri-City Voice

By order of the Board of Directors of Union Sanitary District.

DATE: May 16, 2016

MEMO TO: Board of Directors - Union Sanitary District

FROM: Paul R. Eldredge, General Manager/District Engineer
Sami E. Ghossain, Manager of Technical Services
Raymond Chau, CIP Coach

SUBJECT: Agenda Item No. 10 – Meeting of May 23, 2016
Accept the Final Seismic Assessment Reports from Degenkolb Engineers

Recommendation

Staff recommends the Board accept the final seismic assessment reports, dated April 22, 2016, from Degenkolb Engineers.

Background

The District owns and operates a large number of facilities, including 86 structures that were built between 1962 and 2013. The majority of the structures were built in 1978, 1985 or 1995. Nearly all of the structures are reinforced concrete structures with a mixture of precast and cast in place components. The District also operates 26 miles of wastewater forcemains and other large pipelines for the conveyance of wastewater.

On November 25, 2013, the Board authorized the General Manager to execute an agreement and Task Order No. 1 with Degenkolb in the amount of $148,399 to conduct preliminary seismic assessments of the District's structures and major pipelines. The goal of the assessments was to identify major seismic vulnerabilities and determine the serviceability of the District facilities after a major seismic event. Based on the findings on the preliminary assessment, Amendment No. 1 to Task Order No. 1 with Degenkolb in the amount of $62,336 was executed on January 26, 2015, to conduct a detailed seismic assessment of the Administration, Field Operations, Control, and Primary Clarifiers 1-4 Buildings.

Staff presented Degenkolb's findings to the Board during a workshop on March 21, 2016. During that workshop, three (3) seismic performance levels were introduced and discussed. However, the definitions of the performance levels were somewhat unclear. So, staff has provided a summary from the American Society of Civil Engineers on the performance levels definitions, as staff believes they provide more clarity.

Preliminary Seismic Assessment

Design Basis Earthquake

Degenkolb assessed the District structures and major pipelines against the 2013 California Building Code. Degenkolb determined that a 6.3 magnitude earthquake on the nearby Hayward Fault, is the Design Basis Earthquake.

Seismic Performance Levels

Based on staff's input, Degenkolb assessed the structures at the "Life Safety" performance level. The seismic performance levels are defined by the American Society of Civil Engineers in their standard, *Seismic Evaluation and Retrofit of Existing Buildings*. Below the "Operational" performance level, the standard defined three other performance levels that are briefly summarized as follows:

1. Immediate Occupancy – "Immediate Occupancy" means the post-earthquake structural damage is very limited and the risk of life-threatening injury is very low, and, although some minor structural repairs might be appropriate, these repairs would generally not be required before re-occupancy. Continued use of the building is not limited by its structural condition but might be limited by damage or disruption to nonstructural elements of the building, furnishings, or equipment and availability of external utility services.
2. Life Safety –"Life Safety" means the post-earthquake structural damage is significant but some margin against either partial or total structural collapse remains. Injuries might occur during the earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons, this repair might not be practical. Although the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing before re-occupancy.
3. Collapse Prevention –"Collapse Prevention" means the post-earthquake damage state in which the building is on the verge of partial or total collapse. Significant risk of injury caused by falling hazards from structural debris might exist. The structure might not be technically practical to repair and is not safe for re-occupancy because aftershock activity could induce collapse. It is noteworthy to clarify that "Collapse Prevention" may not necessarily allow occupants to evacuate a building safely.

Staff and Degenkolb determined that assessing the structures at a level more stringent than the Life Safety performance level is not necessary since it is not required by code and since retrofitting structures to this level would not be feasible.

Seismic Performance Rating

Staff and Degenkolb selected 25 structures that were a representative sampling of the District's 86 total structures. The structures were grouped according to construction type and vintage of construction. Based on their preliminary seismic assessment, Degenkolb found many of the structures have significant seismic deficiencies and a significant amount of seismic remediation work will be required to protect the structures from extensive damage and potential loss of life in a significant earthquake event. Table 1 lists the 25 structures with their seismic vulnerability rating in descending order; the higher number represents the more vulnerable the structure is. The seismic vulnerability rating is the product of each structure's seismic performance rating and importance rating.

Table 1 – Seismic Vulnerability Rating

Structure Name	Seismic Vulnerability Rating 1 to 100 (A x B)	Seismic Performance Rating 1 to 10 [1] (A)	Importance Rating 1 to 10 [2] (B)
Administration Building	72	8	9
Irvington Pump Station	64	8	8
Field Operations Building	63	7	9
Control Building	60	6	10
Primary Clarifiers 1-4	54	9	6
Primary Clarifiers 5-6	54	9	6
Maintenance Shop Building	54	6	9
Degritter Building	50	10	5
Alvarado Pump Station	48	6	8
EBDA Pump Station	40	5	8
Generator Building 2	36	6	6
EBDA Surge Tower	32	4	8
Paseo Padre Lift Station	30	6	5
Primary Digester 5	25	5	5
Aeration Basins 1-4	24	6	4
Secondary Digester 1	20	5	4
Thickener 1	15	3	5
Chlorine Contact Tank	14	2	7

Lift Station 1	12	3	4
Covered Storage Building	10	5	2
Main Electrical Distribution Building	9	1	9
Heating and Mixing Building 2	8	2	4
Secondary Clarifiers 1-4	8	2	4
Control Box 3	8	2	4
Alvarado Influent Valve Vault	8	1	8

[1] A seismic performance rating of 10 indicates that the structure has a very low probability of meeting "Life Safety" performance in the design basis earthquake.

[2] The importance ratings prioritize mitigating life-loss as a result of a seismic event.

Major Plant Pipelines

Degenkolb found the plant's gravity liquid piping that is made of welded steel pipe with double flexible joints are expected to perform well. The combination of welded steel pipe and double flexible joints should accommodate expected differential settlement without causing pipe failure. However, the pressure piping that is shallow and the sludge piping that connects facilities towards the western side of the plant are subject to differential settlement and pipe failure.

Based on Degenkolb's preliminary assessment of the plant's buried piping, the overall seismic performance rating of the piping is 6 on a scale of 1 to 10. A seismic performance rating of 10 has a low probability the piping's ability to transport flows after an earthquake event. Further study of the piping should be conducted and localized mitigation efforts where flexible joints and coupling were not used should be considered.

Force Mains

The District owns twin 33-inch diameter force mains that convey wastewater between the Irvington Pump Station and the Newark Pump Station, and twin 39-inch diameter force mains that convey wastewater from the Newark Pump station to the plant. The Irvington to Newark force mains are each 40,513 feet in length for a total of 81,026 feet of pipe for both mains. The Newark to Alvarado force mains are each 26,171 feet in length for a total of 52,342 feet of pipe for both mains. The force mains, built in the late 1970s, are constructed with 12-foot segments of reinforced concrete pipe with bell and spigot single-gasketed joints.

The loose, poorly graded sands below the groundwater table are highly susceptible to liquefaction. Considering there are randomly occurring lenses of liquefiable material along the force main corridor, it is estimated that only 25 percent of the corridor along the liquefiable area will liquefy based on the opinion of Degenkolb and their geotechnical and pipeline subconsultants. An average of two inches of settlement is estimated in the areas that liquefy.

However, Degenkolb noted that settlement due to liquefaction of up to three to five inches is expected in the area of the Newark Pump Station, and ½ to 3-½ inches in the vicinity of the plant with an average of one to two inches. Settlement due to liquefaction is also anticipated along the 1,850-foot force main corridor that approaches (from the south) and crosses Alameda Creek located west of Ardenwood Boulevard in Fremont.

The preliminary seismic performance rating for the pipeline at the Alameda Creek is 9, but Degenkolb recommended further investigation is required to better assess the mitigation of liquefaction concerns along this Alameda Creek force main corridor.

Based on the estimated 25 percent of the force mains is located in areas susceptible to an average of two-inch settlement due to liquefaction, Degenkolb estimates a total of nine failures in the force mains in areas beyond the Alameda Creek corridor. Because the expected failures of the force mains beyond Alameda Creek are localized, the seismic performance rating of this section of pipe is 6. The location of these failures will be distributed along the force mains so mitigation of the entire force main would be prohibitively expensive. Degenkolb recommended that the District address this deficiency by enhancing their ability to quickly make repairs. The District can purchase repair sleeves for both the 33-inch and 39-inch force mains and store them, making them available for repair in the days following the earthquake.

<u>Retrofit Estimate</u>

Based on the preliminary investigation, Degenkolb recommended seismic mitigation at the force mains near the Alameda Creek crossing and for structures that are critical for life-safety or the primary transport and disinfection process. Degenkolb estimated that a rough order of magnitude construction cost to seismically upgrade the most vulnerable structures and pipelines will be on the order of $40,000,000 in 2016 dollars.

Detailed Seismic Assessment

Three of the four structures with the highest seismic vulnerability rating in Table 1 are buildings that house the majority of the District's personnel. In order to minimize the number of injuries during an earthquake event, staff decided to pursue detailed seismic assessments of these three buildings and the development of the strengthening schemes to mitigate the structural deficiencies. Additionally, staff included the Primary Clarifiers 1-4 Building in the detailed seismic assessment scope due to its importance in receiving wastewater from the force mains and in providing at least primary wastewater treatment.

Based on the detailed assessments performed by Degenkolb, the nature of the seismic deficiencies and scope of retrofit work required to mitigate those deficiencies are in line with the findings of the preliminary assessment.

In the Administration Building, the major seismic deficiencies are non-ductile braced frames and inadequately braced precast panels. The deficiencies can be mitigated by replacing the existing braces with new buckling restrained braces and bracing the precast panels.

In the Field Operations Building, the major seismic deficiencies are inadequate connections between the roof and the precast panels and the potential for pounding between the two separate structures (office building and the taller warehouse and auto shop). The deficiencies can be mitigated by improving the diaphragm to precast panel connection and reducing the anticipated displacement between the structures with new exterior buttresses.

The Administration and Field Operations Buildings were designed to the 1994 Uniform Building Code before significant code revisions were made as a result of the Northridge Earthquake (1994) and Kobe Earthquake (1995).

In the Control Building, the major seismic deficiencies are inadequate shear walls, discontinuous shear walls and diaphragms. The deficiencies can be mitigated by strengthening the existing shear walls with plywood, strengthening the diaphragm with plywood, and strengthening the connections at the discontinuous walls and diaphragms.

In the Primary Clarifiers 1-4 Building, the major seismic deficiencies are the inadequate inter-connection between adjacent precast roof beams and the connection between the precast walls to the roof and cast-in-place concrete walls below. The deficiency can be mitigated by improving the connections of precast beams and precast wall panels.

Degenkolb prepared a rough order of magnitude cost estimate for the seismic strengthening schemes that is summarized in Table 2.

Table 2 – Seismic Strengthening Construction Cost Estimate

Building	Area (sf)	Cost/sf	Cost
Administration Building	28,328	$166	$4.7 Million
Field Operations Building	19,065	$79	$1.5 Million
Control Building	11,855	$160	$1.9 Million
Primary Clarifiers 1-4 Building	26,430	$129	$3.4 Million
		Total	**$11.5 Million**

Next Steps

Based on the assessment results, staff will proceed with the following next steps:

1 Proceed with the design phase for the retrofit of the Administration Building in FY 17. This will also include remodeling of the existing Maintenance Shop Building that will house some of the personnel when the construction in the Administration Building begins. Construction will begin after the new FMC Building is complete.
2 Proceed with the design phase for the retrofit of the Field Operations Building, the Control Building, and the Primary Clarifiers 1-4 in FY 22.
3 Conduct further geotechnical investigation along the force main corridor near the Alameda Creek to address liquefaction concerns. This can be included in the force main condition assessment that is tentatively scheduled for FY 17.
4 Conduct further study of the plant buried piping and develop localized mitigation efforts where flexible joints and coupling were not used.
5 Conduct additional preliminary and detailed seismic assessments of District structures. Staff will review the next structures to include in the assessments in FY 17.

PRE/SEG/RC:ks

Attachments: Seismic Vulnerability Assessment Final Report
Detailed Seismic Assessments & Conceptual Strengthening Concepts Final Report

Degenkolb

Union Sanitary District

Seismic Vulnerability Assessment
Union City, California

FINAL REPORT

April 22, 2016
Degenkolb Job Number B3215013.00

REGISTERED PROFESSIONAL ENGINEER
ROGELIO S. PARRA
No. 4510
Exp. 12-31-17
STRUCTURAL
STATE OF CALIFORNIA

Roger S. Parra

This report has been prepared solely for the benefit of Union Sanitary District and is not for any other person or entity. Third party use and/or reliance or information contained in the report is at the third party's sole risk.

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Executive Summary

This report summarizes the findings from an initial seismic assessment of the Union Sanitary District's major pipelines and a representative sampling of its structures. This report discusses the vulnerability that the Union Sanitary District's major pipelines and structures have with respect to a significant seismic event, and then discusses how these seismic vulnerabilities can be mitigated. Based on the conclusions of this study, we believe that some of the structures and pipeline sections critical to delivering service to system are seismically vulnerable and therefore damage could be expected following a large seismic event. From discussions with Union Sanitary District management, it has been determined that protecting loss of life during the seismic event and restoring a minimal level of service shortly following a seismic event should be the primary targets of seismic mitigation efforts. Consequently, this report rates structures and pipeline sections based on seismic vulnerability and relative importance to inform a targeted mitigation effort. The most critical structures to retrofit are important structures with significant occupancy and seismically venerable structures with pre-cast concrete roofs. The main force main pipelines are expected to perform relatively well in a seismic event except at the Alameda Creek crossing. Based on this limited preliminary study, we estimate that a rough order of magnitude construction cost to seismically upgrade the most vulnerable structures and pipelines will be on the order of $40,000,000.

1.0: Introduction

The Union Sanitary District is located in a seismically active region that could see strong ground shaking from earthquakes on the Hayward, San Andreas and Calaveras faults. Of the three faults, the nearby Hayward fault has the greatest chance of generating an earthquake strong enough to produce significant ground shaking at the Union Sanitary District site. Because the magnitude of an expected earthquake at the Union Sanitary District site increases based on the length of the time period considered, it is necessary to define the magnitude of the earthquake for the purposes of structure and pipeline assessment. This study assessed the structures and major pipelines for an earthquake equal to the level of earthquake that a new building per the 2013 California Building Code would be designed for. This earthquake represents approximately a M6.3 magnitude earthquake on the nearby Hayward Fault, assuming that the fault ruptures nearby the Union Sanitary District Site. This level of ground motion at the site is expected to occur roughly every 200 years, and therefore has a significant chance of occurring within the lifetime of the structures and pipelines. This earthquake is commonly referred to as the “DBE” (“Design Basis Earthquake”) throughout this report.

The demands on the Union Sanitary District’s systems following this “DBE” seismic event will be driven by the loads from the Alameda County Water District (ACWD). It is our understanding that ACWD has conducted seismic assessments for earthquakes with similar magnitudes as the DBE earthquake used for this study. Based on discussions with ACWD personnel, we would expect to see some user discharge to the sewer system within 3 days following the DBE event associated with provision of impaired water service to critical customers and near normal user discharge to the sewer system within 8 days following the event associated with impaired flow to customers. Consequently, it will be desirable to be able to restore some process capacity to the Union Sanitary District system within the days following the significant seismic event.

2.0: Importance Rating and Seismic Vulnerability

To establish a targeting seismic mitigation approach, it is necessary to establish the relative importance of different structures and pipelines. Based on discussions with USD Management, the following importance ratings have been established. These importance ratings prioritize mitigating life-loss as a result of a seismic event, and protecting pipelines and structures that are critical to the primary transport and primary disinfection of sewage.

Table 1: District Facilities and Importance Ratings

Importance Rating	Categories
10	**Life Safety – occupied full time** • Control Building
9	**Life Safety – occupied during working hours** • Administrative Building • Field Operations Building • FMC Building (includes Generator Room No. 1) • Paint Shop
9	**Electrical – EBDA PS** • Substation Nos. 1, 2, and 4 • Main Distribution Building (Generator Room No. 3)
8	**Electrical – Pump Stations** • Generators at pump and lift stations
8	**Hydraulic transport** • Force Mains • Force Main ARV, blowoff, and access manholes • Surge Towers (IPS, NPS, EBDA) • Irvington PS • Newark PS • Hayward 60" valve vault • Alvarado Influent PS • EBDA PS • Reclaimed Water Room (This is part of the EBDA PS building) • EBDA Effluent Valve Vault • Alvarado Force Main Influent Valve Vault • Newark PS Influent Valve Vault • Newark PS Effluent Valve Vault • Irvington PS Valve Vault • Emergency Outfall Control Valve Structure
7	**Disinfection** • Chlorine Contact Tank • Odor Control Building • Odor Control Building (OCB) Chemical Containment
7	• Boyce LS
6	**Primary Treatment** • Control Box Nos. 1 and 2 • Headworks • Primary Clarifier Nos. 1 thru 4 (includes Sludge Pump Room No. 1) • Primary Clarifier Nos. 5 and 6 (includes Sludge Pump Room No. 3) • Site Waste PS
6	**Electrical – Other** • Generator Building No. 2 • Cogeneration Building

Importance Rating	Categories
5	**Sludge Treatment – Phase 1** • Degritter Building • Thickener Nos. 1 thru 4 • Thickener Control Building • Primary Digester Nos. 1 thru 6 • Sludge Transfer Tank
5	• Cherry PS, Fremont LS, Paseo Padre LS • Irvington PS Equalization Storage Tank
4	**Secondary Treatment** • Lift Station Nos. 1 and 2 • Aeration Basin Nos. 1 thru 4 • Aeration Basin Nos. 5 thru 7 • West Aeration Blower Building • East Aeration Blower Building • RAS Pump Station • Control Box Nos. 3 and 4 • Secondary Clarifier Nos. 1 thru 4 and Sludge Pump Room No. 2 • Secondary Clarifier Nos. 5 and 6 and Sludge Pump Room No. 4 • Hayward Marsh (Dechlorination Facility, Parshall Flume)
4	**Sludge Treatment – Phase 2** • WAS Thickening Building • Centrifuge Building • Secondary Digester Nos. 1 and 2 • Heating and Mixing Building Nos. 1 thru 4 • Heating and Mixing Building #1 Electrical Room
2	**Truck Storage** • Covered Storage • Solar Carport • Fuel Island
1	**Miscellaneous** • Safety Center • FMC Mgmt Trailer • IPS Satellite Trailer • IPS Emergency Storage Pond • INKA MCC Building (will be demolished soon) • Alvarado Influent PS Flow Meter Pit

Coupled with the relative expected seismic performance of a structure, a "seismic vulnerability rating" can be established. A very critical structure that is expected to perform well in a seismic event would not be as high a priority to retrofit as a somewhat critical structure that is expected to collapse in a seismic event. Likewise, a non-critical structure that may sustain heavy damage in a seismic event would not be as high a retrofit priority as a critical structure that might sustain moderate damage in a seismic event. Therefore, a "seismic vulnerability rating" for a structure or pipeline is the product of its expected seismic performance rating (with higher values indicating worse performance) and its importance rating. The higher the seismic vulnerability, the more critical it is to seismically retrofit the structure.

3.0: Structure Assessment

3.1 Structure Performance Summary

This section summarizes the findings from an initial seismic screening of 25 structures at the Union Sanitary District sites. The 25 structures were selected as a representative sampling of the 86 total structures that comprise the Union Sanitary District structural stock. Based on a preliminary review of the drawings for each structure, the structures were grouped according to construction type and vintage of construction. From discussions with Union Sanitary District staff, we selected one structure from each group (typically the most critical structure for day-to-day operations) on which to perform an initial seismic screening. This structure serves as the “archetypal” structure for the group. The findings from each of these screenings can be found in the *Structure Assessment and Possible Seismic Remediation* section of this report.

Based on our preliminary seismic assessment of these 25 structures, we have found that many of the structures have significant seismic deficiencies and a significant amount of seismic remediation work will be required to protect Union Sanitary District’s structural stock from extensive damage and potential loss of life in a significant earthquake event. While the 25 preliminary seismic assessments that were conducted on each archetypal give us a general understanding of the vulnerability of the district’s structural stock, they do not necessarily give us an idea of what remediation work will be required for each individual structure. For example, we have evaluated a number of structures that have been previously seismically retrofitted, and have found seismic deficiencies based on our evaluation to the current standards. Consequently, it is not always appropriate to extrapolate the deficiencies of each archetypal structure (or lack of deficiencies) to the other “similar” structures.

In an effort to better understand how our 25 evaluations fit into the larger vulnerability of the site, we have done a very cursory review of the documents in each group of structures. We have then used our engineering judgment and the knowledge from the evaluations on the archetypal structures to document how reasonable we believe it is to extrapolate the findings of the archetypal structure to that group as a whole. Where we believe it is unreasonable to extrapolate our findings, we have noted that further assessment of more structures within the group is recommended. These discussions can be found in the *Structure Assessment and Possible Seismic Remediation* section of this report.

Each of the 25 structures considered as part of this study has been given a seismic performance rating 1 through 10, and an importance rating 1 through 10. The seismic ratings are based on our seismic assessments, and the importance ratings are based on our discussions with Union Sanitary District Management per the previous section of this report. The product of these ratings identifies the “seismic vulnerability rating” for the structure. This information is summarized in Tables 2a and 2b below. Note that the both the seismic performance rating scale is relative, not absolute. As more structures at the Union Sanitary District are assessed, the ratings of the 25 structures included in Tables 2a and 2b may be adjusted to reflect a more complete knowledge of the structural stock. Table 2a is sorted by seismic performance rating, while Table 2b is sorted by seismic vulnerability rating.

Table 2a: Seismic Vulnerability Summary – Sorted by Seismic Performance Rating

UNION SANITARY DISTRICT - SEISMIC VULNERABILITY SUMMARY					
	Structure Name	**Structure Type**	**Seismic Performance Rating**	**Importance Rating**	**Seismic Vulnerability Rating**
NOT LIFE SAFE	Degritter Building	Precast Concrete Walls and Roof (1985)	10	5	50
	Primary Clarifiers 5-6/Sludge Pump Room #3	CIP concrete walls below grade, Precast Concrete walls and roof (1978/1985)	9	6	54
	Primary Clarifiers 1-4/Sludge Pump Room #1	CIP concrete walls below grade, Precast Concrete walls and roof_Primary Clarifiers (1978)	9	6	54
	Irvington Pump Station	CIP concrete walls below grade, CMU Walls with steel framed roof and decking_some retrofit (1978)	8	8	64
	Administration Builidng	Tilt-up concrete walls with steel framing (1999)	8	9	72
	Field Operations Building	Tilt-up concrete walls with steel framing (1999)	7	9	63
	Paseo Padre Lift Station	CMU Walls With Flexible Roof (1978/1984)	6	5	30
	Generator Building #2	CIP Concrete walls and precast concrete roof (1985)	6	6	36
	Alvarado Pump Station	CIP concrete walls below grade, Precast Concrete walls and roof_some retrofit (1978/1985)	6	8	48
	Aeration Basins	CIP Concrete walls and precast concrete roof (1985)	6	4	24
	Control Building	Metal strap bracing light framed (1978)	6	10	60
	FMC Maintenance Building/Generator Building #1	CMU Walls With Flexible Roof (1978/1984)	6	9	54
	EBDA Pump Station/Reclaim Water Room	CIP concrete walls below grade, Precast Concrete walls and roof_some retrofit (1978/1985)	5	8	40
	Covered Storage	1994 or Later UBC Steel Structure (1999-2011)	5	2	10
	Primary Digester #5	CIP Concrete walls and steel dome (1962/1978/1985)	5	5	25
	Secondary Digester #1	CIP Concrete walls and steel dome_tall walls_roof replaced (1978)	5	4	20
	EBDA Surge Tower	CIP Tower (1978)	4	8	32
LIFE SAFE	Thickener #1	Round CIP Structure_Thickeners (1978/1985)	3	5	15
	Lift Station #1	CIP concrete (1985-Current)	3	4	12
	Heat Mix Building #2	CIP concrete (1962)	2	4	8
	Chlorine Contact Tank	CIP concrete (1978)	2	7	14
	Secondary Clarifiers 1-4	Open CIP Structure_Clarifyers (1978/1985)	2	4	8
	Control Box #3	Underground Concrete Walls Precast/CIP	2	4	8
	Alvarado WWTP Force Main Influent Valve Vault	Underground Concrete Walls Precast/CIP	1	8	8
	Main Electrical Distribution Building	CMU Walls Steel framed roof and metal decking/CIP Walls Steel framed roof and metal decking (1990-1993)	1	9	9

Table 2b: Seismic Vulnerability Summary – Sorted by Seismic Vulnerability Rating

UNION SANITARY DISTRICT - SEISMIC VULNERABILITY SUMMARY				
Structure Name	**Structure Type**	**Seismic Performance Rating**	**Importance Rating**	**Seismic Vulnerability Rating**
Administration Builidng	Tilt-up concrete walls with steel framing (1999)	8	9	72
Irvington Pump Station	CIP concrete walls below grade, CMU Walls with steel framed roof and decking_some retrofit (1978)	8	8	64
Field Operations Building	Tilt-up concrete walls with steel framing (1999)	7	9	63
Control Building	Metal strap bracing light framed (1978)	6	10	60
Primary Clarifiers 5-6/Sludge Pump Room #3	CIP concrete walls below grade, Precast Concrete walls and roof (1978/1985)	9	6	54
Primary Clarifiers 1-4/Sludge Pump Room #1	CIP concrete walls below grade, Precast Concrete walls and roof_Primary Clarifiers (1978)	9	6	54
FMC Maintenance Building/Generator Building #1	CMU Walls With Flexible Roof (1978/1984)	6	9	54
Degritter Building	Precast Concrete Walls and Roof (1985)	10	5	50
Alvarado Pump Station	CIP concrete walls below grade, Precast Concrete walls and roof_some retrofit (1978/1985)	6	8	48
EBDA Pump Station/Reclaim Water Room	CIP concrete walls below grade, Precast Concrete walls and roof_some retrofit (1978/1985)	5	8	40
Generator Building #2	CIP Concrete walls and precast concrete roof (1985)	6	6	36
EBDA Surge Tower	CIP Tower (1978)	4	8	32
Paseo Padre Lift Station	CMU Walls With Flexible Roof (1978/1984)	6	5	30
Primary Digester #5	CIP Concrete walls and steel dome (1962/1978/1985)	5	5	25
Aeration Basins	CIP Concrete walls and precast concrete roof (1985)	6	4	24
Secondary Digester #1	CIP Concrete walls and steel dome_tall walls_roof replaced (1978)	5	4	20
Thickener #1	Round CIP Structure_Thickeners (1978/1985)	3	5	15
Chlorine Contact Tank	CIP concrete (1978)	2	7	14
Lift Station #1	CIP concrete (1985-Current)	3	4	12
Covered Storage	1994 or Later UBC Steel Structure (1999-2011)	5	2	10
Main Electrical Distribution Building	CMU Walls Steel framed roof and metal decking/CIP Walls Steel framed roof and metal decking (1990-1993)	1	9	9
Heat Mix Building #2	CIP concrete (1962)	2	4	8
Secondary Clarifiers 1-4	Open CIP Structure_Clarifyers (1978/1985)	2	4	8
Control Box #3	Underground Concrete Walls Precast/CIP	2	4	8
Alvarado WWTP Force Main Influent Valve Vault	Underground Concrete Walls Precast/CIP	1	8	8

3.2 Tier-1 Screening Procedure

We have completed an initial seismic screening for 25 structures at the Union Sanitary District site using the ASCE 41-13 Tier-1 screening procedure. The Tier-1 procedure uses a checklist approach to identify potential seismic deficiencies for different building types. Based on the structural type of a given structure, an engineer uses the appropriate Tier-1 checklist and simplified calculation procedures to identify deficiencies based on structural plans and site observations.

Based on observations of damage to structures in past earthquakes, seismic engineers have noticed particular patterns of damage for different types of structural systems, and the Tier-1 checklist procedure works off the recognition of these patterns of damage. For example, light framed wood structures that are not properly attached to their foundation have been damaged in past earthquakes. Consequently the Tier-1 checklist for light framed wood structures asks specific questions about how the walls are anchored to the foundation. The checklists for each of the 25 structures that were evaluated as a part of this study can be found in the appendix of this report. A summary of potential deficiencies identified by the Tier-1 evaluation for each of the 25 structures is provided in the summary section of each structure.

It is important to note that the Tier-1 checklist procedure attempts to identify *potential* deficiencies. Because the Tier-1 procedure is a screening procedure that uses limited calculations, it tends to err on the conservative side when identifying seismic deficiencies in structures. In other words, the deficiencies reported by a Tier-1 checklist, may or may not actually represent a substantial seismic hazard for the structure. Follow-up Tier-2 and Tier-3 procedures which rely on much more detailed analysis and calculation procedures are used to determine whether or not a deficiency identified in the Tier-1 screening actually represents a significant potential seismic hazard. Consequently, where deficiencies have been identified by the Tier-1 procedure for the 25 structures as part of this study, we have noted where "further analysis/study" is recommended. Where possible, we have commented on what we believe the outcome of the "further analysis" may reveal. Additionally, we have conduced further analytical analyses than required by the Tier-1 procedure where appropriate (for example, preliminary analysis of sloshing forces in the tank structure) to get a better understanding of how a given structure might perform in a seismic event. These analyses and calculations can be found in the Appendix of this report.

In many cases, we do not need to conduct "further analysis" to know that something identified in the Tier-1 analysis represents a significant seismic hazard. In these instances, we have not noted that "further analysis" is needed to confirm the deficiency, and have stressed the seriousness of the deficiency. Likewise, there are many cases where we do not believe that potential deficiencies identified in the Tier-1 procedure actually represent a significant hazard for the 25 structures included in this study. For example, for concrete structures (like many of the 25 structures that were studied as a part of this screening effort), the Tier-1 checklist prompts the evaluator to identify whether or not the "complete secondary frames" independent of the lateral force resisting system are provided. Essentially, this provision is a potential deficiency for concrete bearing wall structures that we do not believe is a significant seismic hazard for the structures included as part of this study. These potential deficiencies are therefore not discussed in each structure's section.

Each of the 25 structures included in this study has been assessed at the "life-safety" performance level per ASCE 41-13. For structures that are normally occupied like the Control Building, meeting "life-safety" performance indicates that there is a very low probability that earthquake damage to the structure will result in the loss of life. For unoccupied structures meeting "life-safety" performance indicates that there is a very low probability that earthquake damage will cause partial or complete collapse of the structure. For an unoccupied tank, for example, "life-safety" performance would indicate that the tank has not been damaged to the point where it "partially collapses" and rapidly loses its contents. It's important to note that "life-safety" performance does not indicate that a given structure will not be damaged to the point where it is immediately usable following an earthquake. Furthermore, structures can be considered to have met "life-safety" performance even though they have been damaged to the point where repair is not feasible and they may need to be demolished following an earthquake.

Note that the scope of this report is limited to the seismic assessment of the *structural systems* of the 25 structures included as part of this study. Evaluation of non-structural equipment and its seismic anchorage is not within the scope of this study. From our various site visits we noticed that some pieces of equipment seemed to be adequately anchored, and other pieces of equipment appeared to lack any seismic anchorage. Consequently it is possible that buildings like the Main Electrical Distribution Building have very limited damage to its structural system following the DBE, but there may be significant damage to the equipment and utilities that this structure houses. Consequently, we recommend that a seismic assessment of important equipment at the site be done as a follow-up to this study.

Note also that the report on each of the 25 structures does not include a specific assessment of the piping that enters and exits the structure. From site visits, we noticed that most of the piping is not fit with flexible couplings. This means that the pipes have the potential to rupture as adjacent structures move in opposite directions (in the case of a pipe running between two structure), or the as the structure moves relative to the surrounding soil (in the case of a pipe going into or exiting the structure at the ground level). From the geotechnical report included as part of this study, there may be liquefaction inducted settlements on the order of a few inches in the DBE (depending on the location of the structure). While this does not typically pose a structural concern from a life-safety perspective, the liquefaction induced settlements may damage piping and this should be studied further on a case by case basis, particularly for pipes critical to plant operations. Furthermore, while most of the 25 structures we assessed are relatively stiff structures that will not displace more than a few inches relative to the base of the structure, further analysis on a case by case basis is recommended.

3.3 Individual Structure Assessment and Possible Seismic Remediation

The Tier-1 structural deficiencies and seismic performance rating for each of the 25 archetypal structures are summarized in this section. A seismic performance rating of 10 indicates that the structure has a very low probability of meeting "life-safety" performance in the DBE. A seismic performance rating of 1 indicates that the structure has a very high probability of meeting "life-safety" performance in the DBE. Ratings in between 1 and 10 indicate varying levels of probability for "life-safety" performance. Note that these ratings are approximations and have been given based only on Tier-1 analysis procedures and engineering judgment. Consequently the ratings, and the level of damaged expected to each of these structures, can be refined by conducting further analysis.

In addition to the performance rating and deficiencies, possible seismic remediation strategies are proposed for each of the 25 structures. Rough order of magnitude construction costs are given to the anticipated scope of the life-safety upgrade work using the guidance of FEMA 156 – *Typical for Seismic Rehabilitation of Existing Buildings* and our experience. These retrofit strategies and costs should be considered very preliminary, as any structural retrofit project will require further analysis that will refine the scope of the seismic upgrade project. Furthermore, these estimated costs are construction cost estimates in 2016 dollars that do not include soft costs related to service and site disruption, design fees, permitting etc. The costs are for seismic upgrade of the structure and do not include costs for upgrade of the nonstructural components. The costs also assume that a contractor is mobilized to complete multiple projects at one time. If projects are done individually the cost can be expected to increase.

Finally, a brief discussion how the archetypal structure's deficiencies (or lack thereof) may apply to the other buildings in the group is included for each of the 25 archetypal structures.

3.3.1 Degritter Building – Seismic Performance Rating = 10

The Degritter Building is a partially cast-in-place, partially precast concrete structure that was constructed during the 1985 phase of construction at the site. Figure 1 shows an exterior view of the Southwest corner of the Degritter building. The base slab, second floor slab and beam elements, column and pilaster elements, and interior first floor walls are cast-in-place elements. The exterior walls and roof structure are comprised of precast concrete elements. The precast concrete roof elements are not well tied into the walls below and are not well tied to each other. Therefore, they are not expected to perform well in a seismic event and pose a serious collapse hazard. These seismic deficiencies are discussed in greater detail below.

Figure 1: Exterior View of Degritter Building

The main lateral force resisting system of the Degritter Building is its concrete walls. The first floor concrete wall locations are highlighted in Figure 2.

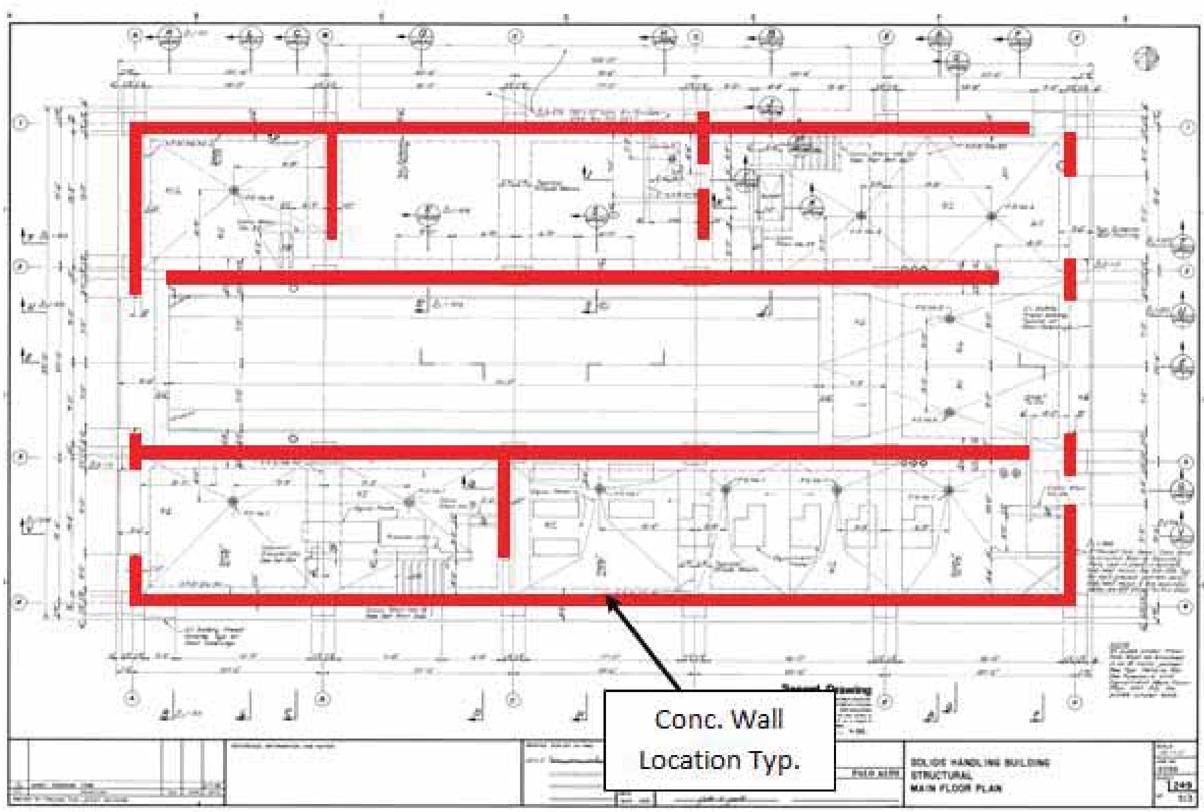

Figure 2: Degritter Building First Floor Concrete Wall Locations

3.3.1.1 Tier-1 Deficiencies

<u>Undefined Load Path/Transfer to Shear Walls/Diaphragm Continuity/No Topping Slab at the Precast Concrete Roof:</u>

There is no positive attachment of roof diaphragm to North and South wall panels. At the East and West wall panels, the concrete fin walls that infill the gaps between double-t precast roof beams are not positively attached to the roof diaphragm. The roof double-t beams are only positively connected to the precast wall panels at one end of the structure, and these elements are likely to break out from the concrete panels at a relatively low force level.

Based on our preliminary analysis the diaphragm ties that connect precast beam elements together are overstressed in the DBE and could rupture. Based on the condition of the roof ties at Primary Clarifiers 1-4, it is possible that the roof ties at the Degritter are corroded, making them even less effective at transferring seismic forces. Furthermore there is virtually no diaphragm in the center of the structure and therefore no way to transfer the mass of the elements that frame the roof openings at the center of the structure out to the concrete walls. Notice the sheet metal enclosure that covers a typical large roof opening in Figure 3 below.

In a seismic event these deficiencies indicate that there is the potential for the roof elements to break apart and fall through the roof or off the side of the structure, posing a very serious hazard. Further analysis could be conducted to better understand how the roof structure might behave in a seismic event, but it will almost certainly be found deficient.

Undefined Load Path at the New Cast-In-Place Concrete Walls on the Second Floor:

New cast-in-place concrete walls at the second floor near the Degritter equipment are tied into the cast-in-place beam elements above (noted "diaphragms" in the drawings). These beam elements were originally designed and intended to take gravity loads associated with framing the large openings in the roof. Note that these "diaphragm" elements were designed by the contractor so it is not completely known how they are tied into the precast beam elements surrounding them. However, these beams are very likely deficient for lateral forces and eccentricity placed on them by the new concrete wall elements because the beam elements would have to resolve the eccentricity into the very thin diaphragm of the precast double-t beams. The new concrete walls and their attachment to the "diaphragm" beams are shown in Figure 3.

Figure 3: New Concrete Partition Wall in Degritter Building

Adjacent Structures:

The odor scrubber unit adjacent to the Degritter Building seems to be braced independently from the Degritter, but is close enough that pounding is possible. Further study is recommended.

Shear Stress Check:

The maximum wall stress is ~100 psi in the East-West direction at the first floor, which meets the ASCE 41-13 Tier 1 check requirements but is close to the limit. Note that wall panel bars are weld connected at the base, which creates a brittle connection. Checking this connection as a force controlled action, the Demand Capacity Ratio (DCR) is approximately 200%. However, there are also cast-in-place concrete walls at this level that are more ductile than the precast wall panel connections. Additionally, the concrete beams and very large column elements will participate to resist lateral loads through frame action. Therefore, there appears to be enough lateral force resisting elements are provided at a life-safety level, but further analysis is recommended given the 3 types of lateral resisting elements in
this critical direction.

3.3.1.2 Remediation Strategies

With additional analysis it is probable that the current amount of shearwall in the Degritter Building can be shown to be sufficient without any additional strengthening. However, it is unlikely that advanced analysis and study will be able to justify the Degritter roof for adequate seismic performance in its current condition. In addition to attaching the roof to the walls for shear transfer, the roof itself will likely need substantial strengthening. We can explore possible strengthening strategies like adding FRP (Fiber Reinforced Polymer) to the roof, but these strengthening strategies would likely also involve adding additional bracing members at the cast-in-place roof beam members ("diaphragms" per the drawings) and the added concrete partition wall. Consequently, if it is even possible to employ a roof strengthening scheme, it will likely be very complex and expensive. Without substantial further analysis, we therefore recommend completely replacing the roof with a new structural steel roof and a new steel deck. With a complete roof replacement, it will also be easier to properly brace the new concrete partition to the roof. We estimate the cost of the seismic retrofit work to be on the order of $250 per square foot.

3.3.1.3 Other Structures in Group

The Degritter building is the archetypal building for precast concrete roof and wall non-tank structures that have not been retrofitted. Other structures in this class include the East Aeration Blower Room, Odor Control Building, and Heat Mix Building #3.

From a brief review of the structural drawings, the construction of the East Aeration Blower Room and Heat Mix Building #3 seem to be very similar to the Degritter Building. These two structures were both part of the 1985 phase of construction (all designed by Kennedy/Jenks Engineers) at the site and very likely have similar deficiencies to the Degritter Building. One notable exception is that both structures appear do not appear to have the same extent of openings in their roofs. Therefore, it may be possible to retrofit these structures without completely replacing their roofs (i.e. similar roof related deficiencies to the Primary Clarifiers 5-6).

The Odor Control Building was also constructed in the mid 1980's but the design appears to have been conducted by another firm. From a brief review of the drawings, it appears that some positive connection between the roof and the concrete walls has been provided. Consequently, the deficiencies with the Odor Control Building may not be as extensive as the Degritter Building and a specific assessment of this structure is recommended.

3.3.2 Primary Clarifiers 5-6 – Seismic Performance Rating = 9

The Primary Clarifiers 5-6 and associated pump room #3 are partially cast-in-place, partially precast concrete structures that were constructed during the 1985 phase of construction. Figure 4 shows an exterior view of the pump room on the west side of Clarifiers 5-6, and includes a view of the odor scrubber towers that are supported off the clarifier roof. The construction of Primary Clarifiers 5-6 is very similar to the Degritter Building: the base slab, pilaster elements, and tank walls are cast-in-place elements. The exterior walls above grade and roof structure are comprised of precast concrete elements. The precast concrete roof elements are not well tied into the walls below and are not well tied to each other. Therefore, they are not expected to perform well in a seismic event and pose a potential collapse hazard. These seismic deficiencies are discussed in greater detail below.

Figure 4: Exterior View of Primary Clarifiers 5-6/Pump Room #3

The primary system that resists inertial and incremental fluid loads of the clarifiers and pump room are the concrete walls. The precast concrete wall panels directly under the precast roof are highlighted in Figure 5.

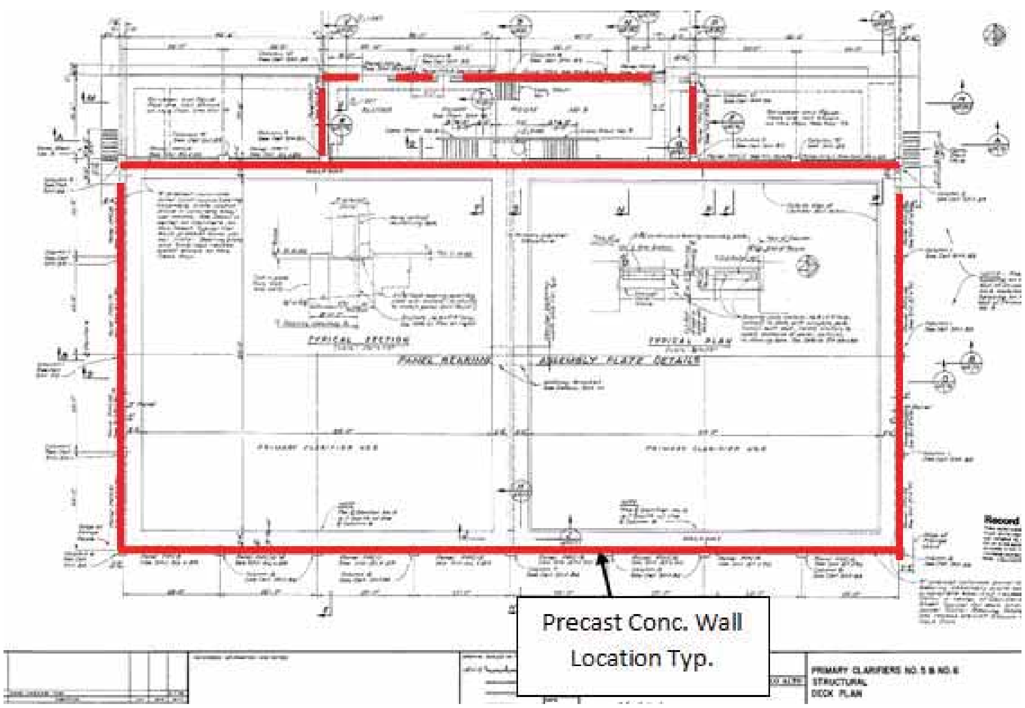

Figure 5: Primary Clarifiers 5-6 Precast Concrete Wall Locations

3.3.2.1 Tier-1 Deficiencies

Undefined Load Path/Transfer to Shear Walls/Diaphragm Continuity/No Topping Slab at the Precast Concrete Roof:

There is no positive attachment of roof diaphragm to North and South wall panels. At the East and West wall panels, the concrete fin walls that infill the gaps between double-t precast roof beams are not positively attached to the roof diaphragm (and the East fin walls have louver openings at every other fin creating further discontinuity). The roof double-t beams are only positively connected to the precast wall panels at the East end of the structure, and these elements are likely to break out from the concrete panels at a relatively low force level. During construction, the pilasters on the East side were strengthened with respect to the typical pilasters. This may have been done so that the pilasters could take the seismic load from the precast roof beams. Unfortunately, as seen in our 3/28/14 site visit, the pilaster strengthening stops at the soil level and does not extend to the foundation. Therefore, even if the precast double-t roof beam anchorage did not break out from the panels and even if the pilasters strengthening were sufficient to resist these inertial forces, the strengthening of this potential lateral load transfer path would be largely ineffective.

The diaphragm ties that connect precast beam elements together will be overstressed in the DBE and have the potential to rupture. Furthermore, based on the condition of the roof ties at Primary Clarifiers 1-4, it is possible that the roof ties at Primary Clarifiers 5-6 are corroded, making them even less effective at transferring seismic forces.

In a significant seismic event these deficiencies indicate that there is the potential for the roof elements to break apart and fall through the roof or off the side of the structure, posing a serious hazard.

Positive Connection at Girder/Column:

The precast concrete beams are not positively attached to the pilasters at the shared wall between the clarifiers and the pump room. Unseating of the precast concrete beams is possible and further analysis is recommended.

Adjacent Structures:

The large tower structure adjacent to Primary Clarifiers seems to be braced to the roof of the clarifiers. Given that the clarifier's roof attachment is deficient, this connection will place additional inertial demands on the roof diaphragm and increases the likelihood that the roof diaphragm on the clarifiers will fail.

Furthermore, the center stirring mechanisms and non-structural access walkways are not within the scope of this study, and further analysis may be warranted.

Out-Of-Plane Wall Anchorage:

The second floor precast walls are sufficient to span horizontally and are not reliant on being anchored to the diaphragm.

However, the typical pilasters are slightly overstressed to cantilever the weight from the precast walls down to the foundation. With further study of the actual load transfer (3 sided attachment), the small possible benefit of using the walkway and channel elements as diaphragms, and with the possible significant benefit geotechnical input on the passive soil pressures, it is likely that the wall/pilaster system can be shown to be sufficient for the out-of-plane forces from the precast walls (concurrent with other inertial and fluid forces). Note that the preliminary check was conducted at the base of the pilasters, where some longitudinal reinforcing has been discontinued. With the presence of soil pressures it may be possible to show the point of maximum moment is near the top of the soil where in many cases no reinforcing is discontinued.

Deflection Compatibility:

Intermediate pilaster #1 @ North/South side was checked as a typical column and may develop a flexural hinge between roof and second floor based on the column going into double curvature. This is not likely a deficiency with advanced analysis because at corners of building, the structure should be stiff enough to protect the columns and at the middle of the building the second floor diaphragms are not likely stiff enough to enforce double curvature in the pilasters.

The walkways at the perimeter of the structure may try to act as diaphragms and help buttress the wall. Our preliminary analysis indicates that some ductility demand may be required in these diaphragm elements and further analysis seems warranted.

The roof diaphragm may experience damage at re-entrant corners from the sludge pump room. These re-entrant corners are small relative to the overall footprint of the digesters and wouldn't be a significant concern for a well detailed building with a true "rigid" diaphragm. However, these sludge pump room end walls (even though they are short) are stiff compared to the diaphragm, and may try to take significant amount of load relative to their length. Without introducing a collector, this could lead to local diaphragm damage. Further analysis is recommended.

3.3.2.2 Remediation Strategies

The most obvious deficiencies that need to be addressed for Primary Clarifiers 5-6 are the lack of shear transfer at the walls, the discontinuities in the roof diaphragm, and the adjacent tower anchorage. The shear transfer issue can be addressed either with the installation of plates (similar in scope to the 1991 retrofit of Primary Clarifiers 1-4) or the installation of FRP angles at the roof/wall interface. The diaphragm discontinuity issue can be remediated through a number of strategies including adding FRP strips at the double-t joints. This strategy would add continuity to the roof diaphragm without adding mass. Finally, the adjacent tower should be cut-free from the roof for lateral load transfer. This can be done for a low-cost from the Primary Clarifier's perspective, but it will very likely involve supplementing the bracing system of the tower.

Addressing these obvious issues will greatly enhance the probability that the Primary Clarifiers 5-6 survive a major earthquake without collapsing. However, to be considered "life-safe" in the DBE, additional work may have to be done. The final scope will be largely dependent on the findings from further analysis regarding the adequacy of the existing column/wall system. Preliminary modeling of this system indicates that the existing column/wall system has sufficient strength to withstand the earthquake induced pressures and inertial forces with some limited ductility. However, considering incremental fluid pressures and inertial pressure, the displacement at the top of the columns may be on the order of 3 inches. Consequently if the walls are not tied into the roof for out-of-plane load transfer as part of the retrofit scheme, it is possible that the roof diaphragm will be damaged at this location. On the other hand, if the walls are tied into the diaphragm for out-of-plane load transfer, the roof diaphragm may be overstressed to transfer these out-of-plane forces even if the discontinuities at the double-t joints are addressed. Therefore, to avoid completely replacing the roof, an exterior wall buttressing scheme may need to be explored.

Assuming that the final retrofit scope is limited to fixing the diaphragm issues and independently bracing the adjacent tower (i.e. no buttressing required), we estimate that the retrofit work will be on the order of $200 per square foot.

3.3.2.3 Other Structures in Group

Primary Clarifiers 5-6/Sludge Pump Room #3 is the archetypal building for tank structures with concrete walls below grade and precast concrete walls and roofs above grade that have not been retrofitted. Other structures in this class include Sludge Pump Rooms #2 and #4. These structures have similar roofs to Primary Clarifiers 5-6 and are thus very likely to have the same roof related deficiencies.

Sludge Pump Room #2 was constructed during the 1978 phase of construction. Therefore, they are actually more structurally similar to Primary Clarifiers 1-4 than Primary Clarifiers 5-6. From a brief review of the drawings Sludge Pump Room #2 does not have a continuous diaphragm at its intermediate (ground) level so it may be subject to the same deflection/wall related deficiencies as Primary Clarifiers 1-4 (with the added deficiency of not having any structural retrofit).

3.3.3 Primary Clarifiers 1-4 – Seismic Performance Rating = 9

The Primary Clarifiers 1-4 and the associated pump building are partially cast-in-place, partially precast concrete structures that were constructed during the 1978 phase of construction. The base slab, pilaster elements and tank walls are cast-in-place elements. The exterior walls above grade and roof structure are comprised of precast concrete elements. In the original 1978 construction, the precast concrete roof elements were not well tied into the walls below and were not well tied to each other. The clarifiers were seismically retrofitted in 1991 by Carollo Engineers to address some of the issues with the original 1978 construction. The plates added as part of the retrofit are shown in Figure 6. While these retrofits will definitely improve the performance of the clarifiers in a significant earthquake, the Primary Clarifiers 1-4 are still deficient for the DBE in many respects. These seismic deficiencies are discussed in greater detail below.

Figure 6: Portion of 1991 Structural Retrofit to Primary Clarifiers 1-4

The primary system that resists inertial and incremental fluid loads of the clarifiers and pump room are the concrete walls. The precast concrete wall panels directly under the precast roof are highlighted in Figure 7.

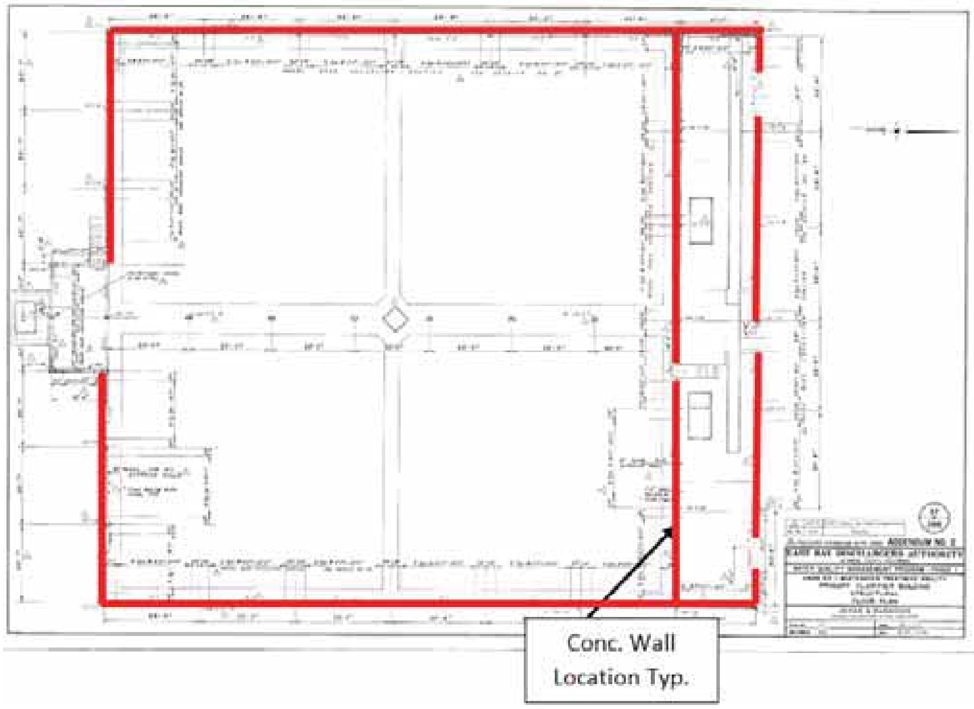

Figure 7: Primary Clarifiers 1-4 Precast Concrete Wall Locations

3.3.3.1 Tier-1 Deficiencies

No Topping Slab at the Precast Concrete Roof:

Diaphragm ties might be sufficient to take the mass of the roof only, but further analysis is recommended. Based on the condition of the ties, however, the diaphragm is very likely deficient to transfer its own weight to the walls. Carollo's 1991 "Evaluation of Existing Roof Diaphragm Connections" Report indicated extensive corrosion in the ties. The 1991 retrofit work fixed the ties over the pump room and along the center of the structure, but did not explore or repair the ties in general. Even if all the ties were replaced in 1991, the ties are very likely to have been corroded in the time since the retrofit (a similar amount of time has elapsed between the original construction and the 1991 report/retrofit). From our 3/28/14 site visit, we found ties completely exposed and completely corroded. Further exploration of the condition of the ties is necessary.

Furthermore, the adequacy of the diaphragm elements that were added in the 1991 retrofit (e.g. chord straps) needs further analysis.

Transfer to Shear Walls:

The 1991 retrofit positively attached the roof diaphragm to the precast concrete walls, but the connections appear to be deficient for the demands. Further analysis is recommended but the retrofit plates installed for shear transfer are very likely to be found deficient for current code criteria.

Note that when the control box was retrofitted in the early 1990's the drawings do not indicate that the new cast-in-place walls were positively connected to the precast concrete roof along the length of the control box. If no positive connection exists in this location, the diaphragm connections that do exist along this wall will be even more overstressed.

Out-Of-Plane Wall Anchorage:

The roof level anchorage that was added as part of the 1991 retrofit is very flexible for resisting out-of-plane loads, and is therefore not expected to function as out-of-plane anchorage. We assume this was done in an effort to limit the load that has to be distributed in the diaphragm in a significant earthquake. Consequently, the precast concrete panels will have to span horizontally to the concrete pilasters for out-of-plane support.

Some precast panels are sufficient to span horizontally to columns while others are insufficient. Note that with further analysis of a 3-sided support condition, it is likely that even in the longer panels the wall anchorage can be found sufficient. If the entire infill panel weight is taken by the columns, the un-retrofitted columns are sufficient to get the load down to the top of the walls based on our preliminary analysis. However, further analysis is recommended, as the columns are close to being overstressed.

If the cantilever wall truly spans to the columns, then the walls will be locally overstressed. Without supplementing the out-of-plane anchorage at the roof, the cantilever wall may be overstressed considering concurrent fluid and inertial forces even if the weight of the panels is uniformly distributed across the width of the panel (which would be the best-case scenario for the wall).

For outward loads, the input of a geotechnical engineer for passive pressures may significantly reduce the demands on the cantilever walls. For inward loads, the circular infill concrete may increase the capacity of the wall if the cantilever wall can span horizontally (these circular infills of concrete do not appear positively attached to the walls, but are positively attached to the foundation and could help buttress the walls). Additionally, modeling the walkway and channel diaphragms may have a minor positive effect on the capacity of the walls.

Note that one of the recommendations in the 1991 “Seismic Evaluation of Primary Clarifiers 1-4” report is to strengthen the pilasters: “During an earthquake, these cantilever columns prevent the wall panels from moving away from the roof. Since the ties in these columns are relatively light, excessive deflections due to the seismic loads could cause brittle failure of the columns.” However, the 1991 retrofit of Primary Clarifiers 1-4 does not appear to have included strengthening the columns on the East, West and South sides of the clarifiers. Many of the exterior columns that were strengthened on the North side of the clarifiers only extend to the top of pipes that run along the walls of the buildings.

If the panels do primarily span horizontally, the deflection of the columns may be on the order of 5-10 inches. While our analysis indicates that there should be sufficient shear strength in the columns to avoid a brittle failure, this level of deflection is undesirable and further analysis is recommended.

Shear Stress Check:

Max stress in the precast concrete walls is ~40 psi which meets the Tier-1 check. Note that wall panel bars are weld connected at the base at discrete locations, which creates a brittle connection. This connection has been supplemented with angle bracket connections. These combined connections are slightly overstressed as a force controlled actions for the weight of the roof only. However, the force to these connections will be limited by yielding in the diaphragm etc. Therefore, with advanced analysis (and depending on the extent of retrofit and whether or not the out-of-plane anchorage of the precast panels at the roof is supplemented) these connections may be sufficient.

Unbraced Mezzanine:

The mezzanine in the pump room is not adequately braced. Further analysis of mezzanine required.

Adjacent Structure:

The clarifiers are adjacent to the expanded and re-built control box #1. The control box is partially integral and partially separated from the clarifiers. Given the potential movement of the clarifier walls, this should be studied further.

The center stirring mechanisms and non-structural access walkways are beyond the scope of this study, and further analysis may be warranted.

3.3.3.2 Remediation Strategies

As with Primary Clarifiers 5-6, the ultimate scope of retrofit work at Primary Clarifiers 1-4 will depend on the results of further analysis. At this time, it seems likely that deficient shear transfer at the roof, the corroded connections between double-t beams, the strength and flexibility of the existing wall system, and the unbraced mezzanine are all life-safety deficiencies that should be remediated. For the first two items and FRP solution similar to Primary Clarifiers 5-6 can be explored. To address the possible deficiency of the wall strength and flexibility, it is likely that an exterior buttressing scheme will need to be developed. This scheme could either be a pilaster strengthening solution (similar to what was originally proposed in Carollo's 1991 report) or a flying buttress solution. The former solution would likely require the temporary removal of the pipes that run around the perimeter of the clarifiers, as well as substantial earthwork to allow for the installation of the pilasters to be continuous to the foundation. Consequently, it appears that a flying-buttress solution would be more cost effective. We estimate that the retrofit work will be on the order of $250 per square foot.

3.3.3.3 Other Structures in Group

Primary Clarifiers 1-4/Sludge Pump Room #1 are the only structures assigned to their group. Consequently, no extrapolation is required.

3.3.4 Irvington Pump Station – Seismic Performance Rating = 8

The Irvington Pump Station is a partially cast-in-place, partially reinforced masonry structure that was constructed during the 1978 phase of construction. It is located to the South of the main Union Sanitary Site. The pump station is a Three-story structure, with the first two floors below grade. An exterior view of the pump station is shown in figure 8. The base slab, second and third floor slabs and beam elements, and below grade walls are cast-in-place elements. The exterior walls above grade are reinforced masonry. Originally, the roof of the structure was comprised of precast double-t beams. However, this roof was replaced by a steel roof in 2001 as part of an expansion and retrofit project. The seismic detailing of the new roof structure has deficiencies which may cause the roof to collapse in the DBE. The seismic deficiencies are discussed in greater detail below.

Figure 8: Exterior View of Irvington Pump Station

The main lateral force resisting system of the Irvington Pump Station is its concrete walls below grade, and its masonry walls above grade. The masonry walls above grade are highlighted in Figure 9. Note that the interior masonry wall was added as part of the 2001 retrofit and expansion project. The line work on the right hand portion of the plan are the below grade concrete beams and walls that form the new below grade structure that was added during the 2001 retrofit and expansion project.

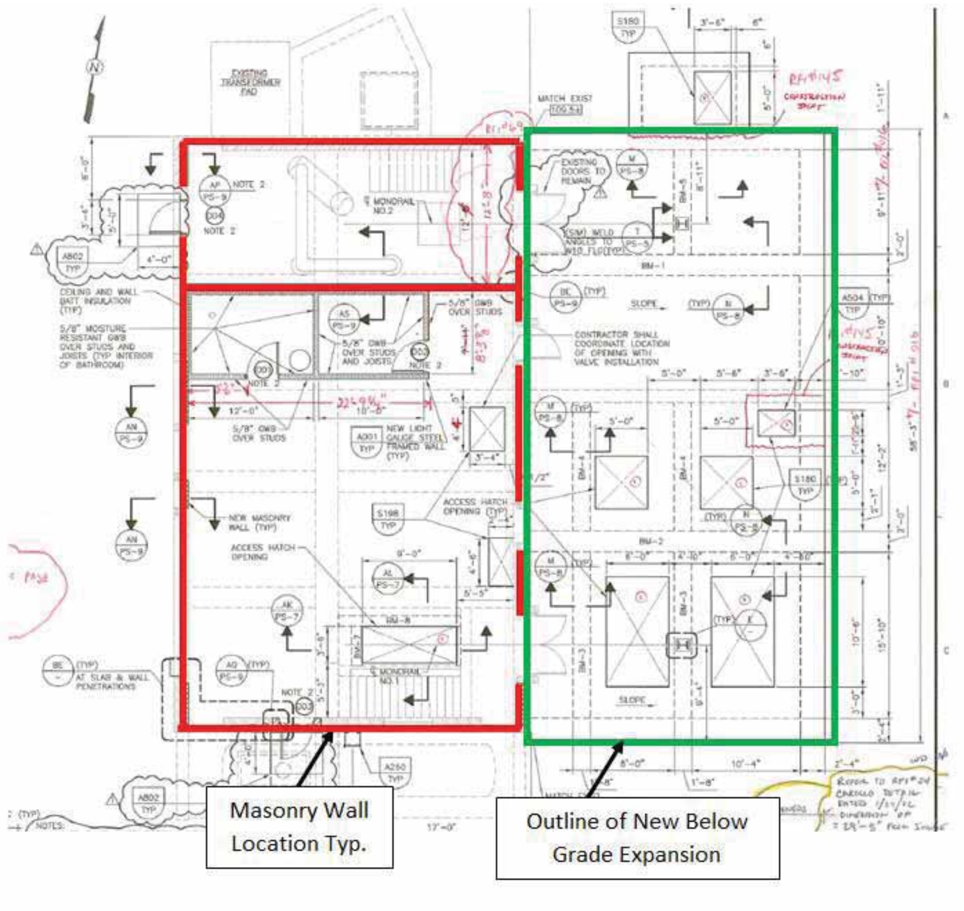

Figure 9: Irvington Pump Station Masonry Wall Locations and Expansion Extent

3.3.4.1 Tier-1 Deficiencies

Wall Anchorage:

The out-of-plane anchorage of the reinforced masonry roof to the steel diaphragm relies on bending of single angle connections and prying on a very torsionally weak weld. This load path is deficient with a demand-to-capacity ratio (DCR) of ~400% not including any gravity load on the weld. Consequently, there is a substantial possibility that the weld will fracture in the DBE which will cause the beam to drop and may partially collapse the roof. This deficiency is the primary reason for the poor seismic rating of the Irvington Pump Station, and something that is relatively easy and inexpensive to remediate (see section below).

Cross Ties:

Cross ties in the long (North-South) direction rely on plate bending and prying on bolted connections. These connections are very likely deficient with further analysis.

Openings at Shear Walls:

There are a series of openings in the second floor diaphragm. Given the inertial loads in the diaphragm, this shouldn't be a life-safety issue for seismic load transfer with additional analysis. However, the cast-in-place slab diaphragms tie the walls of the structure together to resist incremental soil pressures during a seismic event and the slab may be overstressed to serve this purpose. Further analysis is recommended. Note that the openings do not appear as extensive as the openings in the Alvarado Pump Station. Therefore, if further analysis reveals that the Alvarado site is sufficient, than the Irvington site should be sufficient assuming the soils are similar.

Adjacent Structures:

The new below grade building that was constructed as part of the 2001 project is connected to the original pump station building. The new walls are dowelled to the existing walls so this is very likely not a deficiency with further analysis.

Vertical Irregularities:

The middle East-West running wall is offset at the second floor. The wall runs the entire width of the structure so this is very likely not a deficiency for the gravity framing below with further analysis. The diaphragms should be sufficient to distribute the transfer forces, but this should be confirmed with additional analysis.

Mezzanines:

Small office structures on upper floor do not appeared to be tied into the structure and have only gypboard. The small equipment platforms appear to be adequately tied into the structure but further analysis is recommended.

3.3.4.2 Remediation Strategies

With additional analysis to confirm the retrofit scope, it seems likely that the only major deficiency that needs to be addressed is the out-of-plane anchorage and cross-ties at the new steel roof. These deficiencies can be remediated with additional plates connected to the existing steel beam members and perimeter ledger. We expect this retrofit to be simple and relatively inexpensive, and it will drastically improve the probability that the Irvington Pump Station meets life-safety performance in the DBE. We estimate the cost of this retrofit work to be on the order of $50 per square foot.

3.3.4.3 Other Structures in Group

The Irvington Pump Station is the archetypal building for retrofitted structures with concrete walls below grade, masonry walls above grade, and light steel roofs. The other structure in this class is the Newark Pump Station and its associated generator building. From a brief review of the drawings, both the original construction and the retrofit of the Newark site appear to be very similar to the Irvington site. Consequently, it is likely that the Newark site has similar seismic deficiencies to the Irvington site.

3.3.5 Administration Building – Seismic Performance Rating = 8

The Administration Building is a two-story structure built in 1999 using the 1994 Uniform Building Code. The structure is very geometrically and structurally complex; many different structural elements and materials are used to support the structure including rolled steel shape beams and columns, open web steel joists, dimensional timber beams, glue-laminated timber beams and precast concrete panels. The complex geometry can be seen from the exterior view of the Administration Building shown in Figure 10.

Figure 10: Exterior View of Administration Building

Concentrically braced frames serve as the structure's primary lateral force resisting system. The brace locations on the first-story are shown in Figure 11. As noted in greater detail below, the concentrically braced frames do not have ductile detailing and are not expected to perform well in the DBE. Furthermore, the tall architectural precast concrete panels are not well tied into the structure and may collapse in the DBE. Given the Administration Building's poorly detailed systems and structural complexity, it is rife with potential seismic deficiencies as discussed in greater detail below.

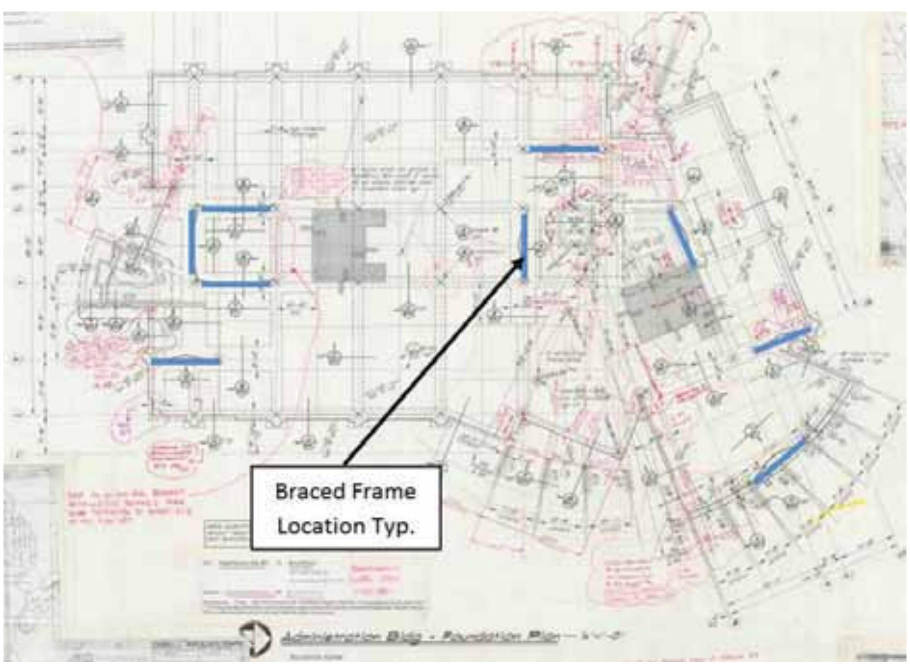

Figure 11: First-Story Braced-Frame Locations in Administration Building

3.3.5.1 Tier-1 Deficiencies

Only One Anchor Provided at Top and Bottom of Precast Panel:

Panels A1 – A4 at the entrance to the structure are not connected to a diaphragm and are only connected at one location at the top of the panel.

For seismic action in the direction of the panels, it is conceivable that the panels can cantilever. In the direction normal to the panels, the eccentric moment has to be taken weak-way through shear tabs, which is almost certainly deficient. At the North end of the panels, the back W16x26 drag is connected mid-height to Panel A5. On the South end of the panels, Panel A1 is connected to the W16x26 into the main building by a series of connections shown as 9/S4.4 (the beams are @ different elevations). This load path is very likely deficient and it is possible that the panels can collapse in the DBE, which is a significant life-safety hazard and may create and egress hazard.

Precast Panel Out-Of-Plane Anchorage:

The typical wall anchorage straps and their development into the diaphragm at roof level is overstressed based on Tier-1 procedures. Furthermore, the typical strap anchors provided have to be cast into the precast panels meaning that there is no field tolerance to install the anchors. In many cases the straps are not installed as intended making them even less effective at bracing the panels. Further field condition assessment is recommended.

Where panels are connected together through steel beams (e.g. Panels A5-A11), the load development into the diaphragm is overstressed per Tier-1 procedures. Further analysis of this load path is recommended, but is likely to be found deficient.

Axial Stress at Braced-Frame Columns:

Tier-1 calculation procedures show that the braced frame columns are overstressed.

For a two-story structure with chevron bracing, this Tier 1 check would typically be very conservative because there is likely little frame action, and the columns at the first-story are only subject to the force from the braces at the second-story. However, some of the braces in the administration building are inverted-V bracing. The size of the column is the same size as the second floor brace. Therefore, at a capacity level, the column is overstressed to develop the tension capacity of the bracing and could buckle in the DBE causing a local collapse in the structure. Further analysis is recommended, but this condition is likely to be found deficient.

Vertical Irregularities:

The brace on gridline 8 is discontinuous at the second floor. At one end of the brace, the load is taken on a transfer beam. At the other end of the brace, the load is taken eccentrically into a precast concrete wall element.

The small brace on line G that braces the back of the curved low roof portion is discontinuous. The small brace is in line with a ledger beam, which is connected to a larger full-height brace at its mid length. Differential deformation along this drag will induce bending at the full-height brace.

The Mansard roof is attached to the roof diaphragm for shear transfer. This load path should be assessed in greater detail, particularly because these points of shear transfer do not typically align with brace locations.

Further analysis is recommended to explore the possible deficiencies associates with all of these conditions.

Beam Strength at Chevron Bracing:

The beam strength at chevron brace locations is insufficient to take the unbalanced load when a brace buckles in compression concurrent with a brace yielding in tension. This is a significant deficiency which could cause a brace beam to fail, leading to possible local collapse.

Brace Connection Strength:

The HSS column wall, the anchor bolt connection at the base of the braces and the welds at the gusset plate are all deficient to develop the yield capacity of the braces. This is a significant deficiency because if any of these aspects of the connection fail, the brace will not be able to resist any lateral seismic forces.

Diaphragm Cross Ties:

No continuous cross ties are provided at the curved low roof precast panel portion of diaphragm. It is doubtful that sub-diaphragms are strong enough for anchorage forces and doubtful that the joists will be able to serve as chords for sub diaphragm. Note that nominal unit shear capacity of low diaphragm is only 640 lb/ft (1/2" w/ 10d @ 6").

Braced Frame Redundancy:

Per Tier-1 checklist procedures, the Administration Building does not have enough bracing locations. This deficiency assesses the possibility of deficient seismic performance as the building's structural system degrades during the course of an earthquake. Typically this is not a significant life-safety concern. However, for the Administration Building which has many transfers and potential irregularities as the building degrades, the lack of redundancy could pose a significant life-safety concern. Further analysis is recommended.

Openings at Frame Locations:

The stair and atrium openings are adjacent to frames. Typically this is not a significant life-safety concern, but because of the irregularities and transfer requirements of the diaphragms in the Administration Building, this potential deficiency should be studied further.

Brace Compact Members:

The sections used for brace members do not meet the Tier-1 analysis compactness requirements.

3.3.5.2 Remediation Strategies

The remediation strategy for the Administration Building is very likely to center on the deficiencies associated with the precast tilt-up panels and the braced frames. Supplemental anchoring and additional framing members are likely needed at the precast panels. At the braced frames, it will likely be necessary to at least supplement the weld-to-gusset and gusset-to-column connections, strengthen the beams at the Chevron brace locations, and strengthen the columns at inverted Chevron locations. Instead of strengthening the connections of the existing Chevron braces, another possibility would be to replace the tube braces with more ductile and weaker buckling-restrained braces. This solution would employ a different type of structural system that would be strong enough to resist seismic forces, but weak enough to protect the surrounding members and connections. Further strengthening, including the possible introduction of additional bracing members, is pending the results of additional analysis. Unlike at some of the other structures at the site, the retrofit work to the Administration Building will very likely involve the removal and replacement of architectural finishes which will add to the cost of the work. We estimate the cost of this retrofit work to be on the order of $150 per square foot.

3.3.5.3 Other Structures in Group

The Administration Building is the archetypal building for steel framed structures with tilt-up concrete wall panels. The other structure in this class is the Field Operations Building. From a brief review of the drawings, the Field Operations Building is slightly different from the Administration Building in that it does not have steel braced frames. Instead, the lateral system for the high-bay portion of the structure is concrete tilt-up wall panels and the lateral system for the low-bay portion of the structure is plywood shear walls. Based on the detailing of the

Administration Building, we would expect to find deficiencies in the Field Operations Building associated with the tilt-up concrete wall panels and possibly at the intersection of the high-bay and low-bay structures. Further study is recommended to better define the possible remediation scope of the Field Operations Building.

3.3.6 Field Operations Building – Seismic Performance Rating = 7

The Field Operations Building is comprised of two structurally separate, one-story buildings that were constructed in 1999 using the 1994 Uniform Building Code. One structure is a low-bay, wood framed structure with a plywood shearwall lateral system. The other structure is a high-bay wood framed structure with a precast concrete tilt-up wall lateral system. An exterior view showing both parts of the structure can be seen in Figure 12.

Figure 12: Exterior View of Field Operations Building

The locations of the plywood shear walls and precast concrete tilt-up walls are shown in Figure 13. As noted in greater detail below, the main deficiency of the Field Operations Building is the deficient connections between the precast concrete walls and the plywood diaphragms. These limited connections are not expected to perform well in the DBE. Furthermore, the structural separation between the two structures appears to be inadequate which could lead to pounding between the two structures. These deficiencies are discussed in greater detail below

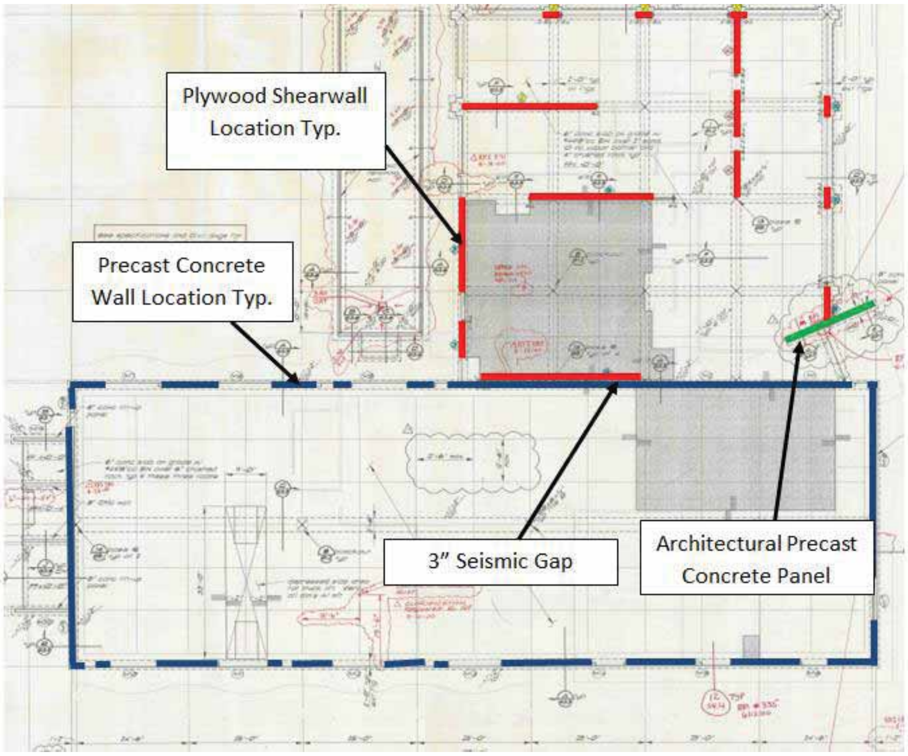

Figure 13: Wall Locations in Field Operations Building

3.3.6.1 Tier-1 Deficiencies

Precast Panel Out-Of-Plane Anchorage:

The typical embedded anchorage straps are overstressed (250% DCR), indicating that they could pull out and be ineffective in the DBE. The sub-diaphragm anchorage development in North-South direction is overstressed (170% DCR), which could lead to damage in the diaphragm.

The architectural concrete panels require further study, but are very likely deficiently anchored. The panels at the west bumpout structure do not appear anchored to the tilt-up walls at the high bay which could lead to pounding damage. The architectural panel at the building entrance to the low-bay portion of the structure is connected directly to a shear wall at one location, but it doesn't seem like the diaphragm shape and capacity will allow the other anchors to be very effective. This could lead to torsional instability and potential collapse of the panel. Further study is recommended.

Adjacent Buildings:

The clear distance between low-bay and high-bay portions of building is only 3 inches (~2%). When diaphragm and wall deflections are considered, the actual deflections could be considerably higher. This could lead to pounding between the two structures, with the diaphragm of the low-bay portion of the structure pounding mid-height on the precast concrete wall of the high-bay portion of the structure.

Shear Stress Check of Plywood Shear Walls:

The plywood shearwalls are overstressed at the Tier 1 quickcheck procedure (DCR ~180% worst case direction). This may not be a life safety issue with further analysis, as preliminary checks indicate the plywood walls are very near their capacity.

The drag connections at the glulams appear overstressed to transfer their tributary mass, so further analysis of the collectors and alternate load paths should be explored.

Diaphragm Continuity:

The mansard roof creates a stepped diaphragm at the roof. The back side has plywood so the load transfer in this direction appears sufficient. In the direction normal to the roof edge, the load has to transfer weak-way through the edge glulam beams. This should be studied further

Unblocked Diaphragms at Low-Bay Structure:

The maximum diaphragm span is ~60'. This should be investigated further but is not likely a life-safety concern provided that the drag connections at the glulam beams are found to be sufficient.

Mezzanines:

The mezzanine in the warehouse portion of the structure is structurally independent of the precast concrete walls. This structure appears to be a pre-engineered moment frame type of structure. Verification of the adequacy of this structure is recommended.

3.3.6.2 Remediation Strategies

The remediation strategy for the Field Operation Building is very likely to center on the deficiencies associated with the precast tilt-up panels. Supplemental anchoring and additional framing members are likely needed at the precast panels. Further work is pending additional analysis, but it is likely that something will need to be done to address the lack of separation between the two structures. This will require substantial work and supplemental framing at the interface between the two structures. Alternatively, a braced frame could be added to the center of the high-bay portion of the structure to limit deflections and reduce the change of pounding between the structures.

Strengthening work to the low-bay portion of the building regarding the diaphragms and shearwalls will very likely be minor, but will involve the removal and replacement of architectural finishes. While the ultimate scope of the retrofit work is pending additional analysis, we estimate the cost of this retrofit work to be on the order of $100 per square foot.

3.3.6.3 Other Structures in Group

The Field Operations Building is in the structure group for buildings with tilt-up concrete wall panels. The other structure in this class is the Administration Building which was assessed as part of the original draft report.

3.3.7 Paseo Padre Lift Station – Seismic Performance Rating = 6

The Paseo Padre Lift Station is comprised of two structures: a cast-in-place embedded structure below grade, and a separate small reinforced masonry structure above grade. The lift station was constructed during the 1985 phase of construction and is located to the South of the main Union Sanitary Site. An exterior view of the pump station is shown in Figure 14.

Figure 14: Exterior View of Paseo Padre Lift Station

The main lateral force resisting system of the Paseo Padre Lift Station is concrete walls below grade, and masonry walls above grade. The masonry walls above grade and concrete walls below grade are highlighted in Figure 15. The main deficiency with the lift station is the lack of out-of-plane anchorage of the masonry walls, which is discussed in greater detail below.

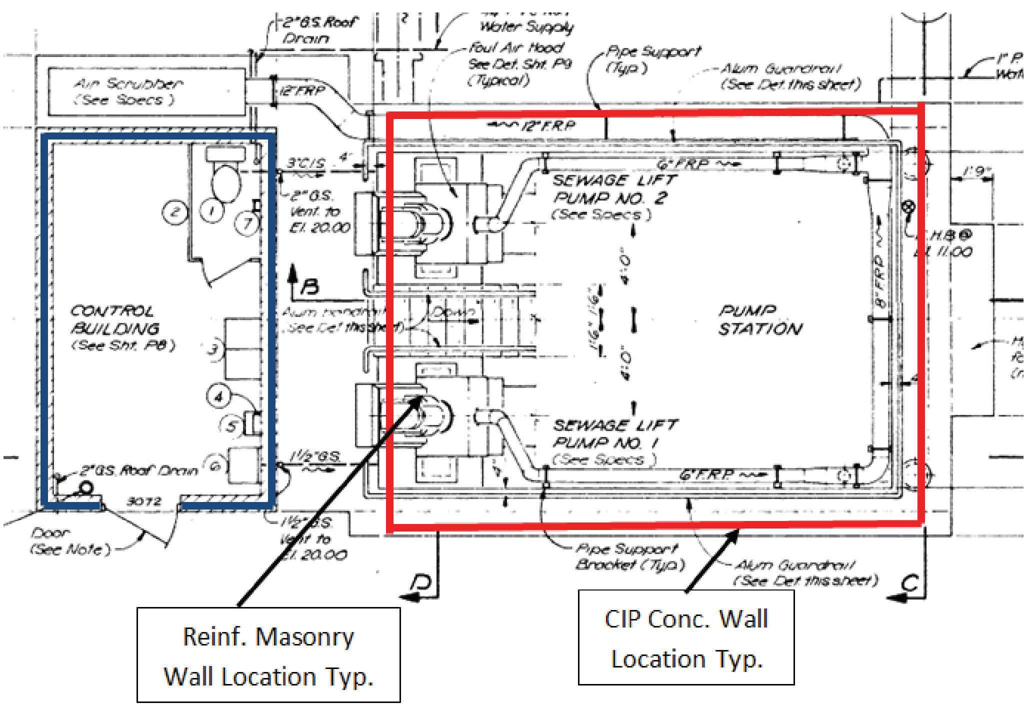

Figure 15: Paseo Padre Lift Station Wall Locations

3.3.7.1 Tier-1 Deficiencies

Wall Anchorage/Wood Ledgers:

The out-of-plane anchorage of the reinforced masonry wall induces cross-grain tension in the ledger plate. Also this out-of-plane action can potentially roll rim joist. Note that the reinforced masonry walls can span horizontally to the cross walls but the lack of anchorage is still a deficiency from a deflection compatibility issue that could cause the roof framing to collapse.

Cross Ties:

There are no cross ties in blocking direction. The joists can serve as ties in joist direction.

Openings at Shear Walls:

The embedded lift station is a completely open structure except for the landing near the base of the stairs (which itself is open). The minimal diaphragm continuity at the base is very likely not a lift-safety issue with further analysis. Also, the walls did not appear to under structural distress due to soil pressures at the time of site visit so they are likely sufficient for additional seismic induced soil pressures. However, this should be studied further to confirm.

3.3.7.2 Remediation Strategies

With additional analysis to confirm the retrofit scope, it seems likely that the only major deficiency that needs to be addressed is the out-of-plane anchorage and cross-ties at the small roof. These deficiencies can be remediated with the addition of holdowns to the roof framing. We expect this retrofit to be simple and relatively inexpensive. We estimate the cost of this retrofit work to be on the order of $50,000, with most of the cost relating to mobilizing a contractor to the site. Consequently, because we expect this to be a simple retrofit, we recommend completing the seismic retrofit concurrently with other work that needs to be done at the site.

3.3.7.3 Other Structures in Group

The Paseo Padre Lift Station is in the structure group for reinforced masonry structures with flexible roofs built between 1978 and 1984. The other structures in this class are the Fremont Lift Station and FMC Maintenance Building. The FMC Maintenance building was assessed as part of the original Draft Report. The drawings of the Fremont Lift Station appear incomplete, so it should be confirmed that the construction is similar to the Paseo Padre Lift Station.

3.3.8 Generator Building #2 – Seismic Performance Rating = 6

Generator Building #2 is a small one-story structure with cast-in-place walls and a precast concrete roof. This structure was constructed in 1988 and is shown in Figure 16.

Figure 16: Exterior View of Generator Building #2

The precast concrete roof is covered with a reinforced concrete topping slab which is tied down to the East and West concrete walls with dowels. The North and South concrete walls are tied to the roof beam stems with steel embed plates, and the center concrete wall is tied to the concrete roof beam stems with small steel angles. These concrete walls resist seismic induced forces and the wall locations are shown in Figure 17. Based on the preliminary screening, the main seismic deficiency of Generator Building #2 is related to overstressed connections between the diaphragm and the cast-in-place concrete walls, as discussed in further detail below.

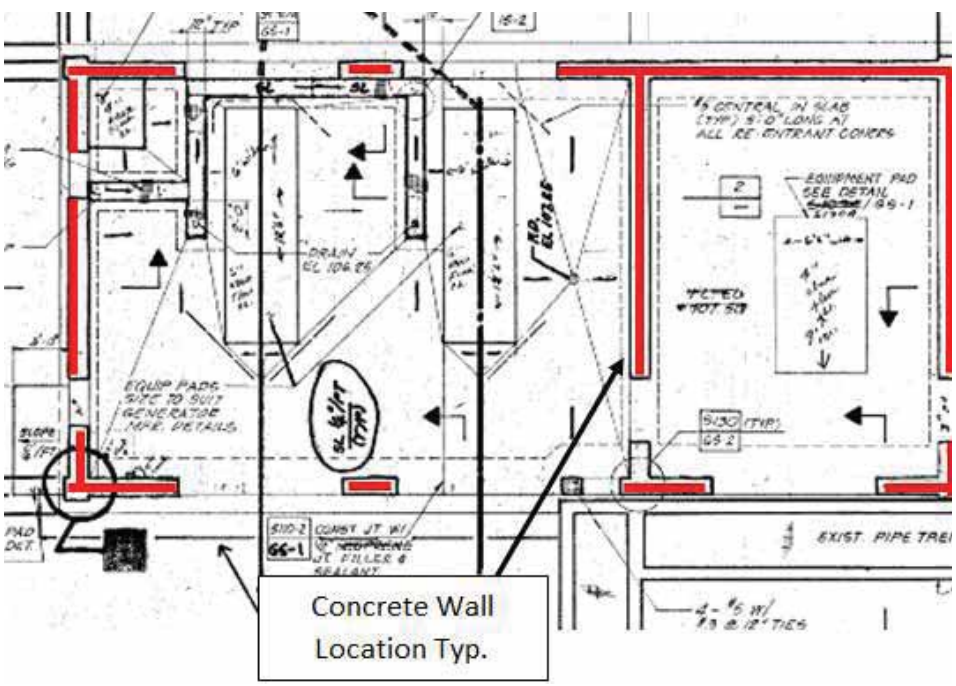

Figure 17: Generator Building #2 Wall Locations

3.3.8.1 Tier-1 Deficiencies

Wall Anchorage:

Based on tier 2 anchorage forces into the small rigid diaphragm, the anchorage connections and dowels provided are sufficient. However, the East-West running walls are attached to the precast roof beam stems which are likely not detailed to transfer out-of-plane loads into the diaphragm as they induce flexure into the thin roof diaphragm slab.

Transfer to Shear Walls:

The roof diaphragms are connected to the walls for shear transfer, but this load path is deficient. In the North-South direction, the dowels between the topping slab and concrete walls are slightly overstressed. In the East-West direction, no dowels are provided at the walls, so the inserts between the roof diaphragm beams have to transfer shear and are slightly overstressed. The embed attachments at the end walls are overstressed (300% DCR) and could break out of the concrete walls.

Topping Slab Connection to Walls or Frames:

The topping slab is not dowelled into the East-West running concrete walls.

3.3.8.2 Remediation Strategies

The remediation strategy for Generator Building #2 will involve supplementing the attachment between the precast concrete roof beams and the cast-in-place concrete walls. Additionally, it is likely that either the precast concrete beam interfaces will need to be strengthened, or additional ties between the precast beams and topping slab will need to be provided. We estimate the retrofit cost will be on the order of $150 per square foot. While the retrofit work is relatively simple and limited, this high cost is due to the small square footage of the structure.

3.3.8.3 Other Structures in Group

Generator Building #2 is in the structure group for cast-in-place concrete with precast concrete roofs. The other structures in this class are the Odor Control Building and Aeration Basins 1-4, which were also constructed during the mid-1980's expansion of the site. The Aeration Basins were assessed as part of the original Draft Report. Like Generator Building #2, the Odor Control Building was designed by James M. Montgomery Engineers during the same time period so the detailing and deficiencies are likely similar.

3.3.9 Alvarado Pump Station – Seismic Performance Rating = 6

The Alvarado Pump Station is a partially cast-in-place, partially precast concrete structure that was constructed during the 1985 phase of construction. The pump station is a three-story structure, with the first two floors below grade. The base slab, second and third floor slabs and beam elements, column and pilaster elements, and below grade walls are cast-in-place elements. The exterior walls above grade and roof structure are comprised of precast concrete elements. In the original 1985 construction, which is very similar to the Degritter building, the precast concrete roof elements were not well tied into the walls below and were not well tied to each other. The Alvarado Pump Station was seismically retrofit in 2000 by Carollo Engineers to address some of the issues with the original 1985 construction. A view from the inside of the upper-story of the pump station showing the bracing added as part of the retrofit is shown in Figure 18. While the retrofits measures will enhance the seismic performance of the pump station, they did not mitigate all the seismic deficiencies. The possible deficiencies of the structure discussed in greater detail below.

Figure 18: Alvarado Pump Station Retrofit Bracing

The main lateral force resisting system of the Alvarado Pump Station is its concrete walls below grade, and its pre-cast and cast-in-place concrete walls above grade. The concrete walls above grade are highlighted in Figure 19.

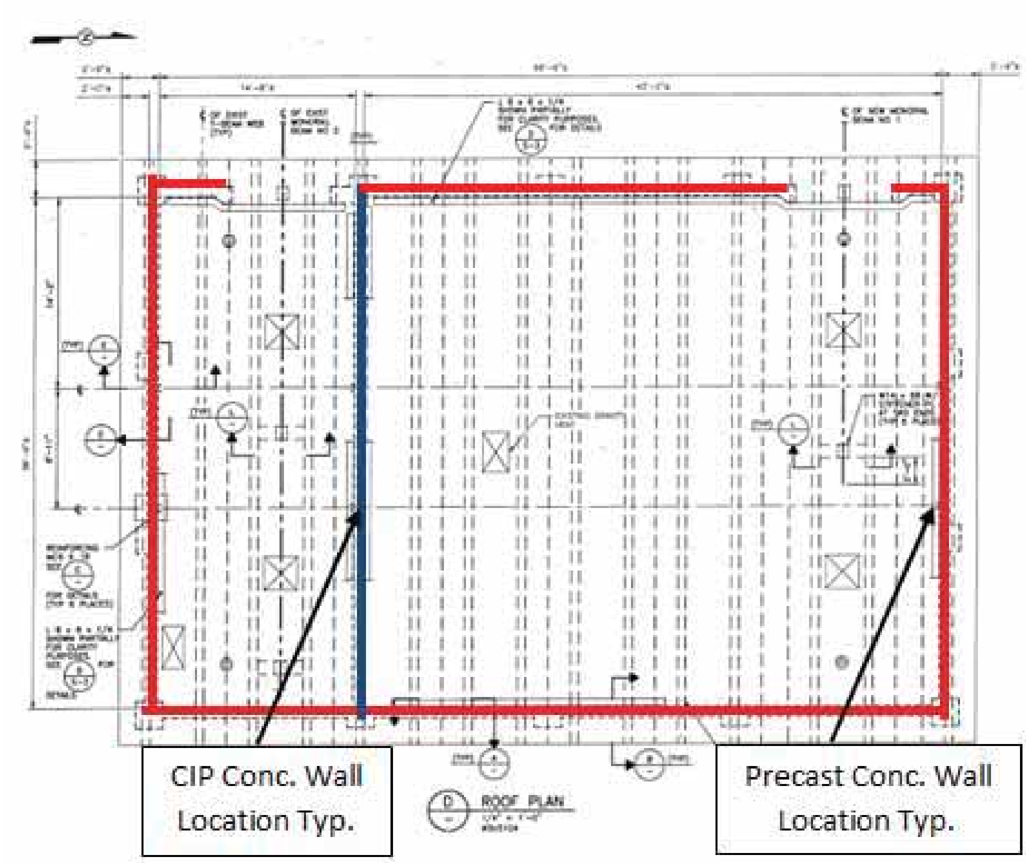

Figure 19: Alvarado Pump Station Concrete Wall Locations

3.3.9.1 Tier-1 Deficiencies

No Topping Slab at the Precast Concrete Roof:

The new steel bracing diaphragm installed as part of the 2000 retrofit is only connected to the columns and is not connected to the diaphragm. The primary load path for the precast wall panels is to span to the column elements, so it is likely that the retrofit intended to strengthen and stiffen this load path. Our preliminary analysis indicates that the columns are strong enough to cantilever from the third floor slab (at grade level) to the roof. Furthermore, the expected deflection at the top of the column is only about 1” without the added bracing. While this deflection is relatively small, it’s possible that the retrofit bracing system was intended to protect the in-plan shear transfer strengthening elements from trying to take out-of-plane forces and failing. Further analysis of the relative stiffness of the wall/column system, the bracing system, and the new in-plane shear connection seems warranted.

Because the new steel bracing diaphragm is not connected to the precast diaphragm above, the precast double-t beams and the diaphragm ties that connect the double-t beams still have to serve as the mechanism to transfer at least the self-weight of the roof to the concrete walls. Based on our preliminary analysis, the diaphragm ties are slightly overstressed.

Further analysis is necessary to confirm that actual accelerations at the roof level, and a condition assessment should be conducted to confirm the condition of the diaphragm ties. Based on observations from Primary Clarifiers 1-4, it is possible that the diaphragm ties are corroded and thus largely ineffective.

Transfer to Shear Walls:

The retrofit connections at the existing walls appear to be intended for in-plane seismic transfer but are deficient, especially at the North Wall. The new connections provided at the North Wall have a 700% DCR only considering the force from the diaphragm self-weight and the lowest possible roof acceleration. Note that the connections added at the middle wall appear to be different from what is shown on the retrofit drawings and require further study.

New retrofit connections to the East and West walls appear to only be provided at the East wall. From the original 1985 drawings the East wall is the end where the roof double-t girders are positively attached, so there appears to be no positive attachment at the West wall. Field exploration and testing are necessary to confirm the actual condition, as it may differ from what is shown in the original drawings. Still, even if the diaphragm were attached at both ends, the load path is almost certainly deficient because it relies on weak-way bending of the precast beam webs which will induce flexure into the thin diaphragm of the double-t beams.

Mezzanines:

The new mezzanines that were installed as part of the 2000 project do not appear adequately braced. Some of the mezzanines that are not shown on either the original drawings or the 2000 retrofit drawings do not appear to have any bracing. Further analysis is recommended.

Openings at Shear Walls:

There are a series of openings on both sides of the second floor diaphragm. Given the inertial loads in the diaphragm, this shouldn't be a life-safety issue for seismic load transfer with additional analysis. However, the cast-in-place slab diaphragms tie the walls of the structure together to resist incremental soil pressures during a seismic event and the slab may be overstressed to serve this purpose. Further analysis is recommended.

Adjacent Structures:

There is a large tower adjacent to the pump station that is supported off one of the corbels of the station. The movement of the structures should be minor but further analysis seems warranted.

Vertical Irregularities:

The middle East-West running wall is offset at the both the second and third floors. The wall runs the entire width of the structure so this is very likely not a deficiency for the gravity framing below with further analysis. The diaphragms should be sufficient to distribute the transfer forces, but this should be confirmed with additional analysis.

3.3.9.2 Remediation Strategies

For the Alvarado Pump Station to meet life-safety performance in the DBE, the in-plane shear transfer connections will need to be supplemented and it is very likely that the roof diaphragm connections will need to be supplemented. The shear transfer issue can be addressed either with the installation of additional connections similar to the ones installed in the 2000 retrofit or the installation of FRP angles at the roof/wall interface. The diaphragm discontinuity issue can be remediated through a number of strategies including adding FRP strips at the double-t joints, similar to the likely strategy at the primary clarifiers. Additionally, new bracing will have to be installed at the mezzanines.

Further work necessary to meet life-safety performance is pending additional analysis. It seems probable that some amount of connection enhancement and member strengthening of the 2000 retrofit bracing system will be necessary to protect those members. Likewise, additional analysis is necessary to confirm that openings in the floor levels at the pump stations will not leave the station vulnerable to effects of seismically induced soil pressures. Assuming that this additional work does not need to be completed, we estimate the retrofit cost to be on the order of $75 per square foot.

3.3.9.3 Other Structures in Group

The Alvarado Pump Station is the archetypal structure for retrofitted structures with cast-in-place concrete walls below grade, precast concrete walls above grade, and precast concrete roofs. The other structure in this class is the EBDA Effluent Pump Station and associated Reclaim Water Room. This structure's preliminary assessment can be found in Section 3.3.13.

3.3.10 Aeration Basins 1-4 – Seismic Performance Rating = 6

Aeration Basins 1-4 is a cast-in-place, partially embedded concrete wall structure that is partially filled with fluids. This structure was constructed during the 1985 phase of construction on the site. The basins are structurally very similar to other square celled tank structures at the site (e.g. secondary clarifiers) except that the basins have a precast concrete beams that form a lid on the structure. The precast concrete beams are not positively connected to one another and are only tied into the center influent portion of the structure. This roof structure is not expected to perform well in a seismic event. A view from the roof of the basins showing the precast roof beams is shown in Figure 20.

Figure 20: View of Aeration Basin Roof

In a seismic event, the thick walls at the perimeter of the basins and the thick walls through the center of the cells will cantilever from the foundation to resist the mass of the precast roof, the self-weight of the walls, and the impulsive and sloshing loads from the fluid. These concrete walls are shown in Figure 21. Based on preliminary analysis, the walls are sufficient to resist these seismic forces at a life-safety level. However, damage to the roof and its connections are expected. Furthermore, unlike at the perimeter of the basins, and at the distribution channels that run in the East-West direction, the center North-South running influent channel is not supported by a continuous wall. Rather, it is supported by columns ("bents" per the drawings). In addition to supporting the weight of the channel, the channel and the supporting bents must resist or distribute load from the concrete roof elements, and impulsive and sloshing fluid loads. This load path requires further study, as is discussed in greater depth below.

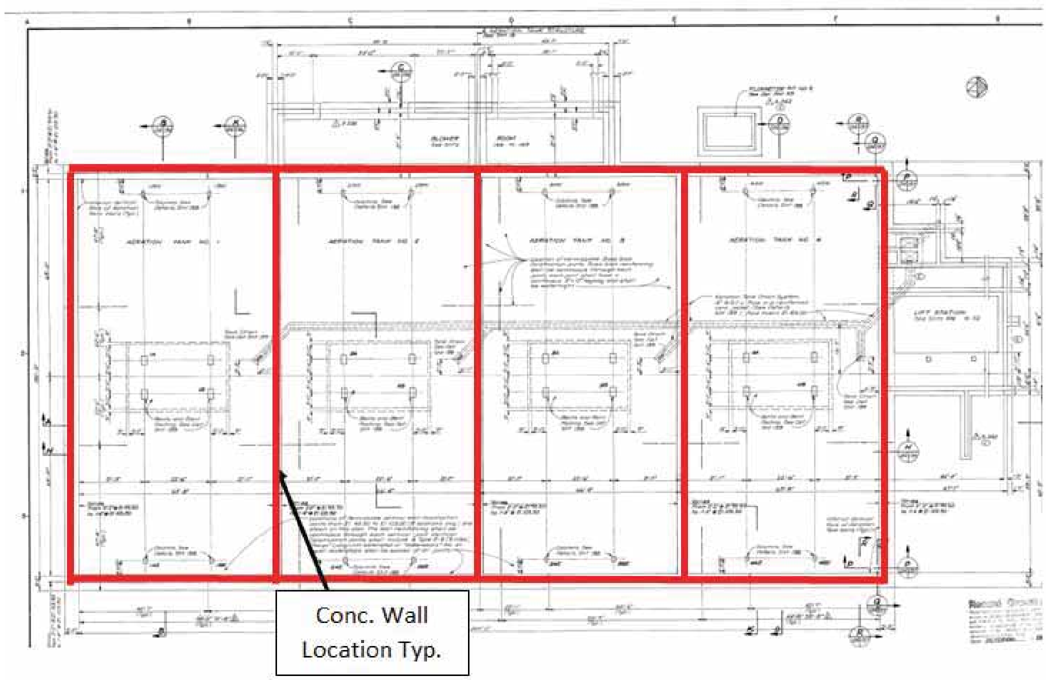

Figure 21: Aeration Basin Concrete Wall Locations

3.3.10.1 Tier-1 Deficiencies

Undefined Load Path/Transfer to Shear Walls/Diaphragm Continuity/Torsion at the Precast Concrete Roof:

For East-West seismic action, all of the load must be taken out at the center influent channel where the precast concrete beams are attached. For North-South seismic action, the roof diaphragm must cantilever from the end to which it is attached. The diaphragm elements are not positively connected to one another, so each double-t precast beam must cantilever its entire length. All of these actions are almost certainly deficient.

While this is not a life-safety hazard, it is possible the beams are damaged to the point where they fall into the tanks. Also localized pounding damage can be expected. The level of expected impact damage, however, is velocity dependent. The actual gap of the precast roof elements to the wall is only 1-2" based on site observations, which will limit velocities of the roof beams after the connections fail. Therefore, with further analysis it may be possible to show that impact related damage will be minimal.

Deflection Compatibility and Diaphragm Continuity for the Elements of the Influent Channel:

A rigorous analysis of the forces on and the behavior of the influent channel are beyond the scope of this report. Based on the discussion below, further analysis is required and strongly recommended.

There are 2 load paths for the center influent channel to resist East-West seismic action: the bents that support the channel can cantilever from the foundation, and the two diaphragms that comprise the channel can span to the East-West running walls. Preliminary analyses of these actions show that some level of ductility will be required for either load path.

Our preliminary analysis indicates that diaphragm action is the primary (stiffer) load path. Based this preliminary analysis, a potential deficiency in the load path is the lack of connection between the diaphragms and the East-West running walls. This load transfer requires further study, but will likely require additional doweled connections or steel plate connections. Furthermore, if the diaphragm load path is stiff enough to force double curvature into the bent supports, the bent supports are likely shear controlled, meaning that the diaphragm action must be stiff enough to prevent the columns from drifting far enough to fail. Our preliminary analysis indicates that the diaphragm is stiff enough to force the bents into double curvature, but the deflection of the diaphragms is small enough to protect the columns.

Adjacent Structures:

The aeration basins are connected to lift station #1. With more advanced analysis this is likely not a deficiency, as both structures appear to be stiff structures.

The aeration basins are also connected to east aeration blower room, which has a un-retrofitted precast concrete roof that could collapse and damage the aeration basins.

3.3.10.2 Remediation Strategies

It is our understanding that a new aluminum roof structure is being considered for the aeration basins. Installing a new lightweight roof would be a very effective way of dealing with the roof-related deficiencies and it would be beneficial from the perspective of reducing the mass of the roof structure (which would help protect the influent channels). Without conducing further, more analytically complex, studies however, it is difficult to know whether or not strengthening of the influent channels will be required. If strengthening of the influent channels is required, we would expect it to be a fairly expensive undertaking that would require temporarily draining the basins. Assuming that limited strengthening is needed at the intersection of the channels and including the replacement cost of a lightweight roof, we estimate the retrofit cost will be on the order of $75 per square foot.

3.3.10.3 Other Structures in Group

The Aeration Basins are the archetypal structures for cast-in-place concrete with precast concrete roofs. The other structures in this class are the Odor Control Building and Generator Room 2, which were also constructed during the mid-1980’s expansion of the site. From a brief review of the drawings, both structures have similar roofs to structures like the Primary Clarifiers, where the precast double-t roof beams run over the top of the concrete walls below. Note that these structures were designed by James M. Montgomery Engineers so the detailing differs slightly from the typical Kennedy/Jenks-designed structures and these two structures there appears to be some level of positive attachment of the roof to the walls. The method of attachment and extent of attachment from the two designs appears to differ (the Generator Room 2 appears to leave the seismic attachment up to the precast double-t contractor). We would expect to find some deficiencies with these two structures and recommend assessing them.

3.3.11 Control Building – Seismic Performance Rating = 6

The control building is a light cold-form steel framed partial two-story structure. The structure has a complex pitched, Spanish Clay Tile clad roof. An exterior view of the Control Building is shown in Figure 22. This Spanish Clay Tile roof is heavy and adds a significant amount of seismic mass to the otherwise light structure.

Figure 22: Exterior View of Control Building

The building was constructed during the 1978 phase of construction, and has similar ¾" plywood diaphragms with gauge metal straps at discrete locations to the FMC Maintenance building. Sheet metal X-bracing straps were originally designed to serve as the Control Building's seismic force resisting system. The locations of the sheet metal X-bracing straps are shown in Figure 23. Note how the bracing locations do not align.

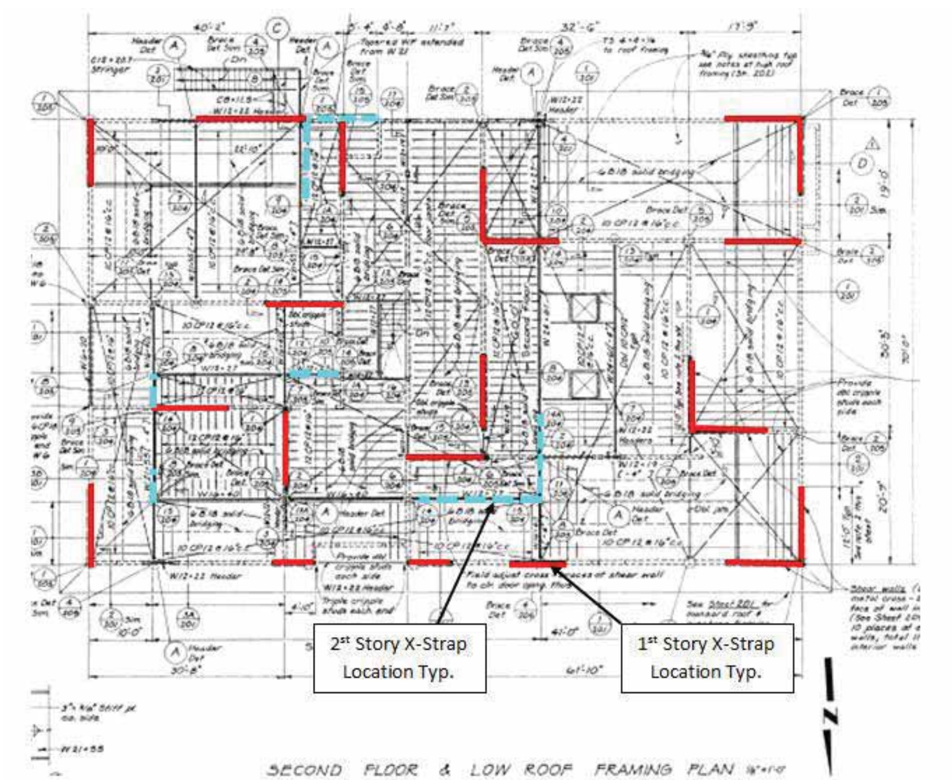

Figure 23: Control Building X-Strap Wall Locations

As noted in the deficiencies below, these straps are likely more flexible than the exterior stucco walls and interior gypsum board walls. Therefore, the strap walls will likely only be engaged after the other walls have sustained some level of damage. In a seismic event, the plywood diaphragms will transfer the inertial load to the various walls in the Control Building, which will ultimately transfer the loads to the foundation. Typically, light-framed, low-rise structures like the Control Building perform well in seismic events. However, due to the many discontinuities in the lateral system, and because of its heavy roof, the Control Building may not perform as well in the design seismic event as other light-framed structures. These potential deficiencies are discussed in greater detail below.

3.3.11.1 Tier-1 Deficiencies

Diaphragm Continuity, Chord Continuity, Unblocked Diaphragms:

There are many diaphragm discontinuities created by the mansard roof, offsets at the roof, and complicated joist/steel beam framing. These discontinuities will cause certain portions of the diaphragms to cantilever. The chord on the south side of the Control Building appears discontinuous. These potential deficiencies require further analysis, but are likely to be found deficient in some locations because of the light nailing of the plywood diaphragm.

Vertical Irregularities:

Every second-story metal strap wall does not align with a wall below. Therefore, the second floor diaphragm will have to transfer all the roof mass in addition to its own mass. Note that the plywood diaphragms are not well screwed and the strap bracing that is provided in the diaphragm is expected to be largely ineffective, as it is generally more flexible than the plywood and is only attached to the walls and framing below at discrete locations. Consequently, this vertical irregularity may overstress the diaphragm. Furthermore, the holdowns and vertical load transfer provided at the strap walls will not develop the strength of the straps, meaning that it is possible that connections rupture, studs buckle etc. at the second floor strap wall locations.

Shear Stress Check/Stucco Shear Walls/Gypsum Wallboard or Plaster Shear Walls:

The shear in the Control building will be distributed among the exterior stucco walls, interior gypboard walls and strap walls. While the lateral system does not exclusively rely on stucco or gypboard shear walls, these walls are likely stiffer than the metal strap walls and therefore require further study. It should also be confirmed though site investigations that the typical exterior stucco walls do not have a plywood backing.

All of the wall elements have different stiffnesses, so they will be engaged at different drift levels during an earthquake. It is recommended that further analysis be conducted to determine the actual expected load distribution among the various wall elements. Demand-to-capacity ratios (DCRs) for different wall elements are shown below. Note that we do not believe it is appropriate to consider all 3 different wall elements concurrently without additional analysis.

DCRs for estimated amount of gypboard only ~300-500%.

DCRs for exterior stucco only ~200-300% (considering 350 plf for stucco and the same quickcheck m-factor of 4.0 which is unconservative).

DCRs for exterior stucco and interior gypboard ~150-250%.

DCRs for metal strapping only 200-400%.

Deficient Sill Bolting:

The gypboard and stucco walls are only connected to the foundation with shot pins. This is likely not a life-safety hazard for the interior gypboard walls, but could be a life-safety hazard if the exterior stucco walls are taking most of the seismic load.

Narrow Wood Shear Walls:

There are some very short strap walls on the north side of the building which are likely to be very flexible.

Strap Connections:

The welded strap connections are sufficient to develop the tensile strengths of the straps. However, welding gauge material can be difficult and it is possible that the welds are burned through the gusset plates and/or straps. We recommend performing field investigations to determine the quality of these welds.

3.3.11.2 Remediation Strategies

It is likely that the results of further analysis will show that some level of wall and diaphragm strengthening will be required for the Control Building to meet life-safety performance in the DBE. The wall strengthening can be achieved by installing plywood on select walls, and the diaphragm strengthening can be typically achieved by installing blocking and/or adding additional screws through the framing members. Additionally, it seems likely that new wood or steel members will need to be introduced at select locations where the diaphragm steps or cantilevers. The extent and locations of these members will be largely dependent on the extent and locations of the new plywood shear walls.

Without further analysis it is not possible to identify the scope of wall and diaphragm strengthening required. However, it likely that only certain portions of the diaphragm will require strengthening and only select walls will need plywood. However, both of these strengthening solutions require the removal and replacement of architectural finishes (roofing etc.), and are thus relatively expensive.

It is our understanding that the control building is critical structure that needs to be operational shortly after the DBE event. Therefore, we assume that the retrofit work will be more extensive than for life-safety performance of a similar structure. We estimate the cost of the retrofit work will be on the order of $200 per square foot.

3.3.11.3 Other Structures in Group

The Control Building is the only structure assigned to its group. Consequently, no extrapolation is required.

3.3.12 FMC Maintenance Building/Generator Building #1 – Seismic Performance Rating = 6

The FMC Maintenance/Generator Building is a reinforced masonry bearing wall structure with a pitched, Spanish Clay Tile clad roof. The roof is supported by steel trusses and cold-formed steel joists. An exterior view of the FMC Maintenance Building is shown in Figure 24. The structure was constructed primarily during the 1978 phase of construction, and was an addition to a much smaller cast-in-place reinforced concrete structure which was originally constructed in the early 1960's. The roof is sheathed with a lightly screwed ¾" plywood diaphragm and is strapped with gauge metal straps at discrete locations.

Figure 24: Exterior View of FMC Maintenance Building

In a seismic event, the plywood diaphragms will transfer the load to the reinforced masonry shearwalls which are anchored to the concrete foundation. The masonry wall locations are shown in Figure 25. In buildings with heavy walls and light roofs, like the FMC Maintenance Building, most of the seismic damage is related to deficient anchorage between the heavy walls and the light roof diaphragm because the walls try to pull away from the diaphragm during an earthquake. The anchorage in the FMC Maintenance Building is more robust than many from its era, but it still is deficient as discussed below.

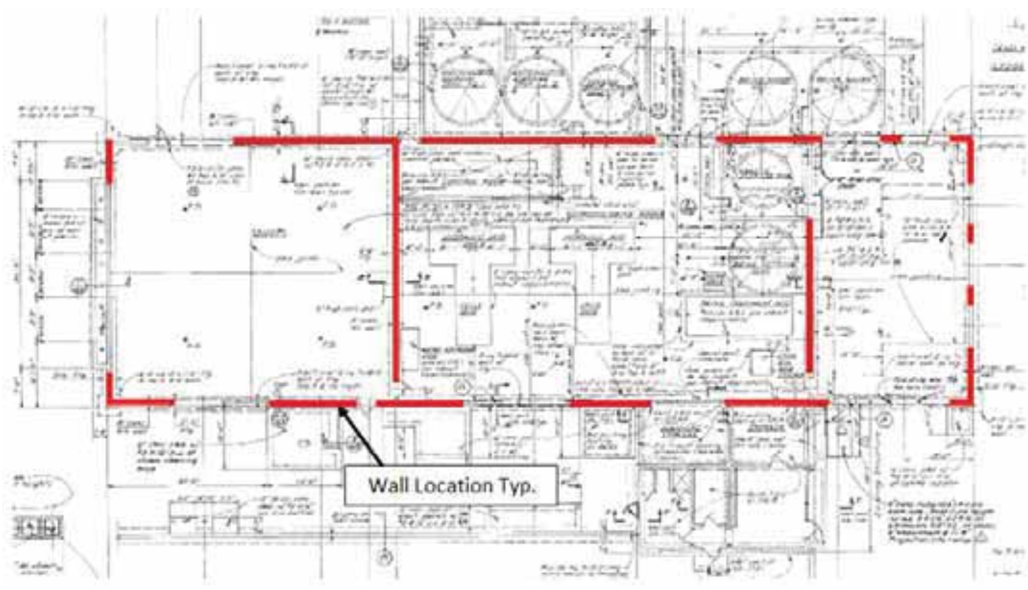

Figure 25: FMC Maintenance Building Reinforced Masonry Wall Locations

3.3.12.1 Tier-1 Deficiencies

Out-Of-Plane Wall Anchorage:

The interior walls are well anchored to bracing and/or studs at the roof level. However, the lateral load path appears to rely on bending of 10 Ga plates (see 2/172) which is not adequate.

The exterior walls are directly anchored to the trusses/framing beams, which creates a strong and stiff out-of-plane wall connection. The North/South walls meet life safety requirements, but the East/West walls are slightly deficient.

Transfer to Shear Walls:

The interior walls rely on strong way bending of 10 Ga plate which may be deficient, but are otherwise well anchored.

The East/West walls have 18 gauge closures to transfer in-plane load, but these rely on 1/8" welds that could have burned through the gauge material. This needs to be investigated further.

From the drawings (6/172), the North/South walls are only connected at the trusses, with no blocking/closures between them. This is inadequate and could lead to the joists rolling. However, blocking/closures is shown on Section A-A on sheet 169. Further site investigation is necessary to determine the actual condition and determine the condition of the welds.

Openings at Exterior Masonry Shear Walls:

The large dormer at south wall does not appear to be connected to the exterior wall and the diaphragm does not appear to be continuous over the dormer. Similarly, the louver on the west side of the roof represents a discontinuity in the rood diaphragm. The smaller dormers at the east and northwest sides of the roof are noted, "Installed Plywood Backing at all Dormer Walls" but the detailing of the plywood should be confirmed.

Unblocked Diaphragms Spanning Greater than 40 Feet:

The diaphragms are not well screwed and have to relatively long spans. The strap bracing that is provided in the diaphragm is expected to be largely ineffective, as it is generally more flexible than the plywood and is only attached to the walls and framing below at discrete locations.

Unbraced Mezzanines:

The main mezzanine in shops area is tied into a center shear wall on one side and the bottom chord of the truss on the other side. The truss should be evaluated to transfer this load, but trusses appear robust and should be able serve as a transfer.

The small offices appear to have been constructed after the original construction. These are typically partial height and do not appear to be well tied into the surrounding structure. These small offices need further study and are very likely deficient.

3.3.12.2 Remediation Strategies

While the final scope of remediation will require further analysis and site investigations, it is likely that the roof-to-wall connection is going to require strengthening at the majority of the wall locations. Details can be developed so that this scope of work can be done from the inside of the building without removing the roofing.

Although it should be confirmed with further study, it is also likely that the roof diaphragm nailing is insufficient given the long spans, and heavy roof and walls. Additionally, details will likely be required to address the discontinuities (louvers and dormers) in the roof diaphragm. It may be most cost-effective to remove the roofing to address these issues. We estimate the cost of the retrofit work to be on the order of $75 per square foot.

3.3.12.3 Other Structures in Group

The FMC Maintenance Building/Generator Building #1 is the archetypal structure for reinforced masonry structures with flexible roofs built between 1978 and 1984. The other structures in this class are the Fremont Lift Station and Paseo Padre Lift Station, both built in 1984. From a brief review of the drawings, the lift station structures are much smaller than the FMC Maintenance

Building and both have underground cast-in-place portions of the structure. Note that the Fremont Lift Station drawings appear incomplete but the structure seems similar to the Paseo Padre Lift Station. Because the above grade portions of the lift stations appear small, we would not expect to find the same diaphragm deficiencies with the lift stations as with the FMC Maintenance Building. However, we would expect to find similar out-of-plane anchorage deficiencies. Note also that based on our review of small similar below grade structures, we would not expect significant deficiencies with the below-grade portions of these structures.

3.3.13 EBDA Pump Station/Reclaim Water Room – Seismic Performance Rating = 5

The EBDA Pump Station/Reclaim Water Room is a partially cast-in-place, partially precast concrete structure that was constructed during the 1978 phase of construction. The pump station is a two-story structure, with the bottom-story below grade. The base slab, second floor slab and beam elements, column and pilaster elements, and below grade walls are cast-in-place elements. The exterior walls above grade and roof structure are comprised of precast concrete elements. In the original 1978 construction, which is very similar to Primary Clarifiers 1-4, the precast concrete roof elements were not well tied into the walls below and were not well tied to each other. The pump station was seismically retrofit in 1995 to address some of the issues with the original 1978 construction. A view from the inside of the upper-story of the pump station showing the bracing added as part of the retrofit is shown in Figure 26. While the retrofits measures will enhance the seismic performance of the pump station, they did not mitigate all the seismic deficiencies. The possible deficiencies of the structure discussed in greater detail below.

Figure 26: EBDA Pump Station Retrofit Bracing

The main lateral force resisting system of the EBDA Pump Station is its concrete walls below grade, and its pre-cast and cast-in-place concrete walls above grade. The concrete walls above grade are highlighted in Figure 27.

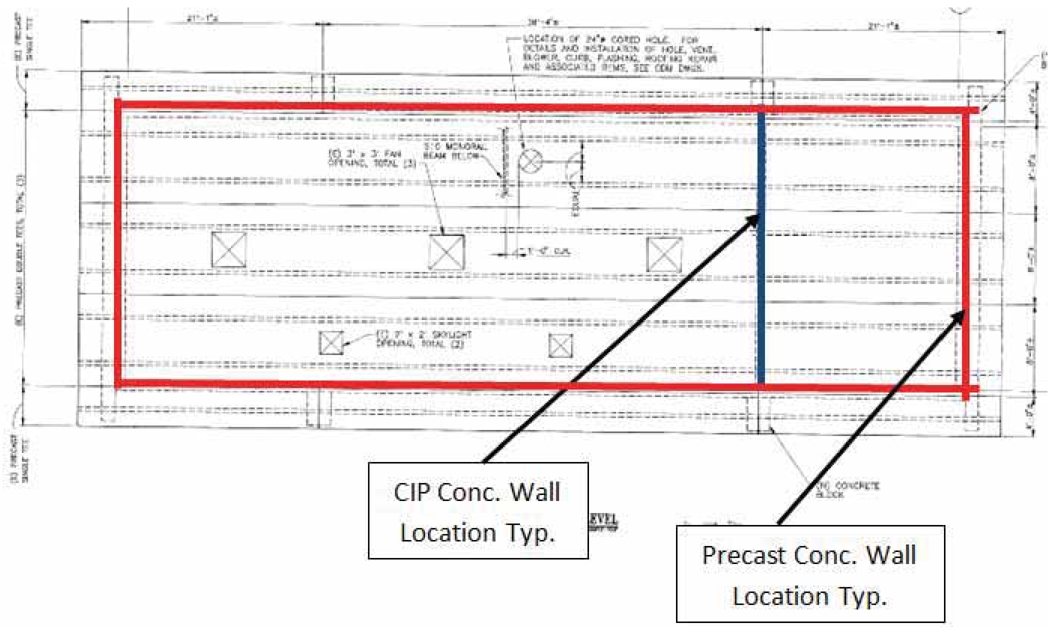

Figure 27: EBDA Pump Station Concrete Wall Locations

3.3.13.1 Tier-1 Deficiencies

No Topping Slab at the Precast Concrete Roof:

The new steel bracing diaphragm is only connected at the walls to take the out-of-plane wall weight. The DCR for diaphragm diagonals is ~70-100% at a force controlled level for the weight of the walls. Further analysis of all connections and members is warranted to confirm this load path is sufficient.

The precast double-t and the diaphragm ties have to serve as the mechanism to transfer the self-weight of the roof. Note that the diaphragm is not tied into the interior cast-in-place concrete wall, so the diaphragm ties along this wall have to transfer shear and moment. Based on this analysis, the diaphragm ties are slightly overstressed to span the load over the interior wall.

The connection between the precast roof beams is not indicated on the drawings. For the diaphragms to act together in East-West seismic loading, additional shear transfer will be required even if there are diaphragm ties between the precast roof beams. However, the roof beams are likely sufficient to span out-of-plane as individual elements without interconnectivity. Further study is required.

Transfer to Shear Walls:

There is no direct connection of the precast diaphragm to the interior cast-in-place wall. The precast beam stems can bear against the infill concrete panels, but this load path relies on weak-way bending of the roof beam stems to be resolved in the thin diaphragm which is very likely deficient. Without shear transfer at the interior wall, the roof beams must rely on the welded inserts that are over the wall to transfer shear and moment to the exterior walls. This load path is deficient.

There is no connection of the precast diaphragm to the walls on gridlines 1 and 2 (bond breaker is provided where beams cross exterior columns). Note that the beams can act as struts, and transfer their inertia through the eve beams which can then transfer this load through the diaphragm bracing, but this load path appears slightly deficient. It may be possible that this load path is sufficient if the diaphragm is tied into the center wall for out-of-plane load transfer. Further study is recommended.

Wall Anchorage:

The original welded inserts at the base of the walls are not sufficient for out-of-plane load. However, the wall panels are either sufficient to span horizontally to the columns or horizontally to the base shear lugs that were provided as a part of the retrofit.

The new retrofit connections at the top of the walls are sufficient to resist the out-of-plane load from the panels but may not be sufficient to resist the in-plane forces that result from the out-of-plane load (shear flow into chord) depending on how much load is transferred into angle chord and how much load directly transfers through the embed plate. Further analysis is recommended.

For in-plane shear transfer at the base of the walls, the shear lugs added as part of the 1995 retrofit appear slightly overstressed.

Openings at Shear Walls:

There are a series of openings on both sides of the second floor diaphragm. Given the inertial loads in the diaphragm, this shouldn't be a life-safety issue for seismic load transfer with additional analysis.

Geometry:

The length of the walls at the base-story is longer than upper-story. This is very likely not a life-safety concern with additional analysis.

Vertical Irregularities:

The last 17 feet of the East North-South running wall is discontinuous. The wall runs the entire width of the structure so this is very likely not a deficiency with further analysis. The diaphragms should be sufficient to distribute the transfer forces.

3.3.13.2 Remediation Strategies

For the EBDA Pump Station to meet life-safety performance in the DBE, the center cast-in-place wall will very likely need to be connected to the roof diaphragm. Further work necessary to meet life-safety performance is pending additional analysis. Possible additional work would include strengthening the connection between the precast roof beams, strengthening the members and connections of the 1995 retrofit, and adding additional shear lugs at the base of the walls. We estimate the retrofit cost to be on the order of $100 per square foot.

3.3.13.3 Other Structures in Group

The EBDA Pump Station is in the structure group of retrofitted structures with cast-in-place concrete walls below grade, precast concrete walls above grade, and precast concrete roofs. The other structure in this class is the Alvarado Pump Station, which has been assessed as part of the original Draft Report.

3.3.14 Covered Storage – Seismic Performance Rating = 5

The covered storage building is a lightweight open steel structure supported by cantilevered columns. An exterior view of the covered storage structure is shown in Figure 28. The building was constructed in 2000 and was designed to the 1994 Uniform Building Code. Because the columns cantilever from the base supports for seismic resistance, the covered storage structure is very flexible structure. A small reinforced masonry mezzanine structure is located at the West end of the covered storage structure. Although the mezzanine is structurally independent of the steel covered storage structure, it is not isolated from the covered storage structure and the two may interact in a seismic event because of the flexibility of the covered storage structure. The column locations and reinforced masonry wall locations are shown in Figure 29.

Figure 28: Exterior View of Covered Storage Building

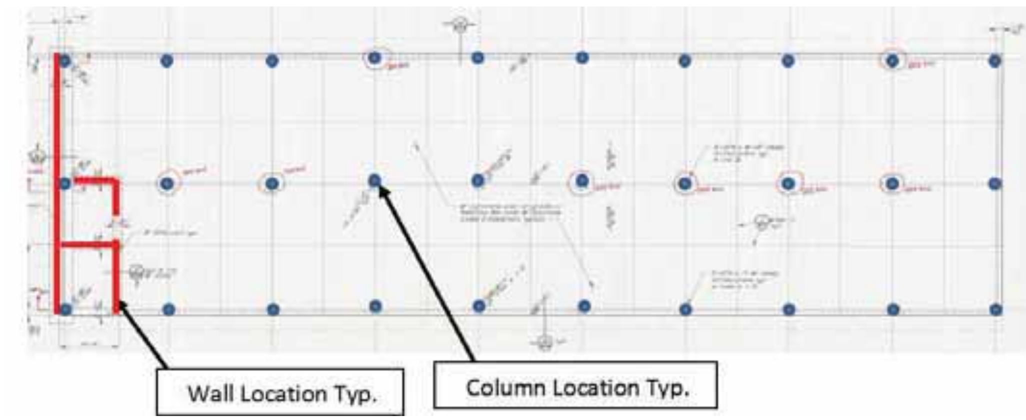

Figure 29: Covered Storage Building Column and Masonry Wall Locations

The findings, as noted below, assume that the columns are positively connected to the concrete bollard pedestals that protect the base of the columns from vehicle impacts. The findings also assume that the bollard pedestals are doweled into the foundations. Because there is no information on the structural drawings regarding these issues, it is recommended that destructive and/or non-destructive tests be conducted to make these determinations. If it is found that the bollards are not positively connected and/or doweled, the drift levels and damage discussed below will increase. The columns will also very likely be overstressed if they are not positively connected to the bollard pedestals.

3.3.14.1 Tier-1 Deficiencies

Adjacent Structures:

At the reinforced masonry structure between gridlines 1 and 2, there is only ~1 ½" separation provided between masonry wall and covered storage column on gridline B. Additionally, this covered storage column is tight to the plywood diaphragm that serves as the roof of the masonry wall structure. The expected drift at middle column line is ~3+" in each direction so pounding damage is expected at the masonry wall and the plywood diaphragm.

Torsion:

The covered storage roof diaphragm is flexible so torsion deficiencies do not technically apply. However, the reinforced masonry walls are MUCH stiffer than cantilever column system. Therefore, if columns get hung up on diaphragm, a large differential deformation will be induced in the roofing members, which could lead to damage.

Flexural Stress Check of Cantilever Columns:

The average stress among all columns is less than the yield stress, so the columns are not deficient per ASCE 41-13 Tier-1 analysis. However, the middle row of columns may be slightly overstressed in E-W seismic action. Additionally, if the columns are not positively connected to the pedestal bollards, the columns would very likely be overstressed. Further testing is required to determine the condition of the pedestal bollards.

Shear Transfer to Steel Frames/Other Diaphragms:

The rounded corrugated metal panel roof will not work well as a diaphragm. However, the diaphragm shears are very small so the roof may be sufficient. For the typical 25’ span in N-S seismic action the diaphragm shear is only ~125 plf. Similar 24 Ga roofs with flat edges (e.g. Verco “Vercor”) have allowable diaphragm shears approaching 150 plf depending on the method of attachment. Further analysis is recommended, and it is necessary to assess the method of connecting the deck to the steel supports, as this is not shown in the drawings.

If the deck does fail, the diaphragm action in the N-S direction will rely on channels spanning weak-way between the truss lines. The channels are susceptible to rolling at truss locations between blocking members which could lead to a partial collapse(s) of the roof.

Insufficient Moment Resisting Connections:

The base connection at the HSS columns is not capable of developing moment strength of tube but it may not have to if load is adequately transferred to concrete pedestal. It is unclear if concrete pedestals are doweled into foundation or are positively connected to cantilever columns. This needs to be confirmed through testing.

The embedded poll foundation capacity cannot develop the HSS tube capacity based on default bearing pressure values. Further analysis of the poll foundations is recommend, but they are likely deficient per new code standards.

Cross Grain Bending in Wood Ledgers:

The wood ledges in the small reinforced masonry structure do not have to be subject to cross grain bending because the walls can span horizontally to cross walls or they can cantilever. However, this load path may be initially more flexible than loading the face mounted joists. The nails on the face mounted joists will withdraw at a very low force level and then the ledger would be subject to cross-grain bending. Because of the alternate load path, this is likely not a significant life-safety concern, but should be studied further.

3.3.14.2 Remediation Strategies

Most of the potential deficiencies identified for the covered storage building require further site investigation and study, but it is almost certain that the separation between the reinforced masonry walls and the steel columns needs to be increased. This may require moving (or demolishing and rebuilding) the reinforced masonry walls.

The potential life-safety deficiencies regarding the column bollards and roof diaphragm can be mitigated fairly easily. Even if the bollards are not doweled into the foundation and positively connected to the columns, these connections can be added with new plates and epoxy anchors.

Similarly, if the roof diaphragm attachment is found to be deficient, it can be supplemented with new fasteners.

The most significant unknown regarding the covered storage building is adequacy of the existing poll foundations. As noted above these will almost certainly be found to be deficient using simplified code procedures, so specific geotechnical input is required. Supplementing these foundations would require the introduction of new foundation (most likely grade beam) elements, which would significantly add to the cost. Assuming that no foundation work is required, we estimate the retrofit cost to be on the order of $25 per square foot.

3.3.14.3 Other Structures in Group

The Covered Storage Building is the archetypal structure for UBC 1994 and later, open steel structures. The other structures in this class are the Fuel Island which was also built in 1999, and the Solar Carport and Emergency Storage Pond at the Irvington Pump Station Site which were built in 2009-2010. From a brief review of the drawings, all 3 structures appear to be cantilever column structures with poll foundations. With the possible exception of finding deficiencies with the depth of embedment of the poll foundations, we would not expect to find serious deficiencies with these simple and relatively modern steel structures. However, based on our findings from our review of the Covered Storage Building, it is quite possible that additional deficiencies exist. Furthermore, during our 3/18/14 site visit of the Covered Storage Building, we noticed that some of the connections at the top of the columns in the Solar Carport may be deficient. Consequently, further study of these additional structures seems warranted.

3.3.15 Primary Digester #5 – Seismic Performance Rating = 5

Primary Digester #5 is a round cast-in-place concrete tank structure with a steel dome that was constructed during the 1985 phase of plant construction. An exterior view of the digester is shown in Figure 30. The steel dome is usually under positive pressure from the gasses within the tank, and is held down by steel tube supports and anchors along the perimeter of top of the tank.

Figure 30: Exterior View of Primary Digester #5

During the seismic event, the fluid within the tank will be accelerated, which will place shear stresses and additional hoop stresses on the circular concrete wall. Similarly, the earthquake will put additional stresses on the HSS anchors that support the steel roof, and it will put additional stresses on the concrete tube column in the center of the tank. While the tank walls and roof anchorage are robust, the seismic induced stresses in these elements are high and further analysis is recommended to determine how they will perform in a seismic event. Possible deficiencies are noted below.

3.3.15.1 Tier-1 Deficiencies

Anchorage at Center Column:

The anchorage at the center of the column is deficient and the column may break off the concrete foundation in the DBE. Note that the calculations indicate that the anchorage is overstressed even not considering that the center pipe is anchored adjacent to another pipe. If this edge condition were considered the anchorage would be even more overstressed.

Freeboard:

The freeboard requirement in the design seismic event is approximately 3 ½ feet, but only ~1 foot of freeboard is provided. The steel dome is anchored down to the concrete walls at the perimeter, so it is possible that this lack of freeboard is not a significant issue. This requires further study.

Shear and Hoop Stress in Concrete Walls:

Based on preliminary analysis, the shear stress in the concrete walls is ~105 psi which meets the Tier-1 life-safety requirement, but is very close to being overstressed. Similarly, based on our preliminary analysis the cracking hoop stress is slightly exceeded in the DBE, but the transverse reinforcement should be sufficient to withstand these additional seismic-induced hoop stresses. Further analysis, particularly geotechnical input on the passive resistance of the soil, may reduce these stresses.

Note however, the preliminary analysis considers the tank to be in good condition. Based on our 3/28/14 site visit, there are large vertical cracks that have been repaired. These represent weak points where the tank is likely to crack in a seismic event. Furthermore, if the rebar was corroded when these cracks initially formed, it is possible that the rebar will be overstressed in a seismic event.

Deflection Compatibility/Other Diaphragms:

The stability of the roof appears, in part, to be reliant on the solid bearing between the thrust apparatus at the edge of the roof and the inside face of the concrete wall. It is unknown how this connection will respond to either sloshing fluid, or deformations to the circular shape of the tank. This is likely not a life-safety concern in the case of vertical pressures due to sloshing, as the roof is already under positive pressure from the methane gasses. Likewise, the tank walls are already under pressure from the hydrostatic weight of the fluid in the tank, and while the seismic forces nearly double the load on the tank walls, it seems unlikely that this added deformation will cause the roof to completely fail. However, further analysis is recommended.

Note that the original calculations for the digester roof designed the thrust ring for conservative vertical loads, but did not analyze the thrust ring connection under sloshing loads and potential wall deformations under seismic loads.

Transfer to Shear Walls:

The calculations indicate that the roof attachment (HSS tubes and anchorage) to the concrete walls is sufficient to withstand the incremental forces from the DBE (as well as the vertical positive gas pressure). However, based on our 3/28/14 site visit, the anchorage attachments are significantly corroded, and therefore may not be able to withstand the additional stresses from a seismic event.

3.3.15.2 Remediation Strategies

The only obvious and immediate deficiency of Primary Digester #5 that requires remediation is the anchorage of the pipe at the center of the tank. The other deficiency that will definitely require eventual attention is the condition of the corroded roof attachments. It is our understanding that the digester roofs are scheduled to be replaced prior to replacing the entire tank. We recommend that when the tank roofs are replaced that the sloshing fluid forces and the deflection compatibly at the tank walls be specifically considered. Not considering the replacement cost of the roof, we estimate that the seismic work to anchor the main pipe and replace the corroded connections would be on the order of $25 per square foot.

3.3.15.3 Other Structures in Group

Primary Digester #5 is the archetypal structure for steel topped concrete dome structures that have not been retrofitted (or have not had their domes replaced). The structure class includes all the primary digesters: Primary Digesters #1 through #4 and Primary Digester #6. Primary Digester #5 is most similar to Primary Digester #4, as they were built at the same time and designed the same way. The only major difference between the two structures would be the condition of the concrete walls and steel dome roof. Consequently, it is reasonable to extrapolate the findings of Primary Digester #5 to Primary Digester #4.

Primary Digester #1 and #2 were constructed in 1978 and have the same structural design as each other. From a brief review of the drawings, it appears that the center pipe anchorage and steel dome anchorage are similar to Digester #5. Primary Digesters #1 and #2 are slightly shorter and smaller, and have slightly thinner walls than Primary Digester #5. The horizontal reinforcing for digesters #1 and #2 is similar to #5, and the tanks are embedded into the surrounding soil a similar amount. Given all these similarities it is reasonable to extrapolate the findings of Digester #5 to Digesters #1 and #2. Note however that #1 and #2 are older than #5, and therefore are expected to be more corroded than #5. Consequently, a condition assessment of these digesters seems warranted.

Primary Digester #3 was constructed in 1962. From a brief review of the drawings, it appears that the Primary Digesters #3 is slightly shorter and smaller, and has slightly thinner walls than Primary Digester #5. The horizontal reinforcing for digesters #3 is similar to #5, and the tanks are embedded into the surrounding soil a similar amount. The main difference between the tanks is that the center pipe does not appear to be anchored into the concrete foundation. This condition should be studied further. Note also that #3 is much older than #5, and therefore is expected to be more corroded than #5. Consequently, a condition assessment seems warranted.

Primary Digester #6 is larger than the other digesters and was constructed more recently. From a brief review of the drawings, the walls are thick and are well reinforced and the roof anchorage appears robust, and is even tied with supplemental reinforcing. Consequently, it does not appear that these conditions are deficient, but this should be confirmed with calculations. As with Primary Digester #3, the center pipe in Primary Digester #6 does not appear to be anchored to the foundation and should be studied in greater detail.

3.3.16 Secondary Digester #1 – Seismic Performance Rating = 5

Secondary Digester #1 is a round cast-in-place concrete tank structure with a steel dome that was constructed during the 1978 phase of plant construction. The steel dome and support attachments were replaced in 2011. A view of Secondary Digester #1 and its steel dome can be seen in Figures 31 and 32. The steel dome is usually under positive pressure from the gasses within the tank, and is held down by steel tube supports and anchors along the perimeter of top of the tank.

Figure 31: Exterior View of Secondary Digester #1

Figure 32: Secondary Digester #1 New Steel Roof and Anchorage

During a seismic event, the fluid within the tank will be accelerated, which will place normal shear stresses and additional hoop stresses on the circular concrete wall. Similarly, the earthquake will put additional stresses on the HSS anchors that support the steel roof. The seismic induced stresses in these elements are high and further analysis is recommended to determine how they will perform in a seismic event. Possible deficiencies are noted below.

Note that the fluid height in Secondary Digester #1 varies, and the preliminary analysis assumed a fluid height of 35 feet above the base. This fluid height is significantly lower than the maximum fluid height indicated on the drawings, but matches the maximum fluid height reached in the digester during the month of March 2014 based on input from the District. Because the probability of a significant seismic event occurring at the same time as the maximum fluid design height is very small based on input from the District, it is judged that analyzing for the maximum height at a particular month is sufficiently conservative.

3.3.16.1 Tier-1 Deficiencies

Tubes Extending From the Roof to the Center of the Tank:

The drawings indicate that there are a number of pipes that extend below the fluid line and are braced back to the steel dome roof. The pipes will be subject to hydrodynamic forces during a seismic event, and further analysis is recommended to determine the adequacy of the pipes and their connections.

Shear and Hoop Stress in Concrete Walls:

Based on preliminary analysis, the shear stress in the concrete walls is ~110 psi which slightly exceeds the Tier-1 life-safety requirement. Further analysis, particularly geotechnical input on the passive resistance of the soil, may reduce these stresses. Also considering the average fluid height in the tank (which is nearly 10 feet lower than the 35 feet assumed) would significantly reduce the maximum stresses on the walls.

Note also, that the preliminary analysis considers the tank to be in good condition. Based on our 3/28/14 site visit, there is a large horizontal crack about mid-height in the exposed portion of the tank. This crack has since been repaired. While the stresses are low enough this high on the tank, similar corrosion/cracking could be a concern lower in the tank, and further exploration and analysis is recommended.

Transfer to Shear Walls/Deflection Compatibility:

Based on preliminary analysis and assuming the anchorage bolts are sufficiently torqued to prevent slippage, the anchor bolts could bend if the base plate slips relative to the grout pad below. In this case, the anchorage would be insufficient. More likely is that the bolts slip, which means that because the connection is slotted in both directions there is no mechanical means of transferring the seismic shear of the roof.

Therefore, the roof is likely to push against the float control basin on one side and pull against it on the other side. This involves a bending of the side sheet plate, and interplay between the seismic reactions and the internal gas pressure. Further study on this mechanism is recommended.

Additionally, although it is beyond the scope of this seismic evaluation, the roof anchorage was checked for the forces from the positive gas pressure indicated on the dome replacement drawings. During the shop drawing process, the drawings indicate that the epoxy anchors were reduced from 1" diameter anchors with a 10" embed, to 5/8" anchors with a 6" embed. We assume that the person making this change neglected to consider the eccentricity of the HSS tube acting on this connection. When the eccentricity is considered, the anchorage is overstressed.

Other Diaphragms:

The roof is a solid welded plate diaphragm and is likely sufficient. Further analysis is suggested to confirm it will not buckle.

3.3.16.2 Remediation Strategies

As with the Primary Digester #5, the remediation scope of Secondary Digester #1 is largely dependent on the findings of further analysis and specific geotechnical recommendations. The only obvious deficiency that needs immediate attention is the current tank roof anchorage, which may break out from the top of the wall under excesses positive pressure from the tank. While the retrofit scope needs to be more clearly defined by additional study, we expect that the retrofit cost will be on the order of $40 per square foot.

3.3.16.3 Other Structures in Group

Secondary Digester #1 is the archetypal structure for steel topped concrete dome structures that have been retrofitted (or had their domes replaced). The structure class also includes Secondary Digester #2. The only significant difference between the two structures is that the digester #2 roof was replaced in 2009. From a brief review of the drawings, it appears that the dome replacements were very similar construction. Additionally, the Secondary Digester #2 replacement drawings indicate that the dome anchorage (5/8" epoxy anchors with 6" embedment) is the same as the digester #1 anchorage. Consequently, it seems reasonable to extrapolate the findings from the Secondary Digester #1 to Secondary Digester #2.

3.3.17 EBDA Effluent Surge Tower – Seismic Performance Rating = 4

The EBDA Surge Tower is a tall, narrow, cast-in-place concrete tower constructed during the 1978 phase of construction at the site. An exterior view of the Surge Tower is shown in Figure 33. A large 60” diameter pipe exits the tower near its base, which is located approximately 15 feet below grade. The walls of the tower are thick and are well reinforced, and the tower is founded on a 2-foot thick 25-foot square reinforced concrete mat foundation. In a significant seismic event, the walls will work to distribute the seismic induced inertial and fluid forces to the foundation and into the surrounding soil.

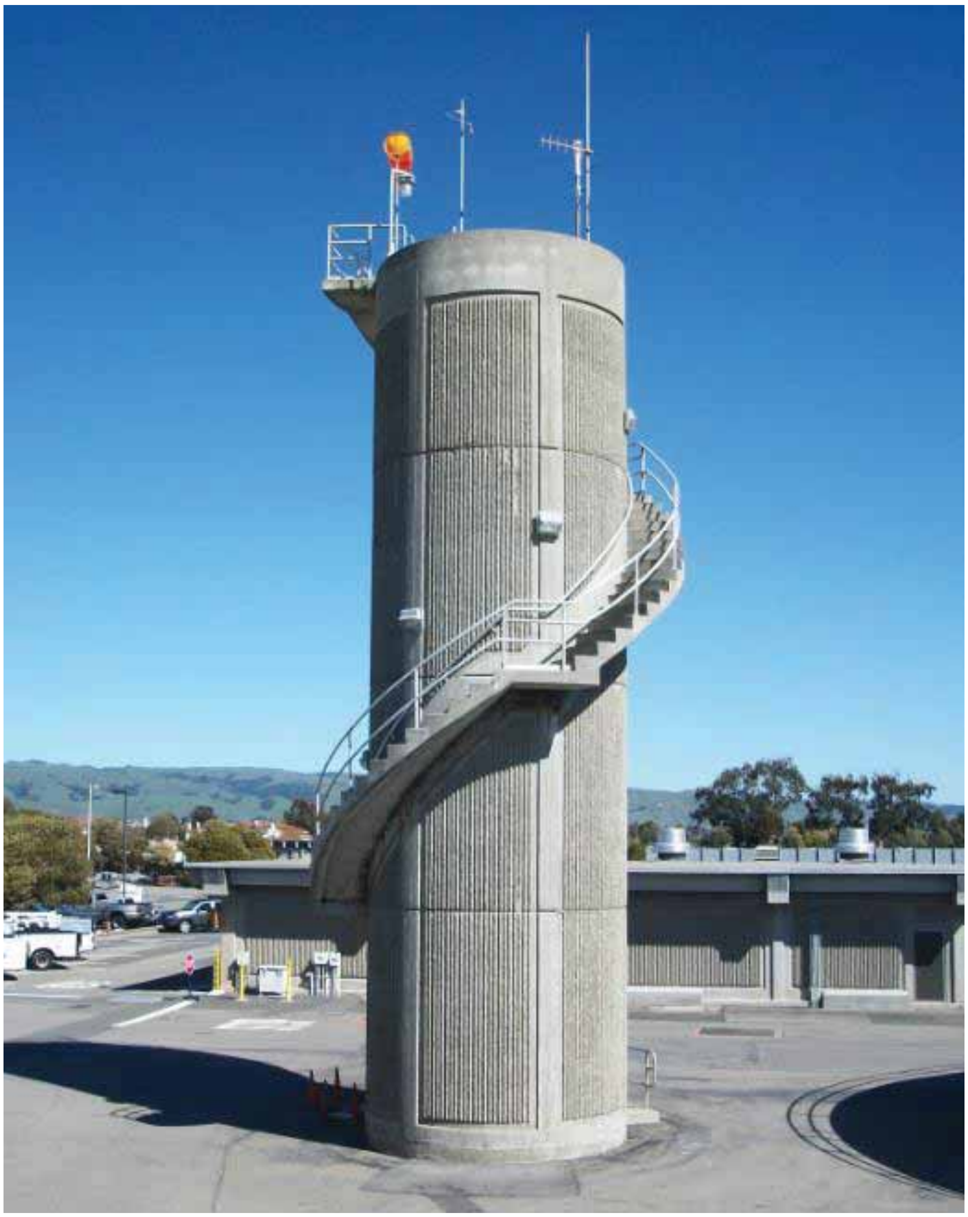

Figure 33: Exterior View of EBDA Effluent Surge Tower

It is our understanding that the height of the fluid in the tower varies significantly. Consequently, we have conducted preliminary analyses considering the fluid height at a full level (near the top platform) and at half level. For a given seismic event, the performance of the tower will be largely dependent on the level of the fluid within the tank, and how the surrounding soil at the base of the tank responds to the movement of the tank. Note that for all of our preliminary analyses, the potential beneficial effect of the surrounding soil has been neglected. This has been done because building codes require geotechnical engineering input on a structure by structure basis to assess potential seismic soil pressures. For many of the structures at the site (mostly partially embedded tank structures like the primary clarifiers), it is unclear to us whether or not the movement near the base of the structures will be significant enough for seismic induced passive soil pressures to develop. For the EBDA Surge Tower, however, an appreciable amount of movement is expected and it is likely that the soil will have a beneficial effect on the seismic performance of the tower. Consequently, further analysis and geotechnical input is recommended.

3.3.17.1 Tier-1 Deficiencies

<u>Shear Stress Check:</u>

The level of deficiency depends on water level and specific consultation from a geotechnical engineer.

The shear stress is ~180 psi when water level is at the height of the grating and is ~130 psi when the water level is at half height, based on a simplified analysis neglecting the large opening at the base of the tower. A finite element analysis considering the large (5' diameter) pipe opening near the base of the wall shows the true max shear stress is closer to 300-400 psi, which is highly stressed even when the contribution from the rebar is considered (note the horizontal rebar also has to resist hoop stresses). This analysis also does not consider the potential beneficial effect of the passive pressure of the surrounding soil (~15 feet from the base). Further analysis that incorporates the specific recommendations of a geotechnical engineer is recommended.

Detailed flexural analysis indicates the wall has enough vertical reinforcing to resist the overturning moment.

Based on preliminary analysis hoop stress in concrete walls and freeboard meet life-safety requirements.

<u>Overturning:</u>

Based on preliminary analysis rocking is possible but may be OK if more advanced analysis is conducted. The foundation may have strength issues in a rocking event, as there is 15' of soil above foundation and top reinforcing in the mat foundation is relatively light.

<u>Transfer to Shear Walls/Other Diaphragms:</u>

The beams framing the grating have horizontal slotted holes and the grating is not tight to the concrete walls. Therefore, there is no means of shear transfer in the direction longitudinal to the beams. In the direction transverse to the beams, the beams and their anchorage are OK to serve as diaphragm based on preliminary analysis. Note that the grating is installed in a number of individual pieces.

Because there is no means of shear transfer in the direction parallel to the grating beams, the grating will pound on the concrete walls. The grating is light enough that it will likely not significantly damage the walls, but the individual grating pieces could break apart and fall into the tank. This would only pose a life-safety concern if someone was on the grating during a significant seismic event.

Liquefaction:

There is possible liquefaction induced settlement at the site. Although differential settlement is expected to be small and not expected to pose a serious life-safety hazard, this should be studied in greater detail for the Surge Tower because of its high height-to-footprint ratio.

3.3.17.2 Remediation Strategies

Specific geotechnical input and further analysis is required to determine whether or not strengthening is required so that the Surge Tower meets life-safety performance in the DBE. However, we expect that some work to the foundation may be done (particularly adjacent to the main pipe entry). Consequently, we believe some seismic retrofit work should be budgeted and propose a preliminary budget of $250,000. Although minor, we also recommend that the roof grating be replaced and directly anchored to the concrete walls.

3.3.17.3 Other Structures in Group

The EBDA Surge Tower is the archetypal structure cast-in-place towers. The structure class also includes the Irvington Site Surge Tower and Newark Site Surge Tower, both of which were also constructed in 1978. Based on a brief review of the drawings, it appears that the construction of all the towers is very similar except that the other two towers are 10-15 feet higher (~20%) than the EDBA Surge Tower, but have smaller pipes that enter the base of the tank.

Additionally, the drawing Irvington Surge Tower's foundation had been strengthened in 2001, and the pipe that enters the base of the tower had been encased in concrete (although the complete extent of the retrofit is unclear because some drawings appear missing). This strengthening work indicates that the foundation and pipe entering the base of the structure might be deficient for all the towers.

3.3.18 Thickener #1 – Seismic Performance Rating = 3

Thickener #1 is a circular cast-in-place concrete tank with a cast-in-place concrete roof. An exterior view of Thickener #1 is shown in Figure 34. The tank was constructed as part of the 1978 phase of construction at the site. Relatively small cast-in-place concrete structures like Thickener #1 usually perform well in earthquakes. Our preliminary calculations indicate that the seismic performance of Thickener #1 meets the life-safety requirements in the DBE. Potential deficiencies are relatively minor and are discussed in greater detail below. However, note that from observations on our 3/18/14 site visit, there is substantial cracking in the walls of Thickener #1, particularly at the base of the wall. While the calculations contained in this report indicate that this cracking does not appear to be the result of structural distress, it's something that should be continually monitored.

Figure 34: Exterior View of Thickener #1

In 2004, a project was done to repair the cracking at the Thickeners. The repair to the concrete walls was limited to repairing the concrete and coating the rebar. This is an indication that the rebar was not significantly damaged. Additionally, the nature of the damage, the pattern of the cracking, and corrosion of the concrete indicate that the cracking and damage was likely the result of the corrosive environment of the tank and not the result of structural distress. However, further study as to the condition of the reinforcing at the thickener may be warranted. Furthermore, the preliminary analysis that was conducted as part of this study assumed the tank walls are in good condition. If the concrete continues to spall/corrode and the rebar is found to be corroded, the seismic performance of the tank may be deficient.

3.3.18.1 Tier-1 Deficiencies

Adjacent Structures:

The walkway leading to the thickener appears to be separated by a small caulked joint, but there may be dowels at this location. This should also be investigated further, and the relative movement between the thickener and the thickener control building should be assessed.

Note that the center rake mechanism and associated piping are not within the scope of this study and further analysis may be warranted.

Openings at Shear Walls:

Openings in the roof diaphragm comprise ~35% of wall length. However, based on the shear transfer DCR, this is not likely to be a deficiency with more advanced analysis.

3.3.18.2 Remediation Strategies

The only seismic remediation expected is pending the study of the walkway between the thickener and the thickener control building. If necessary to remediate, the work is expected to be minor. We estimate that a preliminary budget of $25,000 should be carried for repair work and possible work to the walkway.

3.3.18.3 Other Structures in Group

Thickener #1 is the archetypal structure for cast-in-place tanks with cast-in-place roof. The structure class also includes Thickeners #2 through #4. Thickener #2 was built in 1978 and is structurally the same as Thickener #1. Thickeners #3 and #4 were built in 1985, and based on a brief review of the drawings appear to be almost identical in size, wall thickness and reinforcing to Thickeners #1 and #2. Consequently, we believe it is reasonable to extrapolate the findings from Thickener #1 to the other thickeners.

3.3.19 Lift Station #1 – Seismic Performance Rating = 3

Lift station 1 is a cast-in-place, partially embedded concrete structure constructed during the 1985 phase on construction. An exterior view of Lift Station #1 is shown in Figure 35. The lift station walls are thick and are well reinforced. The structure supports very heavy steel pumps, which pump fluid from a channel on the west side of the structure to a channel on the east side of the structure.

Figure 35: Exterior View of Lift Station #1

In seismic events, cast-in-place concrete structures with high amounts of wall like the lift station tend to perform very well. The lift station would be expected to perform very well in a seismic event, except that there are some potential load path deficiencies regarding how the horizontal diaphragm elements are connected to the concrete walls that require further study. These potential deficiencies are discussed in greater detail below.

3.3.19.1 Tier-1 Deficiencies

Undefined Load Path/Transfer to Shear Walls/Diaphragm Continuity/Openings at Shear Walls:

The lateral load path, particularly for the large mass transfer of the pumps, requires further analysis and an accurate estimation of the mass of the pumps.

The top roof slab must support part of the weight of the pumps, in addition to its own self weight, but is not directly connected along its length to the East-West running walls. There are drag bars to the far South wall at the Aeration Basins (appear to be 2 #5) but otherwise there are no specially defined drag elements in the slab.

It is very possible that East 15" wall can cantilever some of the load to the second floor diaphragm, but this should be confirmed with additional analysis. Even so, this is a much more flexible load path than loading the cross wall(s) directly and could lead to localized diaphragm damage at the cross walls between the pumps.

Also, the short roof diaphragms at the top East level and intermediate west level must cantilever for North-South seismic action. This is likely not a deficiency with further analysis, but needs further study because of the large mass of the pumps and because the cantilever moment reactions must be resisted by small amount of reinforcing/continuity into the cross walls.

The location of the main diaphragms and load resisting concrete walls are shown in Figure 36 below.

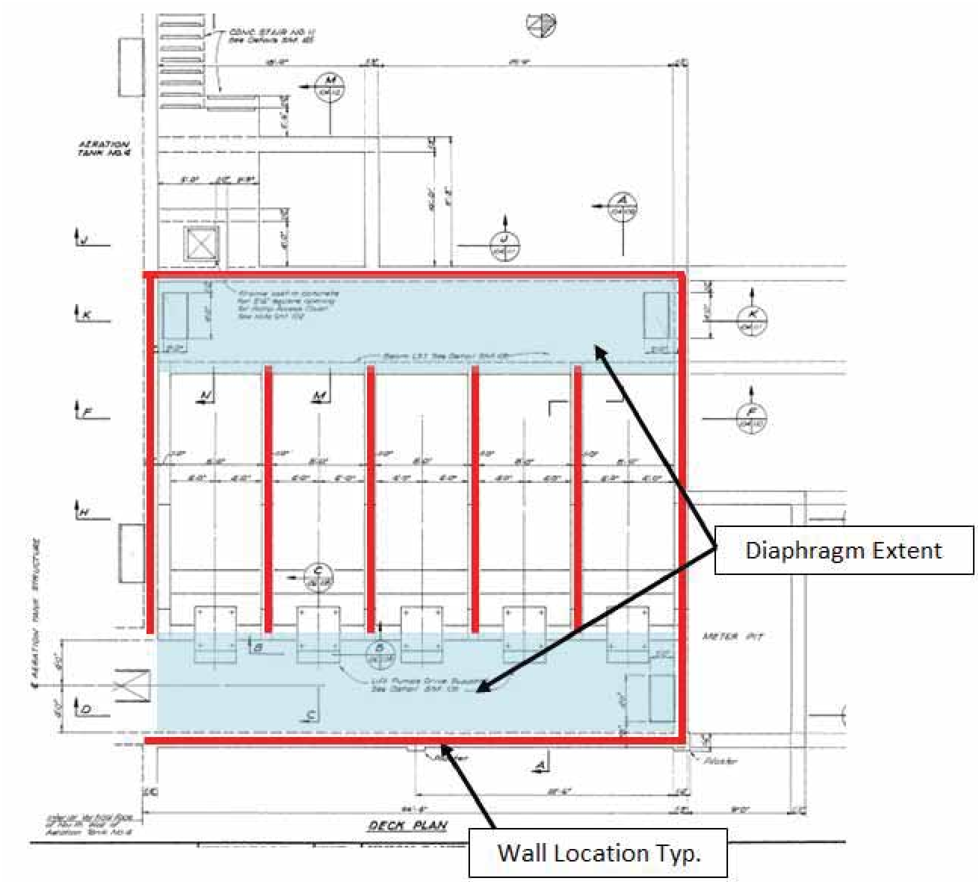

Figure 36: Lift Station #1 Concrete Wall and Concrete Diaphragm Locations

Deflection Compatibility of Secondary Elements:

The columns supporting the East portion of the structure have the shear strength to develop the flexural strength of the column based on simplified Tier 1 analysis assuming the columns have no gravity loads but further analysis is recommended. With further analysis that considers the actual gravity load in the columns, the columns may not be able to develop a flexural hinge before failing in shear. However this would still likely not be a life-safety issue, as the columns should be protected from excessive drift induced demands because the Lift Station is a stiff embedded structure.

Vertical Irregularities of Center Fin Walls:

Force that is taken by the middle fin walls may have to go through the diaphragm to the end walls. This is likely not a deficiency with further analysis.

3.3.19.2 Remediation Strategy

None expected pending further analysis.

3.3.19.3 Other Structures in Group

Lift Station #1 is the archetypal structure for cast-in-place concrete structures constructed in the mid 1980's to present day. The structure class also includes Irvington Equalization Storage Tank, Aeration Basins 5-7, Lift Station #2, West Aeration Blower Room, Control Box #1, 2 & #4, Headworks Building, and Centrifuge Building. Modern (1980's and later) cast-in-place concrete structures generally perform well in earthquake events except when not enough concrete wall has been provided or when there is a significant irregularity or discontinuity associated with the building.

Based on a brief review of the drawings, all of the aforementioned structures appear to have a significant amount of thick reinforced concrete walls. Furthermore, our brief review of the drawings did not reveal any major irregularities or discontinuities. Consequently, we would not expect that a significant of seismic retrofit works needs to be done to these structures for them to meet life-safety performance. Still, further study, particularly of the more complex structures like the Headworks and Centrifuge buildings, may be warranted. For example, the Headworks Building has a number of large openings on the second floor diaphragm and the Centrifuge Building has a very large steel mezzanine structure that should be analyzed further. Without further analysis, we recommend that a seismic repair budget of $50 per square foot be carried for the structures in this group that have not been screened.

3.3.20 Heat Mix Building #2 – Seismic Performance Rating = 2

Heat Mix Building #2 is a small cast-in-place structure that was constructed in the 1960's. An exterior view of Heat Mix Building #2 is shown in Figure 37. During the 1978 phase of construction, concrete infill panels were added on the East and West open portions of the structure. The 1978 drawings indicate that these concrete panels were doweled into the spandrel beam above and the concrete foundation below, which would indicate that the panels were cast-in-place. However, the existence of dowels should be verified with testing in the field. A plan showing the locations of the cast-in-place concrete walls, and infilled precast concrete panels is shown in Figure 38. Furthermore, despite the fact that the detailing is non-ductile as typical of 1960's vintage concrete buildings, we expect the heat mix building to perform well from a life-safety perspective in the DBE because of its small size.

Figure 37: Exterior View of Heat Mix Building #2

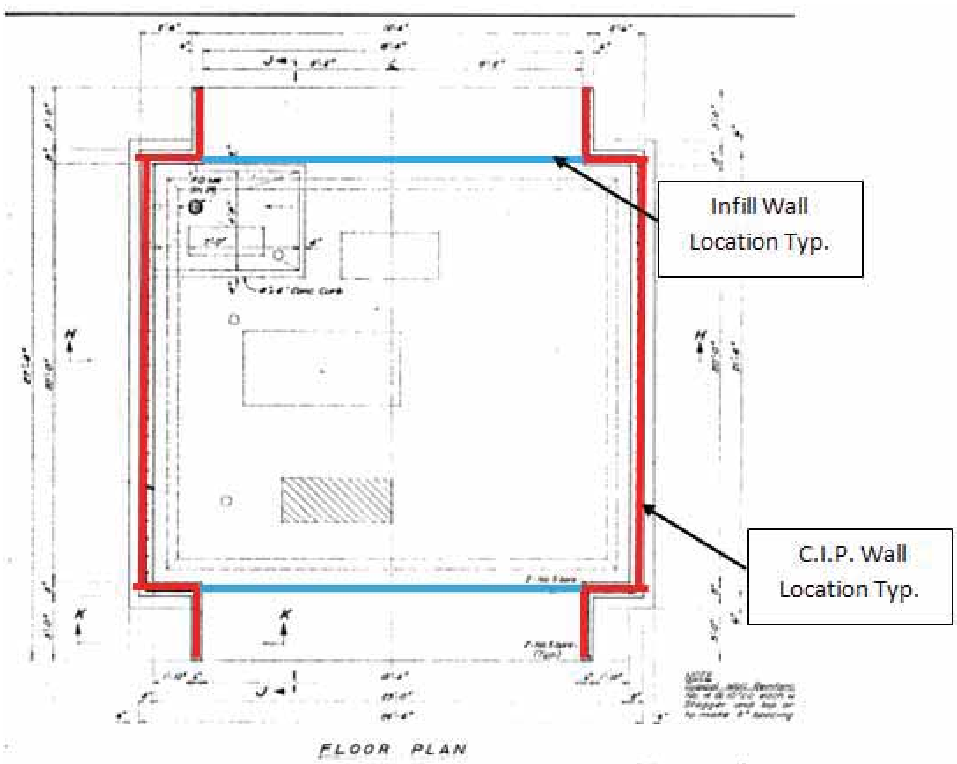

Figure 38: Heat Mix Building #2 Wall Layout

As noted below, the only potential ASCE 41-13 identified deficiency that poses a significant life-safety concern (other than typical building deficiency of piping attachment) is related to the walkway that connects the Heat Mix Building #2 to Primary Digester #3. This deficiency is really a non-structural issue and would only pose a life-safety concern if someone was on or below the walkway.

3.3.20.1 Tier-1 Deficiencies

Elevated Walkway:

There are no apparent seismic joints on the walkway to Heat Mix Building #2 (drawings indicate it is bolted to both structures). This could be a concern if there is differential movement between the Heat Mix Building and Primary Digester #3. However, both structures are stiff concrete structures where the seismic induced displacements should be small, so further analysis is recommended.

3.3.20.2 Remediation Strategies

The only seismic remediation expected is pending the study of the walkway between the heat mix building and the digester. If necessary to remediate, the work is expected to be minor.

3.3.20.3 Other Structures in Group

The Heat Mix Building #2 is the only structure assigned cast-in-place concrete structures constructed in 1962. Consequently, no extrapolation is required.

3.3.21 Chlorine Contact Tank – Seismic Performance Rating = 2

The Chlorine Contact Tank is a series of open-air narrow channels formed by robust cast-in-place, mostly below grade concrete walls. A view of the contact tank is shown in Figure 39. The tank was constructed during the 1978 phase of construction at the site, and sits adjacent to Secondary Clarifiers 1-4 and the Effluent Pump Station. The main potential life-safety deficiency, as discussed below, does not relate to the Chlorine Contact Tank but rather to the adjacent Effluent Pump Station.

Figure 39: Exterior View of Chlorine Contact Tank

The walls of the Chlorine Contact Tank have been preliminarily analyzed for the incremental fluid and inertial forces from the DBE. The thick exterior walls are sufficient to resist the incremental pressure, and the thinner interior walls are sufficient to resist the forces but are highly stressed compared to the exterior walls. Consequently it is possible that a flexural hinge could form at the base of an interior wall during a significant seismic event (some ground motion larger than what is produced by the DBE), causing the wall to partially collapse.

3.3.21.1 Tier-1 Deficiencies

Adjacent Structures:

The Chlorine Contact Tank is physically connected to EBDA pump station and secondary clarifiers. With more advanced analysis this is likely not a deficiency, as all 3 structures are stiff structures.

Note that EBDA pump station roof is a precast roof which as originally constructed was very likely deficient and had the possibility of collapsing and damaging the Chlorine Contact Tank. This roof has been retrofit after the original construction so this risk has likely been mitigated. However, specific study of the EBDA pump station and the retrofit work is beyond the scope of this report, and further study is necessary to confirm this potential hazard.

3.3.21.2 Remediation Strategy

None expected.

Other Structures in Group:

The Chlorine Contact Tank is the archetypal structure for cast-in-place concrete structures built during the 1978 phase of construction. The structure class also includes the Thickener Control Building, Heat Mix Building #1, the Transfer Tank, the Gas Compressor Room and the WAS Thickener Building. As a tank structure, the Chlorine Contact Tank is really quite different structurally from the other 5 structures in building class.

During the late 1970's typical cast-in-place concrete construction did not use what is generally accepted as "modern" standards of seismic detailing. This is of particular concern for structures with relatively small amounts of wall and poor rebar detailing at the beams and the columns. In these types of structure, excessive deformations can lead to damage in the beams and columns, which can lead to collapse. Even older cast-in-place concrete structures, however, generally perform well in earthquake events enough concrete wall has been provided to limit deformations. Based on a brief review of the drawings, it appears that enough wall is provided in all of these other cast-in-place 1978 structures that there is not a major lack of wall related deficiency. Still, given the differences in the structures, it is not reasonable extrapolate the lack of deficiencies in the Chlorine Contact Tank to the others in the group. It is recommended that the WAS Thickener Building be studied, as it appears to be the most complex in the group, and it has very tall and thin concrete walls that may need out-of-plane strengthening. Pending further study of these structures, we recommend that a preliminary seismic retrofit budget of $100 per square foot be carried for these structures.

3.3.22 Secondary Clarifiers 1-4 Seismic Performance Rating = 2

The Secondary Clarifiers 1-4 are a group of four open-air tanks formed by robust cast-in-place, below grade concrete walls. The clarifiers were constructed during the 1978 phase of construction at the site, and sit adjacent to the Chlorine Contact Tank, the Effluent Pump Station, Sludge Pump Room #2 and Control Box #3. The main potential life-safety deficiency, as discussed below, does not relate to the clarifiers but rather to the adjacent structures. The Secondary Clarifiers 1-4 are shown in Figure 40. Note that one of the 4 cells is usually empty at any given time.

Figure 40: Secondary Clarifier 1-4, Empty Cell

The walls of the Secondary Clarifiers 1-4 have been preliminarily analyzed for the incremental fluid and inertial forces from the DBE. The thick and well reinforced walls are sufficient to resist the incremental pressure based on preliminary analysis. The concrete wall locations of Secondary Clarifiers 1-4 are shown in Figure 41.

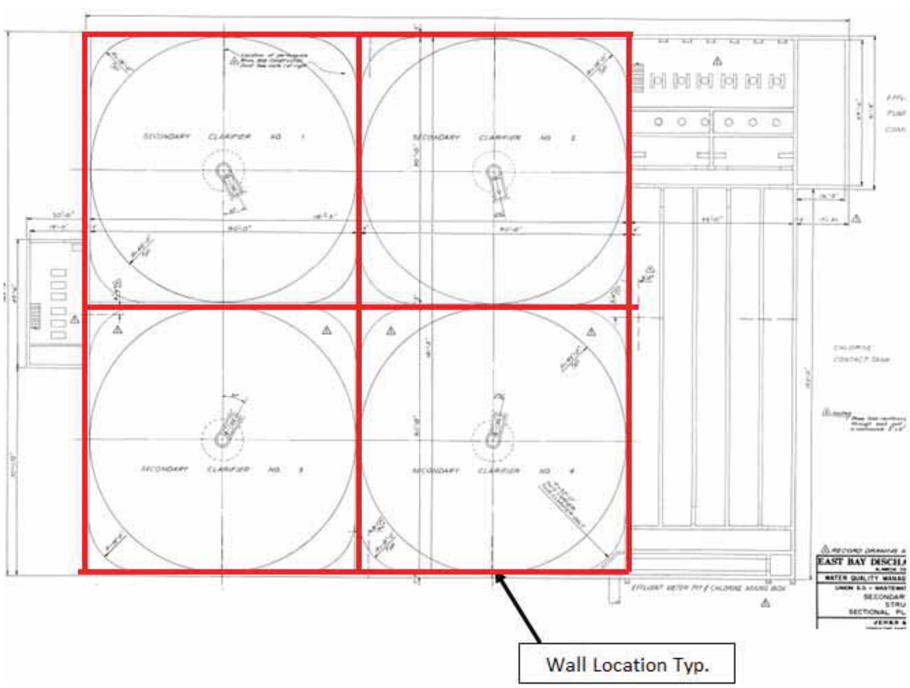

Figure 41: Secondary Clarifiers 1-4 Concrete Wall Locations

3.3.22.1 Tier-1 Deficiencies

Adjacent Structures:

The Secondary Clarifiers 1-4 are physically connected to EBDA pump station and Chlorine Contact Tank. With more advanced analysis this is likely not a deficiency, as all 5 structures are stiff structures.

The roofs of Sludge Pump Room #2 and Control Box #3 are precast concrete roofs which are almost certainly deficient. They represent collapse hazards that could fall off their walls and onto the Secondary Clarifiers.

The EBDA pump station roof is a precast roof which as originally constructed was very likely deficient and had the possibility of collapsing and damaging the clarifiers. This roof has been retrofit after the original construction so this risk has likely been mitigated. However, specific study of the EBDA pump station and the retrofit work is beyond the scope of this report, and further study is necessary to confirm this potential hazard.

Furthermore, the center stirring mechanisms and non-structural access walkways were not specifically considered as part of this study, and further analysis of these items may be warranted.

3.3.22.2 Remediation Strategy

None expected for Secondary Clarifiers 1-4. Retrofit of Sludge Pump Room #2 and Control Box #3 expected. Retrofit of EBDA Pump Station pending further analysis.

3.3.22.3 Other Structures in Group

The Chlorine Contact Tank is the archetypal structure for open-top cast-in-place concrete structure. The structure class also includes Secondary Clarifiers 5-6. Based on a brief review of the drawings, Secondary Clarifiers 5-6 have slightly taller and thicker walls than Secondary Clarifiers 1-4. However, the vertical reinforcing in the clarifier 5-6 walls is substantially less than Secondary Clarifiers 1-4 (approximately ½ the vertical reinforcement). Consequently, it is possible that the walls are will be slightly overstressed in the DBE, and further analysis is recommended. Without studying the Secondary Clarifiers in greater detail we recommend that a preliminary seismic retrofit budget of $100 per square foot be carried for this structure.

3.3.23 Control Box #3 – Seismic Performance Rating = 2

Control Box #3 is a small cast-in-place concrete structure that is predominately below grade. The structure was originally constructed during the 1978 phase of construction at the site. In its original construction, Control Box #3 had an additional floor above grade that was supported by precast concrete walls and a precast concrete roof. These walls and roof were demolished in 1992 when the Rotating Biological Contactor Building was demolished and replaced by Aeration Basins 5-7. Currently, Control Box #3 consists of an elevated slab slightly above grade supported by reinforced concrete walls below grade. The location of the below grade cast-in-place concrete walls are shown in Figure 42. The cells between the cross walls are currently partially filled with fluid. An exterior view of Control Box #3 is shown in Figure 43.

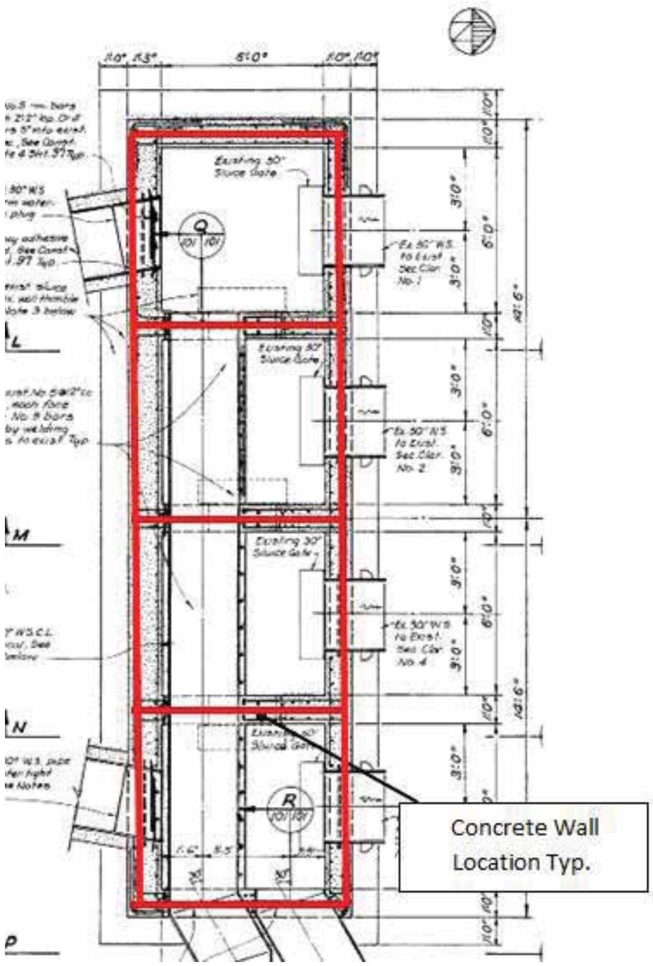

Figure 42: Control Box #3 Wall Locations

Figure 43: Exterior View of Control Box #3

The concrete walls are thick and well reinforced. The horizontal span between walls is small which helps span the soil and fluid pressures to the cross walls. Given the robustness of the construction, Control Box #3 is expected to perform well in a seismic event. The ASCE 41-13 identified potential deficiencies are relatively minor, and are discussed in greater detail below.

Note that our analysis of Control Box #3 assumed that the structure was in good condition. We were not able to access the condition of the below grade walls because they are obscured by fluid. From grade level, the concrete walls appeared to be in good condition, but we recommend that a condition assessment be completed as a follow-up to this study.

3.3.23.1 Tier-1 Deficiencies

Torsion:

The series of cross walls are interrupted by a series of large diameter pipes that run in the long direction of the structure. Consequently, there are large openings in these walls, with the furthest east wall being completely interrupted at the base by pipes. This is not expected to be a life-safety concern with additional analysis because the structure is nearly completely embedded.

Openings at Shear Walls:

There are a series of openings on the ground floor diaphragm. Given the inertial loads in the diaphragm, this shouldn't be a life-safety issue for seismic load transfer with additional analysis.

3.3.23.2 Remediation Strategy

None expected.

3.3.23.3 Other Structures in Group

Subsequent to the demolition of the precast concrete walls and roof, Control Box #3 is part of the below grade concrete structure group. Please see the Draft Report for further information regarding this structure group.

3.3.24 Alvarado WWTP Force Main Influent Valve Vault – Seismic Performance Rating = 1

The Alvarado Vault is a small cast-in-place concrete structure that is completely below grade. The vault's drawings are dated 1977, one year prior to the major 1978 construction phase at the site. The concrete walls are very thick and well reinforced. Furthermore, the walls have horizontal beam elements that help span the soil pressures to the cross walls. The horizontal beams are shown in Figure 44. Given the robustness of the construction, the Alvarado Vault is expected to perform well in a seismic event. The ASCE 41-13 identified potential deficiencies are relatively minor, and are discussed in greater detail below.

Note that our analysis of the Alvarado Vault assumed that the structure was in good condition. We were not able to access the inside of the vault because of safety concerns and therefore do not know what structural condition the vault is in. From grade level, the concrete walls appeared to be in good condition, but we recommend that a condition assessment be completed as a follow-up to this study.

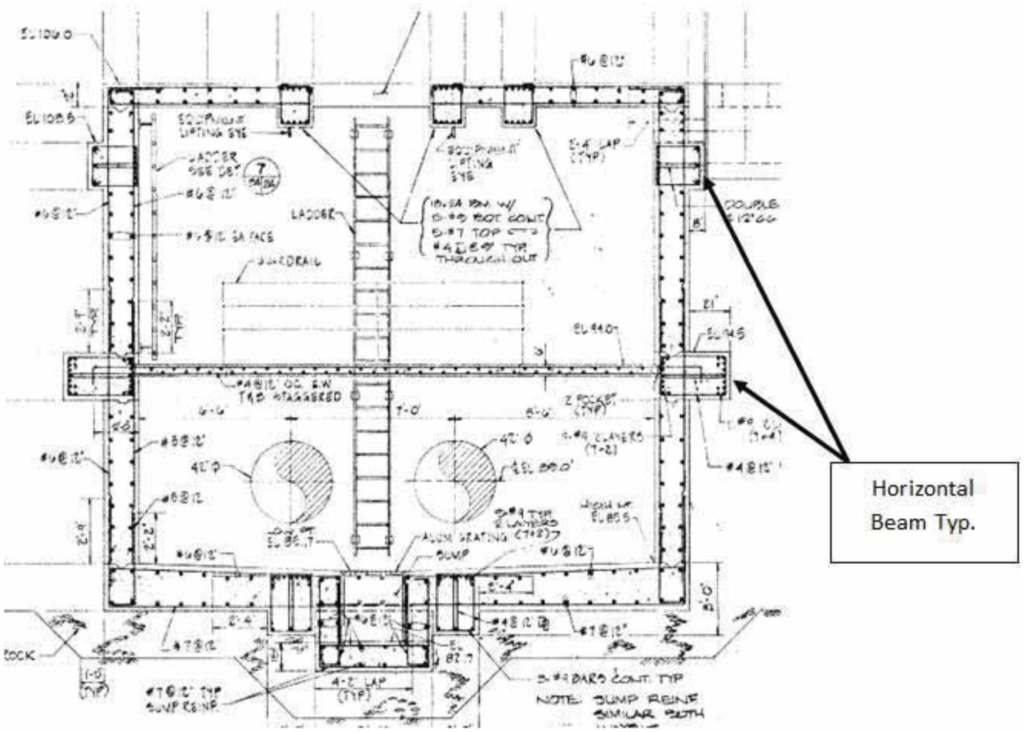

Figure 44: Alvarado Influent Valve Vault Elevation

3.3.24.1 Tier-1 Deficiencies

Adjacent Structures:

A small vent structure is located 1" away from the vault near the ground surface. While this is not of particular concern for the larger vault structure, the seismic surcharge load from the vault on the vent structure may affect and damage the vent structure. This should be studied in greater detail.

Openings at Shear Walls:

The access openings at the ground level are slightly greater than 25% of the wall length. This deficiency is very not likely a life-safety concern with additional analysis.

3.3.24.2 Remediation Strategy

None expected.

3.3.24.3 Other Structures in Group

The Alvarado Influent Valve Vault is the archetypal structure for below grade concrete structures. The structure class also includes the EBDA Effluent Valve Vault, the Emergency Outfall Control Valve Structure, the Site Waste Pump Station, the Alvarado Influent Pump Station Flow Meter Pit, the Cherry Street Pump Station, Control Box #3, and the Newark and Irvington site valve vaults. From a brief review of the drawings, all of these structures appear to be small and robustly constructed. Because the structures are underground, the main potential seismic hazard is from incremental seismic induced soil pressures. It is recommended that a condition assessment be conducted for each of these structures to look for signs of structural distress (cracking, exposed reinforcing etc.). If no signs of distress are found, it can be expected that these small structures will perform relatively well in a seismic event.

3.3.25 Main Electrical Distribution/Generator Room 3 – Seismic Performance Rating = 1

The Main Electrical Distribution Building is a single high-bay cast-in-place concrete shear wall building with a lightweight steel framed roof structure. The structure was constructed in the early 1990's and is shown in Figure 45. Typically buildings like Main Electrical Distribution have sufficient walls to resist seismic forces, but have deficient wall-to-roof anchorage for out-of-plane wall forces. See Figure 46, which highlights the locations of the concrete walls. This is particularly true of high-bay wall structures, where wall-to-roof anchors must resist the forces from a tall section of wall. The Main Electrical Distribution Building, however, is robustly tied into the steel roof members, which are capable of developing the out-of-plane wall forces into the diaphragm. During the analysis of this structure, no structural deficiencies were identified that pose a significant life-safety concern.

Figure 45: Exterior View of Main Electrical Distribution Building

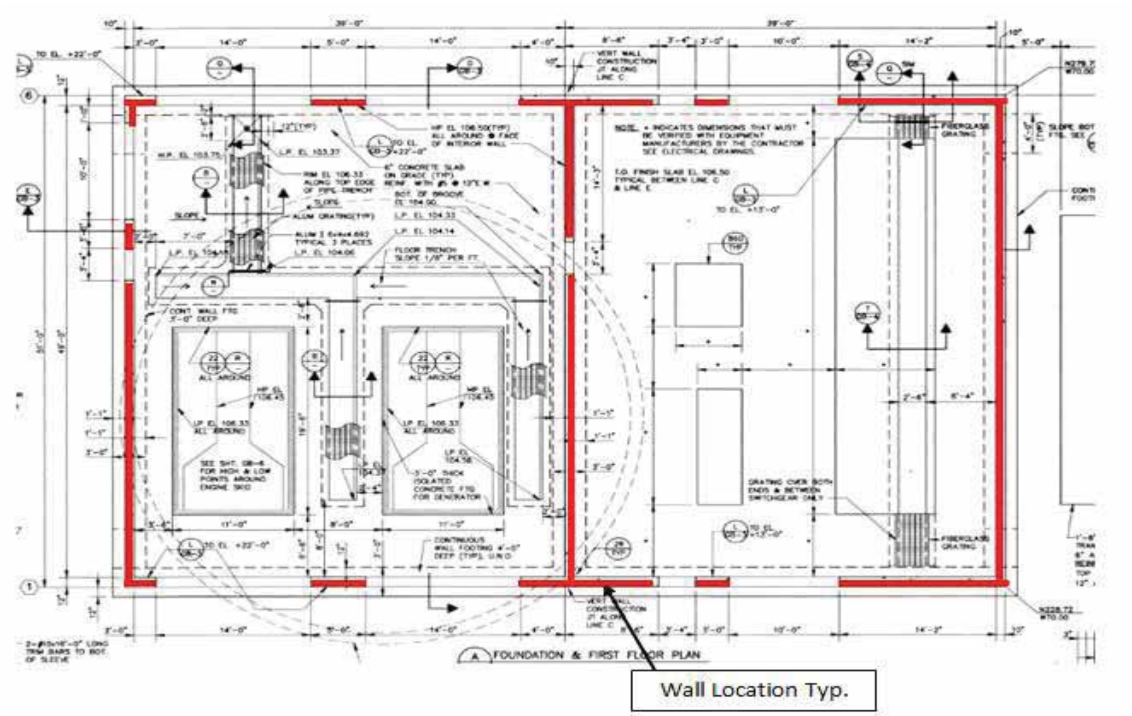

Figure 46: Main Electrical Distribution Building Concrete Wall Locations

3.3.25.1 Remediation Strategy

None expected.

3.3.25.2 Other Structures in Group

The Main Electrical Distribution Building/Generator Room #3 is the archetypal structure for post 1990 cast-in-place concrete or reinforced masonry wall structures with steel framed roofs with bare metal decking. The structure class also includes Hayward Marsh, The INCA MCC Building, the Paint Shop and Paint Booth, and Heat Mix Building #4. Typically we would expect these structures to perform well in earthquakes provided that sufficient out-of-plane wall anchorage is provided at the roof. From a brief review of the drawings (note some drawings appear to be missing), it appears that some out-of-plane anchorage mechanism is provided at all of the structures. However, the anchorage generally does not appear as robust as the Main Electrical Distribution Building, and may need to be supplemented in some of the structures. Further analysis of these other structures is recommended. Fortunately, if the structures' anchorage is found to be deficient, the cost of the retrofit work can be expected to be relatively minor. Pending further study of these structures, we recommend that a preliminary seismic retrofit budget of $50 per square foot be carried for these structures.

4.0: Pipeline Assessment

4.1 Onsite Buried Piping Seismic Assessment

Plant site piping associated with the WWTP's gravity liquid flow train are expected to perform well. Pressure site piping that is shallow and sludge piping that connects facilities towards the western side of the plant site are subject to differential settlement and pipe failure.

Based on data provided by the District, we understand that the gravity liquid flow train piping is all welded steel pipe. In review of the drawings for the original 1978 plant construction, and expansions in 1985 and 1992, double flex joints with bolt harnesses are shown at the interface between structures and buried pipe (See Figure 47). This typical design allows rotation in two locations so as to accommodate differential settlement. The combination of welded steel pipe with double flexible joints should accommodate expected differential settlement without causing pipe failure. While double flexible joints were identified on selected gravity flow train piping, all pipe-structure interfaces were not reviewed, so there is potential that some pipelines do not have the double flexible joints.

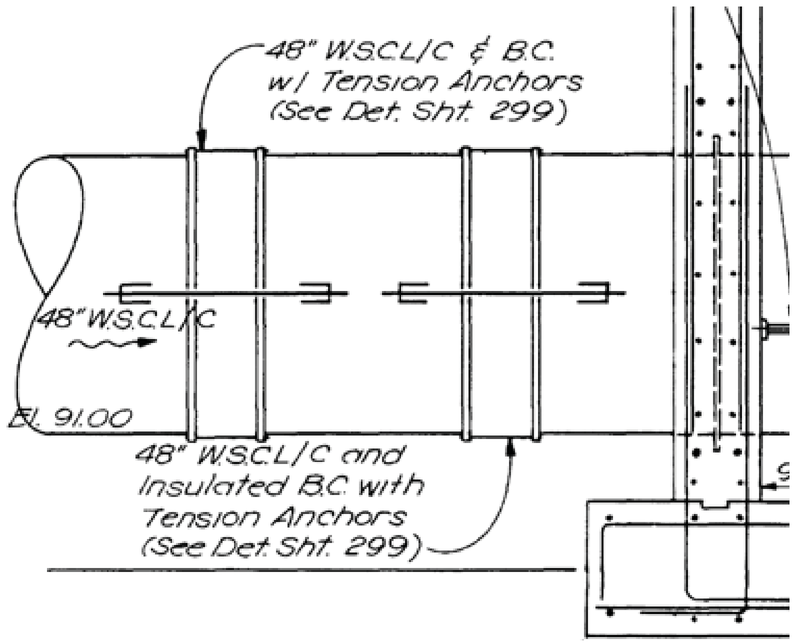

Figure 47: Typical double flexible joint with bolt harnesses. Detail on Meter Pit No. 4.

In addition, it is our understanding that for the original plant and subsequent additions, and specifically for structures and tanks that are interconnected by the plant's gravity liquid flow train, for deep structures extending down to competent soils and for shallow structures that were overexcavated down to competent soils to remove soft soils (e.g. Bay Mud), the structure excavations and overexcavations extended several feet beyond the structure footprints and were mostly side sloped at about 1H:1V or flatter. As a result, inlet/outlet piping at the structure interface is infill (mostly well compacted). That should help mitigate any strong differential settlement at the structure interfaces. However, we don't know of any available records relative to the plant piping bedding and foundation soils conditions.

For those structures on piles it is safe to assume that the Bay Mud was not removed from beneath nor around the structure, so pipelines here are likely underlain by a thin layer of soft Bay Mud. For example during construction of an outfall pipeline (possibly an emergency outfall) about 5 to 10 years ago at the northwest corner of the plant some Bay Mud was encountered. Within the past year, work around the new Cogeneration Building on the west side of the plant has also encountered Bay Mud. In general, the remaining Bay Mud, where present, seems to be mostly on the west side of the plant.

Other structures on piles include: (1) the Dewatering building at the southwest corner of the plant site (the Centrifuge Building No. 81) and (2) the sludge thickener tanks 3 and 4 and the south end of the thickener control building (Building No. 62). At the time of the sludge thickener tanks 3 and 4 construction in about 1985 the Bay Mud could not be removed due to conflicts with nearby existing structures. For these and any other structures that are on piles the differential settlement between structure and inlet/outlet piping could be of concern. We recommend that a detailed assessment of all plant piping be conducted to identify locations where flexible joints were not used, and to identify the detailing of the pipes at the interface of pipe-supported structures.

Shallow pressure piping (and conduits), typically smaller in diameter, could be subjected to differential settlement at the pipe-building interface. Steel or ductile iron pipe subjected to differential settlement are unlikely to fail, but PVC pipe is less ductile and is more vulnerable.

Based on our preliminary screening of onsite buried piping, we believe the overall seismic performance rating of onsite buried piping is 6. Further study of the piping should be conducted and localized mitigation efforts where flexible joints and coupling were not used should be considered.

4.2 Force Main Seismic Assessment

4.2.1 Overview of Force Mains

The District owns twin 33-inch diameter force mains that convey raw sewage between the Irvington Pump Station and the Newark Pump Station, and twin 39-inch diameter force that convey raw sewage from the Newark Pump station to the Alvarado Wastewater Treatment Plant (see Figure 48). The Irvington to Newark force mains are 6-foot-6-inches on center, and each 40,513 feet in length, for a total of 81,026 feet of pipe for both mains. The Newark to Alvarado force mains are 7-feet on-center, and each 26,171 feet in length, for a total of 52,342 feet of pipe for both mains. The force mains generally have four to five feet of cover, but in any given location, the depth may be as great as 15 to 20 feet deep. There is a 2-inch diameter chemical transport line located at the spring line between the two 33-inch force mains, and a 3-inch chemical transport line located at the spring line between the 39-inch force mains.

The force mains, built in the late 1970s, are constructed with 12-foot segments of reinforced concrete pipe with bell and spigot gasketed joints (Figure 49). This joint should allow extension on the order of 1-inch before it starts to leak. The joint detail shows a gap of ¼-inch to ¾-inch between the end of the spigot end of the pipe, and the back of the bell. It is likely that this gap has filled in with debris, so limited compression capability is expected across the joint. Special joint designs are used where required for special situations. In some cases, these special joints may be welded, not allowing movement between the pipe segments. Flexible couples are used in locations where differential movement was expected by the designer.

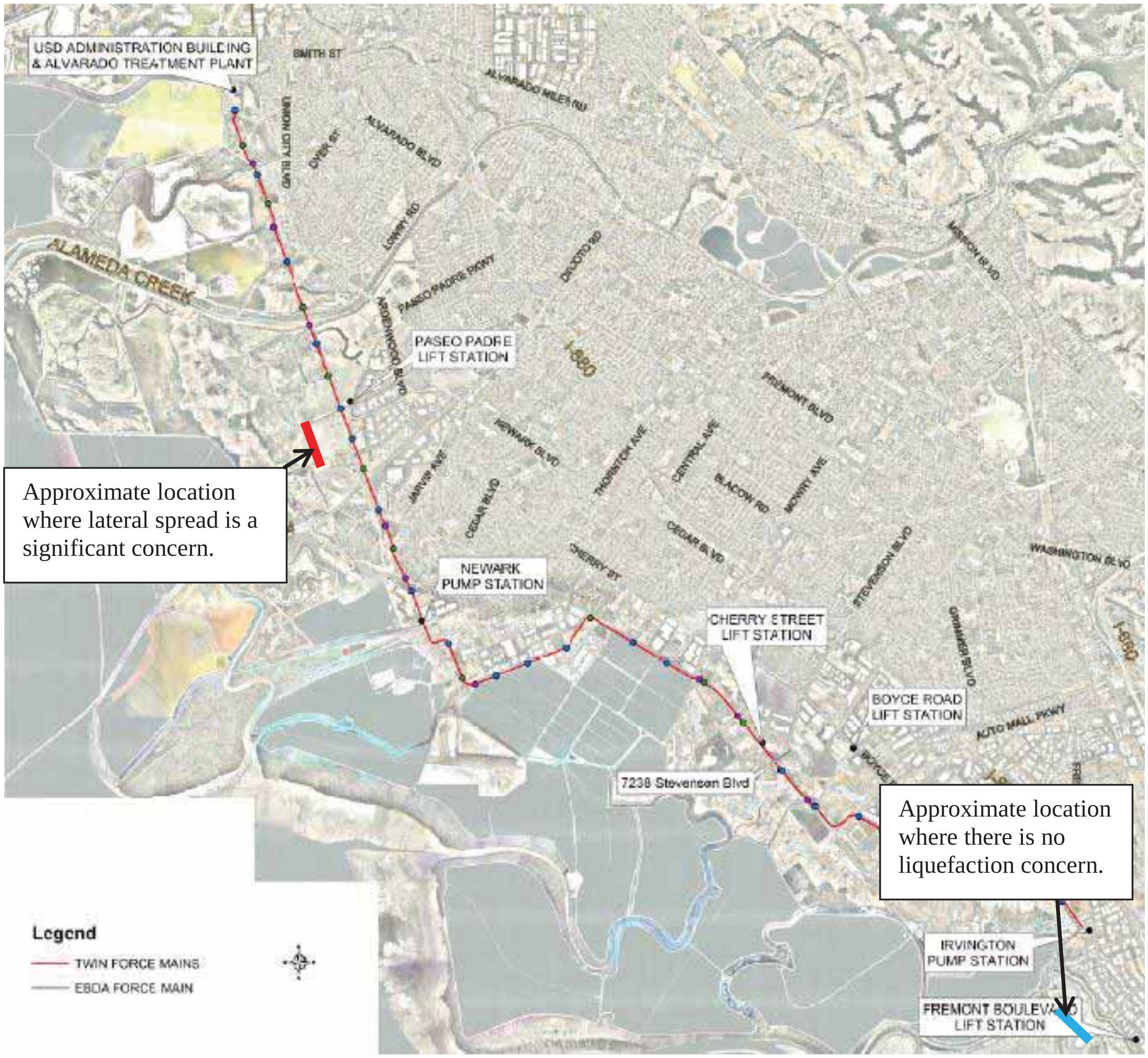

Figure 48: Union Sanitary District Facilities

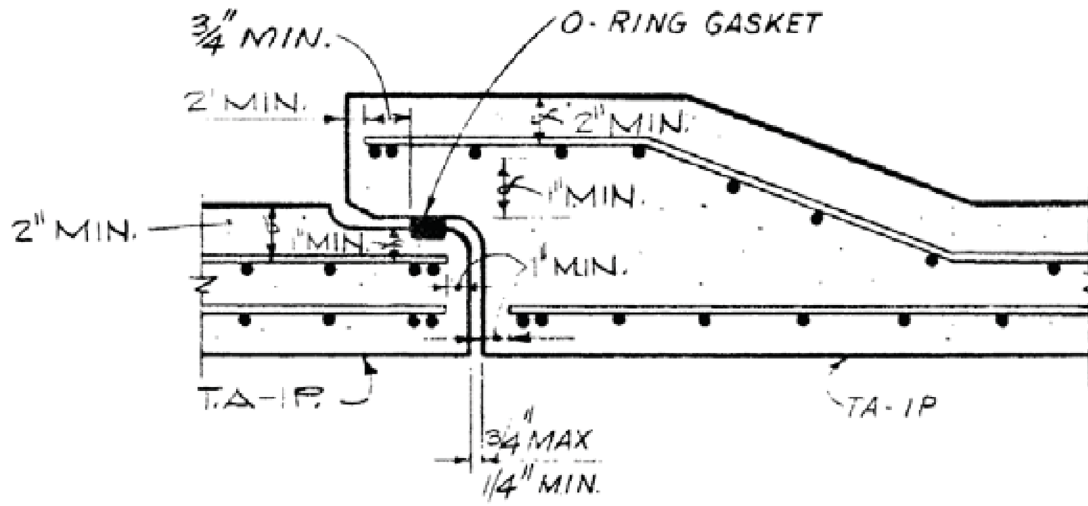

Figure 49: Typical reinforced concrete pipe bell and spigot gasketed joint

The force mains are equipped with access manholes and air release valves at selected intervals. The force mains are installed in steel conductor pipes where they may be subject to external damage, or where force main failure may result in damage to the area above the pipe. The annular space between the force mains and conductor pipe is filled with sand. The force mains pass under various drainage ditches and creeks, and typically are protected from above with a concrete slab.

Historic Earthquake Performance of Buried Pipelines:

Buried pipelines have not performed well in past earthquakes, particularly when they are installed in unstable soil. As these are force mains, pressurized pipelines, they would be expected to perform similar to water mains. In the 2011 Christchurch Earthquake in New Zealand, there were 1,645 water main pipe failures. Water service was restored to areas where houses were intact in just over 40 days. In the 1995 Kobe Earthquake, there were on the order of 1,200 pipeline failures within the City. Portions of the system were without water for as long as 60 days. Many of the pipeline failures in both Christchurch and Kobe were caused by liquefaction and lateral spreading. In the 1994 Northridge Earthquake in Los Angeles, the Los Angeles Department of Water and Power's system suffered on the order of 1,000 pipeline failures causing portions of the system to be without service for almost two weeks.

Gravity sewer mains, not included in this study, perform differently than pressure pipelines. They continue to operate unless the pipe collapses or pulls apart. As they are typically running only partially full, they are buoyant, and will float when the soils around them liquefy. If they move vertically, they may have high points that will cause sewer backups. In the Northridge Earthquake, there were only ten locations where sewer pump-a-rounds were required, compared to 1,000 water main failures.

Earthquake Hazards:

This section summarizes the earthquake hazards relevant to the force mains. Refer to the Technical Memorandum by DCM Consulting, Inc., dated June 17, 2014 for a detailed description of the soils in the force main alignment.

The force mains can be damaged by wave propagation (measured as peak ground velocity, PGV) and permanent ground deformation (PGD).

The earthquake scenario selected for analysis is a moment magnitude 6.33 on the Hayward Fault. It produces PGVs along the force main corridor of 25 cm/se (10 in/sec).

PGD can be caused by surface faulting, liquefaction induced settlement or lateral spread, landslides, and lurching. No surface faulting, landslides, or lurching is anticipated along the force main alignment. The area is generally flat, so no landslides are anticipated. Lurching, the movement of soil blocks due to earthquake shaking that do not liquefy, is not anticipated to any significant degree at the depth of or below the force mains.

Loose poorly graded sands below the groundwater table are highly liquefiable. Fills and bay mud overlie basin deposits, alluvial fan and fluvial deposits along the 13 mile force main corridor. Fills are typically 3 to 5 feet thick, and can be as much as 10 to 15 feet thick at the WWTP. At the WWTP, bay mud is on the order of 4 to 8 feet thick. Basin deposits 5 to 10 feet thick are firm to stiff clays. Below a depth of 5 to 10 feet, the silty clays are interbedded with silty sands and poorly graded sands that are a few to 10 feet thick, with densities varying from loose to medium dense. The groundwater table is 5 to 10 feet deep along the force main corridor. The loose, poorly graded sands below the groundwater table are highly susceptible to liquefaction.

The extent of force main damage from PGD is a function of the probability of liquefaction occurring to the extent where settlement or lateral spreading will occur. This requires that a large area liquefy, not just localized areas that may be evidenced by sand boils. Considering there are randomly occurring lenses of liquefiable material, it is estimated that only 25 percent of the corridor along the liquefiable area will liquefy (based on parameter provided in HAZUS and professional opinions of Ballantyne and Mathy).

Liquefaction induced PGD is of greatest concern. PGD associated with liquefaction can be in the form of settlement and lateral spread. Settlement due to liquefaction of up to 3 to 5 inches is expected in the area of the Newark Pump Station, and ½ to 3-1/2 inches in the vicinity of the WWTP with an average of 1 to 2 inches. An average of 2-inches of settlement is estimated in the areas that liquefy. The topography is generally flat (with exceptions discussed below), particularly perpendicular to the pipe alignment. As a result minimal lateral spread is expected perpendicular to the pipe.

The force mains cross Alameda Creek at 90 degrees where liquefaction is anticipated and lateral spread could occur with movement of several inches to many feet towards the creek banks, with a length of about 940 feet. In addition, the force main corridor just south of Alameda Creek, from Station 106+10 to Station 124+60 appears to be susceptible to liquefaction and lateral spreading. Combined the total corridor length subject to significant lateral spreading is 1,850 feet, or 3,700 feet of 39-inch pipe.

There are a number of shallow drainages that cross the force main alignment. While the drainages are not as deep as Alameda Creek, there is still some limited potential for lateral spread.

Soils from the Irvington Pump Station and found up to one-half mile (2,640 feet) north which are clayey and stiff to very stiff. They are not liquefiable.

4.3 Force Main Earthquake Damage

Pipelines can be damaged when subjected to earthquake induced transient waves passing through the ground (felt as shaking), and to permanent ground deformation (PGD). Damage mechanisms associated with wave propagation can include joint separation, pounding of joints that can damage the pipe bell and spigot, damage due to differential movement at hard points along such as connections to manholes, and damage due to shearing off of small diameter connections such as house services (i.e. in water distribution systems). The expected differential movement between two pipe segments due to wave propagation is a function of ground strain, which is approximately 0.09 percent for scenario evaluated, or about 1/8th of an inch at each 12-foot pipe segment. However, as some joints may no longer allow movement, this displacement could be concentrated in a few joints (e.g. – one in every 10 joints). O'Rourke et al (2012) discuss the potential of joint displacement being 3 times the nominal displacement where the pipe is connected to hard points along the line. Damage of the pipe barrel due to bending to larger diameter reinforced concrete pipe such as the 33- and 39-inch force mains is expected to be minimal due to its beam strength.

The American Lifelines Alliance (ALA) document, Seismic Fragility Formulations for Water Systems (2001) developed empirically based fragility relationships used to estimate for pipelines subjected to transient wave propagation and PGD. For transient wave propagation:

Repair Rate = K_1 x 0.0087 x PGV

Where:

Repair Rate is in terms of repairs/1,000 feet of pipe

K_1 is a constant selected for the expected performance of various pipe materials

PGV is in terms of inches/second.

The values the ALA document proposes for pipe similar to gasketed reinforced concrete pipe have K_1 values shown in Table 3.

Table 3: K_1 values for PGV fragility determination

Pipe Type	K_1 Value
Concrete with steel cylinder, gasketed joint	0.8
PVC, gasketed joint	0.5
DIP, gasketed joint	0.5
AC, gasketed joint (coupling)	0.5

A K_1 value of 0.5 is proposed for this project based on the similarities of reinforced concrete pipe with the K_1 values proposed for other pipe materials.

The ALA document suggests that larger diameter pipe may have a lower fragility than smaller diameter pipe constructed of the same material probably due to its greater beam strength. For this project, a 33-percent reduction (0.67x) will be applied to the overall damage calculated using ALA methods to account for diameter.

The resulting repair rate for transient wave propagation is:

Repair Rate = K_1 x 0.0087 x PGV

Repair Rate = 0.5 x 0.0087 x 10 x 0.67 = 0.029 failures/1,000 feet of pipe

The transient wave propagation equation is applied to the pipeline not subjected to PGD.

PGD can occur parallel to, or perpendicular (lateral spread or vertical) relative to the pipe. A PGD of one inch or greater in the direction parallel to the pipe can cause the joint to separate allowing discharge of sewage. PGD parallel to the pipe could be expected in areas adjacent to free faces such as in the vicinity of Alameda Creek, where the force mains cross it at a 90-degree angle.

PGD perpendicular to the pipe can be caused by settlement or lateral spread. Because of the relatively flat topography, significant lateral spreading perpendicular to the pipe is not expected.

When subjected to settlement, the pipe can fail in shear (with abrupt changes in differential settlement) or can be subject to bending when there is a slow change in the degree of settlement. Near the Newark Pump Station site where maximum settlement is estimated to be 3-5 inches, it is estimated to have a rate of change of settlement of 1-1/2 to 3 inches over 50 feet. If the pipe was running horizontal (e.g. 0 percent slope), and a joint rotated to accommodate 3-inches of settlement 50 feet away, the joint would only have to open 0.2 inches at the top. That is within the limits of the joint's rotational flexibility.

The fragilities developed in the ALA (2001) document were based on empirical data, and specifically for PGD related failures, based on PGDs moving at random orientations to the pipe. As per the earlier discussion about the ability of the pipe to accommodate small PGDs perpendicular to the pipe, for this project, we will assume 20 percent of the PGD estimates the ALA has calculated for PGD damage due to settlement and lateral spread perpendicular to the pipe.

For PGD, the repair rate is calculated as follows (ALA 2001):

$$\text{Repair Rate} = K_2 \times 1.06 \times PGD^{0.319}$$

Where:

Repair Rate is in terms of repairs/1,000 feet of pipe

K_2 is a constant selected for the expected performance of various pipe materials

PGD is in inches

The values the ALA document proposed for K_2 are shown in Table 4.

Table 4: K2 values for PGD fragility determination

Pipe Type	K2 Value
Concrete with steel cylinder, gasketed joint	0.7
PVC, gasketed joint	0.8
DIP, gasketed joint	0.5
AC, gasketed joint (coupling)	0.8

A K_2 value of 0.7 is proposed for this project based on the similarities of reinforced concrete pipe with the K_2 values proposed for other pipe materials.

As with wave propagation, a 33 percent reduction is taken because of the large pipe diameter.

The PGD Repair Rate for settlement is:

$$\text{Repair Rate} = K_2 \times 1.06 \times PGD^{0.319}$$

$$\text{Repair Rate} = 0.7 \times 1.06 \times 2^{0.319} \times 0.20 \times 0.67 = 0.124/1{,}000 \text{ feet of pipe.}$$

ALA (2001) provides direction on the type of repairs, breaks and leaks, which occur as a function of the geotechnical environment. A break is described as loss of hydraulic continuity across the failure to the extent that the pipe is no longer conducting fluid. A break may involve movement of multiple segments of pipe. Mass PGD movements such as the movements described herein that are longitudinal along the pipe result in 80 percent breaks and 20 percent leaks. Small displacements such as from PGV and for this project, lateral spread perpendicular to the pipe and settlement, result in 20 percent breaks and 80 percent leaks.

Only a limited percentage of the force main corridor is expected to have liquefaction that would result in lateral spreading or settlement (with the exception of approximately 1,850 feet at the Alameda Creek crossing and the area just south of the creek crossing). No liquefaction is expected from the Irvington Pump Station up to an estimated 2,640 feet north (5,280 feet of 39-inch pipe). Based on this information, it is estimated that of the total 66,684 foot long corridor (40,513 feet – 33-inch; 26,171 feet - 39-inch), 75,746 feet (2 X (40,513 – 2,640)) of the 33-inch force main, and 48,642 feet (2 X (26,171 – 1,850)) of the 39-inch twin force mains are subject to liquefaction with an average estimated settlement of 2-inches. And of those exposed lengths, only 25% of the force main corridor (with the exception of Irvington Pump Station and the Alameda Creek crossing and the area just south of the creek crossing) is actually expected to liquefy to the extent that settlement or lateral spread will occur (18,937 - 33-inch pipe; 12,161 feet - 39-inch pipe). 25% of the force main corridor is a total length of 15,449 feet.

If there is significant liquefaction/lateral spread at the force main crossing where it crosses Alameda Creek, and the area to the south of Alameda Creek (totaling 1,850 feet of force main corridor), there is potential that a significant percentage of force main in that area would have to be relayed and/or replaced.

There are other possible locations where lateral spreading could occur longitudinally along the force mains, particularly where they cross under drainage ditches. However, in all cases, the pipe is at least several feet below the bottom of the invert of the ditch, so there is only a small likelihood a failure would occur. Two leaks are estimated to occur at one of these locations (assumed to be on a 33-inch force main).

The resulting estimated number of pipe failures is shown in Table 5. While the estimated number of repairs is calculated to the nearest tenth, these estimates represent the mid-point of a range of potential failures; the total number of repairs could range from 5 to 15 plus failures in the vicinity of Alameda Creek.

Table 5: Resulting Pipe Repairs

	Feet of Pipe (double corridor length)	Repair Rate	Total Repairs	Leaks	Breaks
Total Pipe					
33-inch	81,026				
39-inch	52,342				
PGV					
33-inch	62,090	0.029/1000ft	1.8	1.4	0.4
39-inch	39,256	0.029/1000ft	1.2	1.0	0.2
PGD (settlement)					
33-inch	18,937	0.124/1000ft	2.4	1.9	0.5
39-inch	12,161	0.124/1000ft	1.5	1.2	0.3
PGD (parallel to force main)					
33-inch	Ditch crossing – Misc. Location	Observation	1.0	2.0	0
39-inch	3,700 Alameda Creek and south	Note 1			
Total All Pipe			8.9	7.5	1.4
33-inch			6.2	5.3	0.9
39-inch (Note 1)			2.7	2.2	0.5

Notes: 1. Potentially 1,850 feet of 39-inch force main corridor (3,700 feet of 39-inch pipe) could be damaged by significant lateral spreading, and would have to be rebuilt following an earthquake.

4.3.1 Possible Mitigation Alternatives

There are two groups of potential failures to address, failures: (1) due to significant liquefaction/lateral spread at Alameda Creek and immediately south and (2) distributed along the force mains.

Significant Liquefaction/Lateral Spread at Alameda Creek and Immediately South:

This assessment is based on limited information. To proceed, a detailed geotechnical investigation would be required to determine the extent and characteristics of the liquefiable material in the area of concern. A detailed geotechnical investigation at their Alameda Creek crossing and the area immediately south of Alameda Creek should include the following tasks:

1. Complete a detailed topographic survey of Alameda Creek and levees including the creek's low flow channel and the ponds immediately south of the Alameda Creek immediately adjacent to the force mains alignment including the lowest adjacent pond bottom elevation.
2. Complete a series of test borings to minimum depths of 50 feet below the native, original ground surface. A minimum of one test boring should be drilled through each levee, one boring on the north side of Alameda Creek and three borings evenly spaced on the south side of Alameda Creek for a total of six test borings. Standard Penetration Test blow counts (N-Values) per ASTM D1586 should be taken at no more than 5 foot vertical intervals.

3. Complete a series of Cone Penetration Test probes (CPTs) to minimum depths of 50 feet below the native, original ground surface. A total of six to eight CPTs should be completed with a minimum of two CPTs completed adjacent to test borings for ground proofing. If permitting allows, complete two CPTs within Alameda Creek. The combination of test borings and CPTs should result in a spacing of less than 200 feet between points of exploration.
4. Complete laboratory testing on disturbed and "undisturbed" soil samples from the test borings including: moisture content, unit weight, Atterberg limits, grain size distribution and shear strength.
5. Evaluate individual soil layer liquefaction potential by CPT liquefaction assessment software such as cLiq or Liquefypro.
6. Evaluate the extent of lateral spreading into Alameda Creek and into the ponds immediately south of Alameda Creek and the need for any supplemental geotechnical data.
7. Evaluate the application of compaction grout for densification of identified loose sands to mitigate liquefaction. Develop preliminary guidelines for before and after testing of the density and liquefaction potential of loose sands (e.g. the effectiveness of compaction grouting).

The estimated cost of this geotechnical investigation is $85,000 to $110,000.

If it is determined that the Alameda Creek crossing and the force main segment south of Alameda Creek are susceptible to significant liquefaction and lateral spreading, two alternatives are proposed for mitigation of the force mains at Alameda Creek, (1) slip lining the force mains and (2) compaction grouting to mitigate liquefaction.

Slip Lining:

The objective of this approach is to hold the pipe together and in the event some joint separation does occur, bridge the gap. Use of high density polyethylene (HDPE) is recommended compared to other lining materials due to its high ductility. There is a concern that cured in place liners and PVC slip lining products are brittle and would fail under tension and shear loading. One or both of the twin force mains could be slip lined.

Traditional slip lining using HDPE could be used. The wall thickness to be designed to provide the adequate tensile strength to hold pipe segments together and to bridge gaps that developed between segments. The traditional slip lining process would allow the distribution of pipe strain along the entire length of pipe rather than concentrating it at the pipe joints. Traditional slip lining would result in reduction of pipe inside diameter reducing its capacity.

One of three alternative technologies could be employed to minimize the reduction in pipe diameter, (1) Swagelining, (2) fold and form, and (3) roll down. Fold and form and roll down are limited to thinner wall HDPE lining which may not be adequate to achieve the identified performance goals. All three of these methods would result in the HDPE lining being pushed against the pipe wall making it difficult to distribute pipe strain along its entire length.

A recent Swagelining project in the Houston area lining 39-inch PCCP with DR 21 HDPE cost approximately $500 per foot plus overhead and administrative costs. As labor rates in the San Francisco Bay area are significantly higher, an estimate of $600 per foot will be used for comparison of alternatives. At $600 per foot, the cost of lining one force main 1,850 feet long would be $1.1 million plus overhead and administrative costs. While there may be some cost savings if both force mains were lined, the total cost would still be on the order of $2 million.

Traditional slip lining would be somewhat less expensive than Swagelining or the other specialized slip lining processes.

The slip lining process could be accomplished from two or three access pits, one at each end and possible one on the immediate south side of Alameda Creek.

A detailed analysis would be required to select the preferred slip lining methodology and to evaluate the structural demands on the force main lining. The estimated cost of this analysis would be on the order of $100,000 not including the cost of detailed design.

Compaction Grouting:

The objective of this approach is to stabilize the liquefiable material along the corridor. Both of the twin force mains would be stabilized as they are too close together to stabilize just one. Compaction grout (also referred to as displacement grout) would be pumped into the liquefiable soil lenses to density the soils making them stable in a seismic event.

The grout would be injected in a 6 to 7 foot grid spanning the force mains. Using a series of assumptions relative to liquefiable loose sand layer thickness and grout take (i.e. the quantity of compaction grout needed to achieve the desired densification of loose sands), it is estimated that it would take 2,055 cubic yards of grout. Note that these assumptions need to be verified in a subsequent geotechnical assessment (the compaction grout geotechnical assessment will build on the geotechnical investigation described above for evaluation of liquefaction and lateral spreading at Alameda Creek and the ponds south of the creek). The cost of compaction grouting is estimated to be approximately $450 per foot.

However, in addition to the cost of injecting the grout, access has to be provided along the entire length of the force main to move injection equipment into place. Construction of a temporary road will be required. Environmental permits will also be required. The cost of additional geotechnical testing (e.g. before and after N-Value and CPT comparisons), road construction and obtaining permits will increase the cost well beyond $600 per foot.

Conclusion:

The preliminary seismic performance rating for the pipeline at the Alameda Creek is 9, but further investigation is required for both alternatives to better assess their applicability for mitigation of liquefaction/lateral spread concerns along the force main corridor. The compaction grouting alternative has many more unknowns and higher level of uncertainty at this time than the slip lining approach.

Distributed Force Main Failures in Other Locations:

Table 5 shows a total of 9 failures in the force mains in areas beyond Alameda Creek and the corridor to the south. Because the expected failures of the force mains beyond Alameda Creek are localized, the seismic performance rating of this section of pipe is 6. The location of these failures will be distributed along the force mains so mitigation of the entire force main would be prohibitively expensive. It is recommended that the District address this deficiency by enhancing their ability to quickly make repairs. The district should purchase repair sleeves for both the 33-inch and 39-inch force mains and store them, making them available for repair in the days following the earthquake. The District has experience repairing leaks using repair sleeves. Further thought should be given to repair materials that would be required to repair a pipe “break”. This could possibly include several sections of pipe that could be inserted in place of damaged pipe, and a close sleeve between the new pipe sections. The cost of the pipe repair sleeves has historically been on the order of $30,000 each. The cost of acquiring repair sleeves and pipe repair sections for the number of failures estimated would be on the order of $300,000 to $500,000.

Emergency operation enhancements improvements could be considered. It would be desirable if the District could develop the capability to pump around force main failures. However, this would involve installation of numerous line valves to isolate failures. The costs would be substantial.

5.0: Conclusions and Recommendations

Based on our preliminary seismic assessment of the major pipelines and selected structures at Union Sanitary District, some of the pipelines and structures that are critical to life-safety and restoring basic service are vulnerable to seismic damage. To fully assess the complete vulnerability of the system, further studies and investigations are required. Likewise, to fully assess the potential cost of seismic upgrades, specific retrofits will need to be designed for individual structures and pipeline sections, and these retrofits will need to be priced by contractors/estimators based on the market conditions at the time of expected work.

Based on this preliminary investigation and our discussions with USD Management, seismic mitigation is recommended at the force mains near the Alameda Creek crossing, and for structures that are critical for life-safety or the primary transport and disinfection process. Based on this limited preliminary study, we estimate that a rough order of magnitude construction cost to seismically upgrade the most vulnerable structures and pipelines will be on the order of $40,000,000 in 2016 dollars. If the retrofit is limited to structures and pipelines with importance ratings of 6 or greater (primary fluid treatment and above) the estimated construction cost is on the order of $25,000,000 in 2016 dollars.

APPENDIX A - Geotechnical TM 6.17.14 Final Report

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Technical Memorandum

To:	Roger Parra Degenkolb Engineers	Date:	June 17, 2014
From:	Dave Mathy DCM Consulting, Inc.	File:	No. 156
Subject:	Union Sanitary District Seismic Vulnerability Assessment of the Union Sanitary District Facilities		

1.0 INTRODUCTION

This technical memorandum summarizes DCM Consulting, Inc.'s (DCM) geotechnical engineering research and review in support of Degenkolb Engineers' seismic vulnerability assessment of Union Sanitary District facilities in Union City, Newark and Fremont, California. The seismic vulnerability assessment includes as many as 81 structures at the District's Alvarado Waste Water Treatment Plant site, 6 pump stations (Fremont Boulevard Lift Station, Irvington Pump Station, Boyce Road Lift Station, Cherry Street Lift Station, Newark Pump Station, and Paseo Padre Lift Station) and 8 miles of 33-inch, reinforced-concrete twin force main pipelines and 5 miles of 39-inch, reinforce-concrete twin force main pipelines. All of the District facilities included in the seismic vulnerability assessment are in the western (downgradient) side of the District service area within a few miles of the eastern margin of San Francisco Bay. This technical memorandum provides a general characterization of soil conditions (i.e., composition and consistency) and groundwater conditions (i.e., groundwater depth below ground surface) at the District's waste water treatment plant, 6 pump stations and along 13 miles of 33-inch and 39-inch twin force mains as well as conclusions regarding site classification for seismic evaluation, shear wave velocity profiles for seismic evaluation and liquefaction and lateral spreading risks.

2.0 FINDINGS

The attached Figure 1 is a map of surficial soil deposits by the U.S. Geological Survey (Open File Report 97-97) with the District's twin force mains, waste water treatment plant and Newark and Irvington pump stations shown. As shown on Figure 1, the District facilities are roughly parallel to the eastern margin of San Francisco Bay and the East Bay Hills with Niles Canyon opposite the mid-area of the twin force mains. The District facilities are located near the transition between the alluvial outwash plain from the East Bay Hills and Niles Canyon (located about 4 to 5 miles to the east) and interfluvial basin deposits associated with poorly drained areas along the eastern margin of San Francisco Bay (the Bay is located about 2 to 3 miles to the west). The alluvial outwash deposits on Figure 1 are designated as "Qhfp – Alluvial Fan and Fluvial Deposits (Holocene), Brown or tan, medium dense to dense gravelly sand or sandy gravel that generally grades upward to sandy or silty clay." The basin deposits on Figure 1 are described as "Qhb – Basin Deposits (Holocene), Very fine silty clay to clay deposits occupying flat-floored basins at the distal edges of alluvial fans." The predominant surficial soil type along the 13 miles of District facilities are the fine-grained silty clay and clay Basin Deposits. The Basin Deposits are underlain by the Alluvial Fan and Fluvial Deposits which at this distance from the East Bay Hills (4 to 5 miles) are predominantly silts, sands and silty sands with some gravels, particularly in the upper 40 to 50 feet consistent with the "grades upward to sandy to silty clay" description by U.S.G.S. for Alluvial Fan and Fluvial Deposits. At depths of about 40 to 50 feet below ground surface, the Newark Aquifer is encountered. The Newark Aquifer is considerably coarser than the overlying silts, sands and silty sands and includes considerable gravel- and cobble-size material. The Newark Aquifer is part of the Alameda County Water District Aquifer Reclamation Program (ARP).

2.1. Generalized Soil Profile

The generalized soil profile along the 13 miles of District facilities connected to and including the twin force mains consists of:

- **Fills –** Associated with land reclamation, fills along the 13 mile twin force mains alignment and at pump stations are typically 3 to 5 feet thick. The exception to typical fill thickness is at the waste water treatment plant where soft soils (e. g., Bay Mud) have been removed prior to construction of the majority of treatment structures. Fills at the waste water treatment plant can be as much as 10 to 15 feet thick. Fills at the waste water treatment plant are typically well compacted (stiff and medium dense to dense) and composed of a variety of surficial native soils (clays) and import fills including sands and gravels.

- **Bay Mud –** San Francisco Bay Mud locally overlies the Basin Deposits at the waste water treatment plant. The Bay Mud is very soft and weak, high plasticity clay with thicknesses at the waste water treatment plant on the order 4 to 8 feet. Because of its limited thickness, development of the waste water treatment plant has generally included removal of Bay Mud from beneath treatment structures and replacement with engineered fill. There are some areas within the waste water treatment plant where the Bay Mud remains (principally the west side) and some treatment plant yard piping is within and/or underlain by Bay Mud.

- **Basin Deposits/Alluvial Fan and Fluvial Deposits –** The Basin Deposits are principally composed of fine-grained silty clays and clays of medium to high plasticity. These clays are typically firm to stiff and are on average 5 to 10 feet thick. Below a depth of about 5 to 10 feet, the silty clays and clays are interbedded with non-cohesive silts and sands from the Alluvial Fan and Fluvial Deposits. These non-cohesive soils vary from silty sands to clean, poorly graded sands in layers that are a few feet thick to as much as 10 feet thick. The non-cohesive silts and sands vary from loose to medium dense in place.

  The Newark Aquifer underlies most of the District facilities at a depth of about 40 to 50 feet below ground surface. The Newark Aquifer is coarse grained, dense to very dense sand, gravel and cobbles. The Dewatering (Centrifuge) Building at the waste water treatment plant is supported by pre-cast pre-stressed reinforced-concrete piles bearing in and on the dense Newark Aquifer sands and gravels.

The exception to the generalized soil profile is at the Irvington Pump Station. The Irvington Pump Station is underlain by Flood Basin Deposits (Qhbs on Figure 1). These Flood Basin Deposits are consistently clayey to depths greater than 50 feet. The clays are medium to high plasticity and are stiff to very stiff in place.

2.2. Groundwater

Groundwater is encountered along the 13 miles of District facilities at depths of 5 to 10 feet below ground surface.

3.0 CONCLUSIONS

Development of the waste water treatment plant including original construction in 1978 and plant expansions in 1985 and 1992 has resulted in the removal of the majority of the thin Bay Mud layer underlying the plant site (specifically for treatment structures that are part of the plant's hydraulic grade line). Deep structures have been founded on mat foundations bearing on competent soils beneath the Bay Mud and shallow structure footprints have been overexcavated through the Bay Mud to competent soils and backfilled to mat foundation grade with engineered fill. The structure excavations and overexcavations extend several feet beyond the structure footprints and were generally side-sloped at about 1H:1V or flatter. As a result, the inlet/outlet piping at the structure interface is in fill which is mostly well compacted. A small number of structures at the treatment plant are on pile foundations including the Dewatering Building (Building No. 81) and Sludge Thickener Tanks 3 and 4 and the south end of the Thickener Control Building (Building No. 62). The dewatering building was too top heavy for a mat foundation (i.e., piles needed to resist seismic overturning) and at the time of construction of Sludge Thickener Tanks 3 and 4, the area was too congested for earthwork overexcavation as previously described. Remnant Bay Mud has been encountered at the northwest corner and west side of the treatment plant at recent outfall pipeline and cogeneration building projects.

The majority of geotechnical investigations along the 13 miles of District facilities including the waste water treatment plant have concluded that many of the localized and isolated alluvial non-cohesive silt and sand layers interbedded with fine grained (cohesive clay) alluvium and Basin Deposit clays in the upper 50 feet of soils are liquefiable. These layers are confined by the upper clayey Basin Deposits which are on average 5 to 10 feet thick and by intervening clay layers between sand deposits. Therefore, the risk of ground surface failure through surface liquefaction (sand boils) is remote. However, ground surface settlement as a result of confined sand layer liquefaction is predicted. At the waste water treatment plant, various investigations (2, 8, 9 and 10) have predicted ½ to 3½ inches of liquefaction-induced ground surface settlement (average of 1 to 2 inches). In the area of the Newark Pump Station, recent investigations (1 and 2) have predicted as much as 3 to 5 inches of liquefaction-induced ground surface settlement with 1½ to 3 inches of differential settlement over a distance of 50 to 100 feet (i.e. 1½ inches over 50 feet and 3 inches over 100 feet). Based on the central location of these recent investigations and typical soil conditions, this predicted total and differential ground surface settlement upon liquefaction should be used for seismic risk evaluation of the twin force mains and connected pump stations. Because of its unique geologic profile, the likelihood of liquefaction at the Irvington Pump Station is low to nil. All of the remainder of the force main alignment is subject to liquefaction, however for any one seismic event less than 25% of the force main alignment should be impacted.

For purposes of seismic evaluation, the Site Classification (ASCE 7-05, Table 20.3.1) for all District facilities along the 13-mile reach shown on Figure 1 should be taken as Site Class "D".

For purposes of seismic evaluation, the average shear-wave velocity in the upper 30 meters (100 ft.) (Vs 30) should be taken as 270m/sec (900 ft/sec). Bedrock is quite deep under the Basin Deposits and Alluvial Fan and Fluvial Deposits and in the area of the Dumbarton Bridge is in excess of 200 to 300 feet. Therefore, shear wave velocities of 1,000m/sec (3,280 ft/sec) (Z1) typical of soft to medium bedrock and 2,500 m/sec (8,200 ft/sec) (Z2.5) typical of hard bedrock are greater than 200 feet deep. The only possible exception to this bedrock depth is an approximate 20-foot-high bedrock outcrop on the west side of Hickory Street in Newark and about 100 feet west of the force mains where bedrock and Z1 may be in the range of 100 to 200 feet deep. This is a very small and localized area over a length of the exposed bedrock outcrop of about 500 feet trending southeast to northwest.

The twin force mains cross under Alameda Creek and are immediately adjacent to ponds on the south side of Alameda Creek. At this location, the soils consist predominantly of Alluvial Fan and Fluvial Deposits (see Figure 1). As previously discussed, the interlayered silty sands and sands within these deposits are subject to liquefaction. The channel area is approximately 910 feet wide (between two force mains manhole structures). The channel itself is about 750 feet wide (including levees), about 20 feet deep below the top of levees and about 10 feet deep below adjacent local ground surfaces. The depth of cover within the channel and specifically at the low flow channel is only a few feet and as such lateral spreading upon liquefaction of the shallow interlayered alluvial sands is a risk warranting further investigation. The pond area immediately south of Alameda Creek is immediately adjacent to the force mains alignment with the bottom of the ponds at elevations lower than the nearby force mains (see original force main drawing no. 9 and pipeline station 112+00 to 116+00). The ponds immediately adjacent to the force mains extend over a length of approximately 940 feet south of Alameda Creek. The Alameda Creek undercrossing and the ponds south of the creek represents the principal location along the 13-mile twin force main alignment where large-scale ground deformations from lateral spreading directly impacting the force mains are a possibility. Further site-specific investigation is warranted at this location to establish the:

a) design and as-built construction details for the twin force mains creek undercrossing (e.g. concrete encasement, concrete cap, pipe anchorage, etc.),

b) likelihood of liquefaction and lateral spreading into the creek,

c) stability of the creek bottom and levees upon liquefaction,

d) stability of the pond bottoms and slopes upon liquefaction, and

e) lateral extent of spreading north and south of the creek.

Photo 1 - Alameda Creek looking east from the north levee.

Photo 2 - Alameda Creek looking west from the north levee.

Photo 3 – Southward view of ponds on the south side of Alameda Creek and the force mains alignment indicated by 3 marker posts.

Photo 4 - Force mains marker post on the immediate eastern edge of ponds south of Alameda Creek.

4.0 LIMITATIONS

This technical memorandum has been prepared for the exclusive use of Degenkolb Engineers and Union Sanitary District as part of the District's Seismic Vulnerability Assessment as described herein and is based on a desktop study of existing geotechnical conditions at District facilities. This technical memorandum may not be used for any other purpose or for any other project. Within the limitations of scope and budget, DCM Consulting Inc.'s services have been provided in accordance with generally accepted practices in the field of geotechnical engineering in the San Francisco Bay Area. The conclusions presented in this technical memorandum are based on the author's professional knowledge, judgment and experience. No other warranty or other conditions express or implied should be assumed or understood.

5.0 REFERENCES

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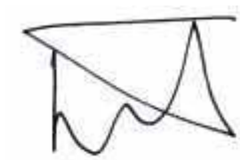

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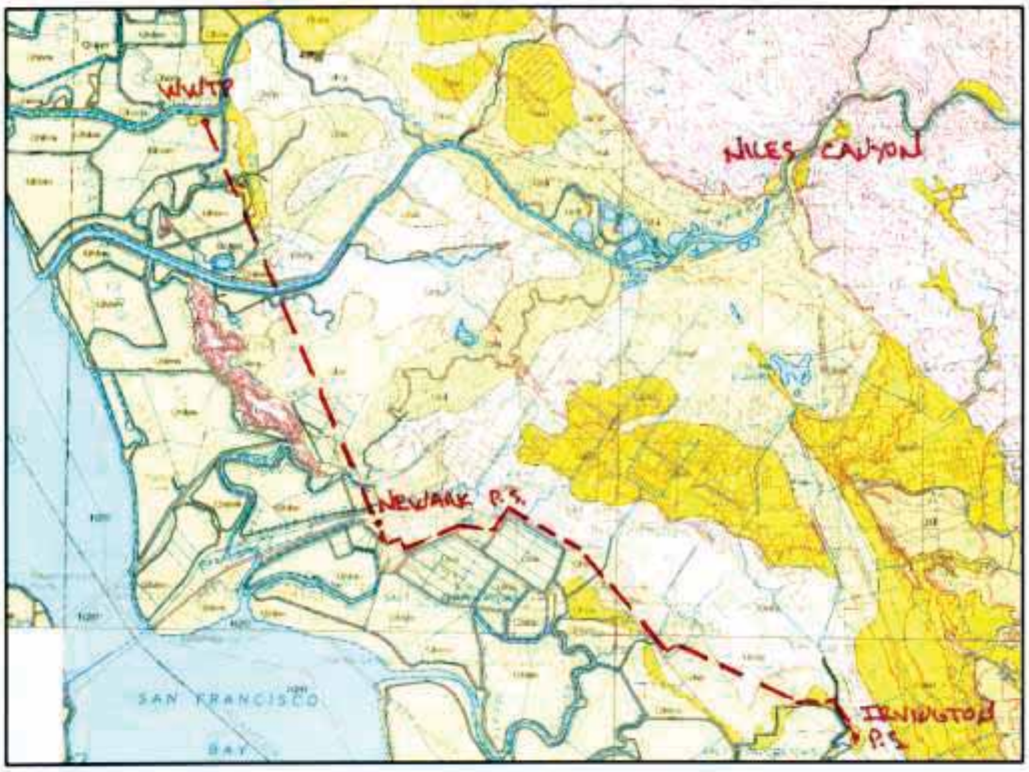
Attachments: Figure 1 – USD Force Mains Alignment Geology

USD Force Mains Alignment Geology

File: 156



Qhb – Basin Deposits (Holocene), Very fine silty clay deposits occupying flat floored basins at the distal edge of alluvial fans.

Qhpf – Alluvial Fan and Fluvial Deposits (Holocene), Brown or tan, medium dense to dense, gravelly sand or sandy gravel that generally grades upward to sandy or silty clay.

Qhbs – Flood Basin Deposits (Holocene), Clay to very fine silty clay deposits similar to Qhb deposits except they contain carbonate nodules and iron stained mottles.

Source: U.S. Geologic Survey Open-File Report 97-97

Figure 1

APPENDIX B - Checklist

Degenkolb
DEGENKOLB ENGINEERS

Building Name: Degritter Building Date: 3/24/14
Building Address: Page: 1 of 3
Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☐	☒	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	**There are numerous and very serious deficiencies in the lateral load path at the roof level.** • **No positive attachment of roof diaphragm to North and South wall panels.** • **Concrete fin walls at the East and West ends of the structure are not positively attached to the roof diaphragm.** • **Roof double-t beams are only positively connected to the precast wall panels at one end of the structure.** • **Diaphragm ties are overstressed.** • **There is virtually no diaphragm in the center of the structure.** • **New cast-in-place concrete walls at the 2nd floor near the Degitter machine are tied into the cast-in-place diaphragm elements above. These diaphragm elements were originally designed and intended to take gravity loads associated with framing the large openings in the roof. These beams are very likely deficient for lateral forces and eccentricity placed on them by the new concrete wall elements.**
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Limited piping and ducts in and out of structure some without flexible couplings. This needs to be studied further.** **Large tower structure adjacent to Degritter seems to be braced independently from the Degritter, but is close enough that pounding is possible. Further study required.**
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	

Degenkolb
DEGENKOLB ENGINEERS

Building Name: Degritter Building		Date: 3/24/14	
Building Address:		Page: 2	of 3
Job Number: B3215013.00	Job Name: Union Sanitary Seismic Evaluations	By: CAH	Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	

Degenkolb
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Building Name: Degritter Building | Date: 3/24/14

Building Address: | Page: 3 of 3

Job Number: B3215013.00 | Job Name: Union Sanitary Seismic Evaluations | By: CAH | Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

Degenkolb
DEGENKOLB ENGINEERS

Building Name: Degritter Building Date: 3/24/14

Building Address: Page: 1 of 3

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				Connections	
☒	☐	☒	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1)	**1st floor walls are adequately anchored to the diaphragms. 2nd floor walls are sufficient to span horizontally and are not reliant on being anchored to the diaphragm.**
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☒	☐	☐	SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.3.1. Tier 2: Sec. 5.5.3.1.1)	**Max stress ~100 psi. Note that wall panel bars are weld connected at the base, which creates a brittle connection. Checking this connection as a force controlled action DCR ~ 200%, but at the critical level (1st floor in the East-West direction) there are also CIP walls that will perform much better. Additionally, the concrete beams and very large column elements will participate to resist lateral loads in frame action. Therefore, there appears to be enough lateral force resisting elements provided, but further analysis is recommended.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.3.2. Tier 2: Sec. 5.5.3.1.3)	
				Diaphragms	
☐	☒	☐	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	**Diaphragm ties are overstressed**
				Connections	

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
☐	☐	☒	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	
☐	☒	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)	**There is almost no positive connection of the roof diaphragm to the walls (with the exception of 1 end of the double t's). This is a major deficiency.**
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2 Sec. 5.6.1)	
☐	☐	☒	☐	GIRDER/COLUMN CONECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

Seismic-Force-Resisting System

C	NC	N/A	U		Comments
☒	☐	☐	☐	DEFLECTION COMPATIBILITY FOR RIGID DIAPHRAGMS: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**Checked weak-way for intermediate/interior columns (only columns/direction that will undergo typical column seismic deformation)**
☒	☐	☐	☐	WALL OPENINGS: The total width of openings along any perimeter wall line constitute less than 75 percent of the length of any perimeter wall with the wall piers having aspect ratios of less than 2-to-1. (Commentary: Sec. A.3.2.3.3. Tier 2: Sec. 5.5.3.3.1)	

Diaphragms

C	NC	N/A	U		Comments
☐	☒	☐	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	**Diaphragm is not technically flexible so not technically a deficiency. It is marked here, however, to flag the reliance on the diaphragm ties as chord elements.**
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	

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Building Name: Degritter Building Date: 3/24/14
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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	
				Connections	
☒	☐	☐	☐	MINIMUM NUMBER OF WALL ANCHORS PER PANEL: There are at least two anchors from each precast wall panel into the diaphragm elements. (Commentary: Sec. A.5.1.3. Tier 2: Sec. 5.7.1.4)	
☒	☐	☐	☐	PRECAST WALL PANELS: Precast wall panels are connected to the foundation. (Commentary: Sec. A.5.3.6 Tier 2: Sec. 5.7.3.4)	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
☐	☐	☒	☐	GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts. (Commentary: Sec. A.5.4.2. Tier 2: Sec. 5.7.4.2)	

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Building Name: Primary Clarifiers 5-6 Date: 3/26/14

Building Address: Page: 1 of 2

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☐	☒	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	**There are numerous and very serious deficiencies in the lateral load path at the roof level.** • **No positive attachment of roof diaphragm to North and South wall panels. Probable that there is positive attachment at the end column corbels but this load path is almost certainly deficient.** • **Concrete fin walls at the East and West ends of the structure do not appear to be positively attached to the roof diaphragm.** • **Roof double-t beams are only positively connected to the precast wall panels at one end of the structure.** • **Diaphragm ties are overstressed.**
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Limited piping and ducts in and out of structure some without flexible couplings. This needs to be studied further.** **Large tower structure adjacent to Primary Clarifiers seems to be braced to the roof of the clarifiers. Given that the clarifier's roof attachment is deficient, this connection will likely be ineffective. Further study required.**
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	

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Building Name: Primary Clarifiers 5-6 Date: 3/26/14

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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Building Name:	Primary Clarifiers 5-6	Date:	3/26/14
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Job Number: B3215013.00	Job Name: Union Sanitary Seismic Evaluations	By: CAH	Checked:

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				Connections	
☐	☒	☒	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1)	**2nd floor walls are sufficient to span horizontally and are not reliant on being anchored to the diaphragm.** **Pilasters are slightly overstressed to cantilever the weight from the pre-cast walls down to the foundation. However, with further study of the actual load transfer (3 sided attachment) and geotechnical input on the passive soil pressures it is likely that the wall/pilaster system can be shown to be sufficient for the out-of-plane forces.**
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.3.1. Tier 2: Sec. 5.5.3.1.1)	**Max stress ~50 psi. Note that wall panel bars are weld connected at the base, which creates a brittle connection. Checking this connection as a force controlled action DCR ~ 70% (note this also assumes an inverted triangular load distribution, which is conservative for this structure). Therefore, there appears to be enough wall provided.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.3.2. Tier 2: Sec. 5.5.3.1.3)	
				Diaphragms	
☐	☒	☐	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	**Diaphragm ties are overstressed.**
				Connections	
☐	☐	☒	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	

Building Name: Primary Clarifiers 5-6 Date: 3/26/14

Building Address: Page: 2 of 4

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
☐	☒	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)	**There is almost no positive connection of the roof diaphragm to the walls (with the exception of 1 end of the double t's). This is a major deficiency.**
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2 Sec. 5.6.1)	
☐	☒	☐	☐	GIRDER/COLUMN CONECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	**NG at one end of the precast roof beams. At pilasters adjacent to sludge room (which have the greatest possibility of losing bearing support, there is no positive connection from the roof beams to the pilaster.**

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

Seismic-Force-Resisting System

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
☐	☒	☐	☐	DEFLECTION COMPATIBILITY FOR RIGID DIAPHRAGMS: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**Checked weak-way for intermediate pilaster #1 @ North/South side for bending between roof and 2nd floor. Not likely a deficiency with advanced analysis because we know at corners of building, the structure is stiff enough to protect the columns and at the middle of the building the 2nd floor diaphragms are not stiff enough to enforce double curvature in the pilasters. Diaphragm may experience damage at re-entrant corners from sludge pump room. These re-entrant corners are small relative to the overall footprint of the digesters and wouldn't be a significant concern for a well detailed building with a true "rigid" diaphragm. However, these sludge pump room end walls (even though they are short) are stiff compared to the diaphragm, and may try to take significant amount of load relative to their length. Without introducing a collector, this could lead to local diaphragm damage. Further analysis required.**
☒	☐	☐	☐	WALL OPENINGS: The total width of openings along any perimeter wall line constitute less than 75 percent of the length of any perimeter wall with the wall piers having aspect ratios of less than 2-to-1. (Commentary: Sec. A.3.2.3.3. Tier 2: Sec. 5.5.3.3.1)	
				Diaphragms	
☐	☒	☐	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	**Diaphragm is not technically flexible so not technically a deficiency. It is marked here, however, to flag the reliance on the diaphragm ties as chord elements.**
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	

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Building Name: Primary Clarifiers 5-6 Date: 3/26/14
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ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	
				Connections	
☒	☐	☐	☐	MINIMUM NUMBER OF WALL ANCHORS PER PANEL: There are at least two anchors from each precast wall panel into the diaphragm elements. (Commentary: Sec. A.5.1.3. Tier 2: Sec. 5.7.1.4)	
☒	☐	☐	☐	PRECAST WALL PANELS: Precast wall panels are connected to the foundation. (Commentary: Sec. A.5.3.6 Tier 2: Sec. 5.7.3.4)	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
☐	☐	☒	☐	GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts. (Commentary: Sec. A.5.4.2. Tier 2: Sec. 5.7.4.2)	

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Building Name: Primary Clarifiers 1-4 Date: 3/27/14

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U	Item	Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	**There is a well-defined load path, but certain aspects of the load path are very likely deficient and need further analysis.**
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Limited piping and ducts in and out of structure some without flexible couplings. This needs to be studied further.** **Adjacent to the expanded and re-built control box #1. The control box is partially integral and partially separated from the clarifiers. Given the potential movement of the clarifier walls, this should be studied further.**
☐	☒	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	**Mezzanine in the pump room is not adequately braced.**
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	

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Building Name: Primary Clarifiers 1-4 Date: 3/27/14

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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Building Name: Primary Clarifiers 1-4 Date: 3/27/14

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C NC N/A U **Comments**

LOW SEISMICITY

Connections

☐ ☒ ☐ ☐ WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1)

Retrofit anchorage (1991) at roof level is insufficient for out-of-plane anchorage, although it was very likely only intended to transfer in-plane loads.

Some panels are sufficient to span horizontally to columns while others are insufficient. If the entire infill panel weight is taken by the columns, the un-retrofitted columns are close to their capacity to get the load down to the top of the walls. With further analysis of a 3-sided support, it is likely that the wall anchorage is sufficient.

If the cantilever wall truly spans to the columns, then the walls will be locally overstressed. Without supplementing the out-of-plane anchorage at the roof, the cantilever wall may be overstressed considering concurrent fluid and inertial forces even if the weight of the panels is uniformly distributed across the width of the panel.

For outward loads, the input of a geotechnical engineer for passive pressures may significantly reduce the demands on the cantilever walls. For inward loads, the circular infill concrete may increase the capacity of the wall if the cantilever wall can span horizontally (these circular infills of concrete do not appear positively attached to the walls, but are positively attached to the foundation and could help buttress the walls). Additionally, modeling the walkway and channel diaphragms may have a minor positive effect on the capacity of the walls.

Note that Carollo recommended strengthening the columns in their 1991 report: "During an earthquake, these cantilever columns prevent the wall panels from moving away from the roof. Since the ties in these columns are relatively light, excessive deflections due to the seismic loads could cause brittle failure of the columns." However, the 1991 retrofit of Primary Clarifiers 1-4 neglected to strengthen the columns on the East, West and South sides of the clarifiers. Many of the exterior columns that were strengthened on the North side of the clarifiers only extended to the top of pipes that run along the walls of the buildings.

If the panels do primarily span horizontally, the deflection of the columns may be in excess of 5 inches. While our analysis indicates that there should be sufficient shear strength in the columns to avoid a brittle failure, this level of deflection is undesirable and further analysis is recommended.

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Building Name: Primary Clarifiers 1-4 | Date: 3/27/14
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ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☒	☐	☐	SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.3.1. Tier 2: Sec. 5.5.3.1.1)	**Max stress ~40 psi. Note that wall panel bars are weld connected at the base at discrete locations, which creates a brittle connection. This connection has been supplemented with angle bracket connections. Checking these combined connections as a force controlled actions they are overstressed. However this analysis assumes an inverted triangular load distribution, considers the wall mass, and does not reduce by a J-factor. Therefore, with advanced analysis this load path may be sufficient.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.3.2. Tier 2: Sec. 5.5.3.1.3)	
				Diaphragms	
☐	☒	☐	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	**Diaphragm ties are might be sufficient to take the mass of the roof only, but further analysis is recommended.** **Based on Carollo 1991 report, the ties are very likely corroded. Further exploration of the condition of the ties is necessary.**
				Connections	
☐	☐	☒	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	
☒	☒	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)	**Diaphragm is positively attached for in-plane shear but the connections appear to be deficient for the demands. Further analysis required.** **Note that when the control box was replaced in the early 1990's the drawings do not indicate that the new cast-in-place walls are positively connected to the precast concrete roof.**
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2 Sec. 5.6.1)	

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Building Name: Primary Clarifiers 1-4 Date: 3/27/14

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	GIRDER/COLUMN CONECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	**Carollo 1991 retrofit addressed this deficiency.**

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

Seismic-Force-Resisting System

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	DEFLECTION COMPATIBILITY FOR RIGID DIAPHRAGMS: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**Checked weak-way for column C-4 (#4 @ 12" which is typical of un-retrofitted column) and retrofitted center column.**
☒	☐	☐	☐	WALL OPENINGS: The total width of openings along any perimeter wall line constitute less than 75 percent of the length of any perimeter wall with the wall piers having aspect ratios of less than 2-to-1. (Commentary: Sec. A.3.2.3.3. Tier 2: Sec. 5.5.3.3.1)	

Diaphragms

C	NC	N/A	U	Item	Comments
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☒	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	**Existing diaphragm ties, new chord elements etc. need further analysis.**

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				Connections	
☒	☐	☐	☐	MINIMUM NUMBER OF WALL ANCHORS PER PANEL: There are at least two anchors from each precast wall panel into the diaphragm elements. (Commentary: Sec. A.5.1.3. Tier 2: Sec. 5.7.1.4)	
☒	☐	☐	☐	PRECAST WALL PANELS: Precast wall panels are connected to the foundation. (Commentary: Sec. A.5.3.6 Tier 2: Sec. 5.7.3.4)	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
☐	☐	☒	☐	GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts. (Commentary: Sec. A.5.4.2. Tier 2: Sec. 5.7.4.2)	

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Building Name: Irvington Pump Sation | Date: 3/26/14
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Job Number: B3215013.00 | Job Name: Union Sanitary Seismic Evaluations | By: CAH | Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☒	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Appears to be large pipes without flexible couplings. New adjacent building but doweled in so very likely OK.**
☐	☒	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	**Small office structures on upper floor do not appeared to be tied into the structure and have only gypboard. The small equipment platforms appear to be adequately tied into the structure but further analysis is recommended.**
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☐	☒	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	**Middle East-West running wall is offset at each floor. The walls run the entire width of the structure so this is very likely not a deficiency with further analysis.**
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☒	☐	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Per geotech report, little liquefaction potential at the Irvington site.**

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C	NC	N/A	U		Comments
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

FOUNDATION CONFIGURATION

C	NC	N/A	U		Comments
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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Building Name: Irvington Pump Station Date: 3/26/14
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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 RM1 / RM1A Life Safety Structural Checklist: Reinforced Masonry Bearing Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than 70 psi. (Commentary: Sec. A.3.2.4.1. Tier 2: Sec. 5.5.3.1.1)	**In-plane stress in masonry walls ~15 psi**
☒	☐	☐	☐	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 in either of the two directions; the spacing of reinforcing steel is less than 48 inches and all vertical bars extend to the top of the walls. (Commentary: Sec. A.3.2.4.2. Tier 2: Sec. 5.5.3.1.3)	
				Stiff Diaphragms	
☐	☐	☒	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	
				Connections	
☐	☒	☐	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	**Wall anchorage relies on bending of single angle connections and prying on a weld which is deficient. DCR ~400%. Note that some of the shear tabs appeared to have slotted holes (between actual slots and short slots so this was probably done in the field for installation tolerance) so they walls may be able to cantilever in certain locations without stressing the weld. A condition assessment to see how long these slots are and whether or not the high strength bolts are torqued should be done if supplementary anchorage is not added. Note that if the high strength bolts are torqued as expected, they would fail the weld before slipping.**
☐	☐	☒	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	
☒	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	

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ASCE 41-13 RM1 / RM1A Life Safety Structural Checklist: Reinforced Masonry Bearing Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.6.4)	
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
☒	☐	☐	☐	GIRDER / COLUMN CONNECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

Stiff Diaphragms

C	NC	N/A	U		Comments
☐	☐	☒	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
☐	☐	☒	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 feet long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)	

Flexible Diaphragms

C	NC	N/A	U		Comments
☐	☒	☐	☐	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	**Cross ties in the long (North-South) direction rely on plate bending and potential prying on bolted connections, which may be deficient and require further analysis.**
☒	☐	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
☒	☐	☐	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 feet long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	

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ASCE 41-13 RM1 / RM1A Life Safety Structural Checklist: Reinforced Masonry Bearing Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	
				Connections	
☒	☐	☐	☐	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 inch prior to engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)	**Reliant on plate bending, but distance is short enough that displacement at plate capacity is less than 1/8”**

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Building Address: | Page: 1 of 3
Job Number: B3215013.00 | Job Name: Union Sanitary Seismic Evaluations | By: CAH | Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	**A deficiency per ASCE 41 because the shear walls are also bearing walls. Very likely not a deficiency with more advanced analysis. Not judged a life-safety issue at this time.**
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**In-plane stress ~30psi. Note that this analysis does not consider the new structure that is doweled into the shared east wall. Based on the stresses in the wall this is OK by inspection (note that the shared wall is no longer a retaining wall so the actual stress in the wall during an earthquake would likely be lower with the new structure than without the new structure).**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	
				Connections	
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	
☒	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	

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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U	Item	Comments
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Seismic-Force-Resisting System	
☐	☐	☒	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	
☐	☒	☐	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	**Appears that CIP slabs bottom reinforcing is not hooked into walls. Not likely a life-safety issue.**
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☒	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☐	☒	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	**There are a series of openings on both sides of the second floor diaphragm. Given the inertial loads in the diaphragm, this shouldn't be a life-safety issue for seismic load transfer with additional analysis. However, the cast-in-place slab diaphragms tie the walls of the structure together to resist incremental soil pressures during a seismic event and the slab may be overstressed to serve this purpose. Further analysis is recommended.**

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Building Name: **Administration Building** Date: **4/2/14**

Building Address: Page: **1** of **2**

Job Number: **B3215013.00** Job Name: **Union Sanitary Seismic Evaluations** By: **CAH** Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☐	☒	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	**See comments @ precast panels**
☒	☐	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☐	☒	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	**Brace on gridline 8 is discontinuous at the 2nd floor. At one end of the brace the load is taken on a transfer beam. At the other end of the brace the load is taken eccentrically into a precast concrete wall element.** **Small brace on line G that braces the back of the curved low roof portion is discontinuous. The small brace is in line with a ledger beam, which is connected to a larger full-height brace at its mid length. Differential deformation along this drag will induce bending at the full-height brace.** **The Mansard roof is attached to the roof diaphragm for shear transfer. This load path should be assessed in greater detail, particularly because these points of shear transfer do not typically align with brace locations.**
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	

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ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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ASCE 41-13 S2 / S2A Life Safety Structural Checklist: Steel Braced Frames with Stiff or Flexible Diaphragms

C	NC	N/A	U	Item	Comments
				LOW SEISMICITY	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	AXIAL STRESS CHECK: The axial stress due to gravity loads in columns subjected to overturning forces is less than 0.10*Fy*. Alternatively, the axial stress due to overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than 0.30*Fy*. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.4.1)	**See calculations.** **Also, for a 2-story structure with chevron bracing, this check would typically be very penal, because there is very little frame action, and the columns at the first story are only subject to the force from the braces at the 2nd story. However, some of the braces in the administration building are inverted-V bracing. The size of the column is the same size as the 2nd floor brace. Therefore, at a capacity level, the column is overstressed to develop the tension capacity of the bracing.**
☒	☐	☐	☐	AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.5.3.4, is less than 0.50*Fy*. (Commentary: Sec. A.3.3.1.2. Tier 2: Sec. 5.5.4.1)	
				Connections	
☒	☐	☐	☐	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames. (Commentary: Sec. A.5.2.2. Tier 2: Sec. 5.7.2)	
☒	☐	☐	☐	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	REDUNDANCY: The number of lines of braced frames in each principal direction is greater than or equal to 2. The number of braced bays in each line is greater than 2. (Commentary: Sec. A.3.3.1.1. Tier 2: Sec. 5.5.1.1)	
☐	☒	☐	☐	CONNECTION STRENGTH: All the brace connections develop the buckling capacity of the diagonals. (Commentary: Sec. A.3.3.1.5. Tier 2: Sec. 5.5.4.4)	**OK except @ concentrated force at HSS column and anchor bolt connections deficient.**
☒	☐	☐	☐	COMPACT MEMBERS: All brace elements meet compact section requirements set forth by AISC 360 Table B4.1. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec. 5.5.4)	
☒	☐	☐	☐	K-BRACING: The bracing system does not include K-braced bays. (Commentary: Sec. A.3.3.2.1. Tier 2: Sec. 5.5.4.6)	

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ASCE 41-13 S2 / S2A Life Safety Structural Checklist: Steel Braced Frames with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Seismic-Force-Resisting System	
☐	☐	☒	☐	COLUMN SPLICES: All column splice details located in braced frames develop 50 percent of the tensile strength of the column. (Commentary: Sec. A.3.3.1.3. Tier 2: Sec. 5.5.4.2)	
☒	☐	☐	☐	SLENDERNESS OF DIAGONALS: All diagonal elements required to carry compression have *Kl/r* ratios less than 200. (Commentary: Sec. A.3.3.1.4. Tier 2: Sec. 5.5.4.3)	
☐	☒	☐	☐	CONNECTION STRENGTH: All the brace connections develop the yield capacity of the diagonals. (Commentary: Sec. A.3.3.1.5. Tier 2: Sec. 5.5.4.4)	**See above. Also, welds deficient for tension connection. Bolts also likely to slip at tension capacity.**
☐	☒	☐	☐	COMPACT MEMBERS: All brace elements meet section requirements set forth by AISC 341 Table D1.1 for moderately ductile members. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec. 5.5.4)	
☐	☒	☐	☐	CHEVRON BRACING: Beams in chevron, or V-braced, bays are capable of resisting the vertical load resulting from the simultaneous yielding and buckling of the brace pairs. (Commentary: Sec. 3.3.2.3. Tier 2: Sec. 5.5.4.6)	
☒	☐	☐	☐	CONCENTRICALLY BRACED FRAME JOINTS: All the diagonal braces shall frame into the beam-column joints concentrically. (Commentary: Sec. 3.3.2.4. Tier 2: 5.5.4.8)	
				Diaphragms (Stiff or Flexible)	
☐	☒	☐	☐	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the braced frames extend less than 25 percent of the total frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3)	**Some stair and atrium openings adjacent to frames.**
				Flexible Diaphragms	
☐	☒	☐	☐	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	**No continuous cross ties at curved low roof pre-cast panel portion of diaphragm. Doubtful that sub-diaphragms are strong enough for anchorage forces and doubtful joists will be able to serve as chords for sub diaphragm. Note that nominal unit shear capacity of low diaphragm is only 640 lb/ft (1/2” w/ 10d @ 6”)**
☒	☐	☐	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	

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C	NC	N/A	U		Comments
☒	☐	☐	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	**Diaphragms are typically blocked.**
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				Connections	
☐	☒	☐	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1)	**See calculations. Typical wall anchorage and development into the diaphragm @ roof level is overstressed.**
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				Seismic-Force-Resisting System	
☐	☐	☒	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.3.1. Tier 2: Sec. 5.5.3.1.1)	**Typically OK by inspection where panels are used as walls. However, further analysis is required where panel A5 takes transfer force from discontinuous brace.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.3.2. Tier 2: Sec. 5.5.3.1.3)	
				Diaphragms	
☐	☐	☒	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	
				Connections	
☒	☐	☐	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	
☒	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)	
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2 Sec. 5.6.1)	
☐	☐	☒	☐	GIRDER/COLUMN CONECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	

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ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Seismic-Force-Resisting System	
☐	☐	☒	☐	DEFLECTION COMPATIBILITY FOR RIGID DIAPHRAGMS: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	
☐	☐	☒	☐	WALL OPENINGS: The total width of openings along any perimeter wall line constitute less than 75 percent of the length of any perimeter wall with the wall piers having aspect ratios of less than 2-to-1. (Commentary: Sec. A.3.2.3.3. Tier 2: Sec. 5.5.3.3.1)	
				Diaphragms	
☐	☒	☐	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	**See S1A checklist**
☒	☐	☐	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	
				Connections	

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C	NC	N/A	U		Comments
				LOW SEISMICITY	
☐	☒	☐	☐	MINIMUM NUMBER OF WALL ANCHORS PER PANEL: There are at least two anchors from each precast wall panel into the diaphragm elements. (Commentary: Sec. A.5.1.3. Tier 2: Sec. 5.7.1.4)	**Panels A1 – A4 are not connected to a diaphragm and are only connected at one location at the top of the panel.** **For seismic action in the direction of the panels, it is conceivable that the panels can cantilever. In the direction normal to the panels, the eccentric moment has to be taken weak-way through shear tabs, which is deficient. At the North end of the panels, the back W16x26 drag is connected mid-height to Panel A5. On the South end of the panels, Panel A1 is connected to the W16x26 into the main building by a series of 9/S4.4 (the beams are @ different elevations). This load path is deficient.**
☒	☐	☐	☐	PRECAST WALL PANELS: Precast wall panels are connected to the foundation. (Commentary: Sec. A.5.3.6 Tier 2: Sec. 5.7.3.4)	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
☐	☐	☒	☐	GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts. (Commentary: Sec. A.5.4.2. Tier 2: Sec. 5.7.4.2)	

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C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Clear distance between low-bay and high-bay portions of building is only 3 inches (~2%). When diaphragm and wall deflections are considered the deflections could be considerably higher. This could lead to pounding between the two structures, with the diaphragm of the low-bay portion of the structure pounding mid-height on the precast concrete wall of the high-bay portion of the structure.**
☐	☒	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	**The mezzanine in the warehouse portion of the structure is structurally independent of the precast concrete walls. This structure appears to be a pre-engineered moment frame type of structure. Verification of the adequacy of this structure is recommended.**
				BUILDING CONFIGURATION	
☐	☐	☒	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	**1 story**
☐	☐	☒	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	**1 story**
☐	☐	☒	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	**1 story**
☐	☐	☒	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	**1 story**
☐	☐	☒	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	**1 story**

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ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
☐	☐	☒	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	**Diaphragm is flexible and there are many walls so no "torsion" deficiency. See W1 checklist for deficiencies related to diaphragm and load path.**
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				Connections	
☐	☒	☐	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1)	**Typical diaphragm straps have 250% DCR** **Sub-diaphragm development in N-S direction 170% DCR** **Architectural panels @ west bumpout do not appear anchored to the tilt-up walls at the high bay which could lead to pounding damage.** **Architectural panel at the low bay requires further study. The panel is connected directly to a shear wall at one location and it doesn't seem like the diaphragm shape and capacity will allow the other anchors to be very effective**
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.3.1. Tier 2: Sec. 5.5.3.1.1)	**25 psi N-S** **10 psi E-W**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.3.2. Tier 2: Sec. 5.5.3.1.3)	**#5 @ 12" E.W. Typ – p = 0.0028** **#3 ties @ 9" at narrow piers – p = 0.0028**
				Diaphragms	
☐	☐	☒	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	
				Connections	
☒	☐	☐	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	

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Job Number: B3215013.00 | Job Name: Union Sanitary Seismic Evaluations | By: CAH | Checked:

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
☒	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)	**There is almost no positive connection of the roof diaphragm to the walls (with the exception of 1 end of the double t's). This is a major deficiency.**
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2 Sec. 5.6.1)	
☒	☐	☐	☐	GIRDER/COLUMN CONECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	**Plates and thru bolts.**

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

Seismic-Force-Resisting System

C	NC	N/A	U		Comments
☐	☐	☒	☐	DEFLECTION COMPATIBILITY FOR RIGID DIAPHRAGMS: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**Flexible diaphragm.**
☒	☐	☐	☐	WALL OPENINGS: The total width of openings along any perimeter wall line constitute less than 75 percent of the length of any perimeter wall with the wall piers having aspect ratios of less than 2-to-1. (Commentary: Sec. A.3.2.3.3. Tier 2: Sec. 5.5.3.3.1)	**Max length of openings ~40%. Narrow piers not required for shear resistance.**

Diaphragms

C	NC	N/A	U		Comments
☒	☐	☐	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	**Hinge connectors rated for horizontal loads are provided along girder.** **Straps provided along purlins.**
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	**Diaphragm is blocked, high load diaphragm which appears to be sufficient at a life-safety level based on Tier 3 check.**

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	
				Connections	
☒	☐	☐	☐	MINIMUM NUMBER OF WALL ANCHORS PER PANEL: There are at least two anchors from each precast wall panel into the diaphragm elements. (Commentary: Sec. A.5.1.3. Tier 2: Sec. 5.7.1.4)	
☒	☐	☐	☐	PRECAST WALL PANELS: Precast wall panels are connected to the foundation. (Commentary: Sec. A.5.3.6 Tier 2: Sec. 5.7.3.4)	**#4 dowels @18" O.C.**
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
☐	☐	☒	☐	GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts. (Commentary: Sec. A.5.4.2. Tier 2: Sec. 5.7.4.2)	

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Job Number: B3215013.00 — Job Name: Union Sanitary Seismic Evaluations — By: CAH — Checked:

ASCE 41-13 Life Safety Structural Checklist for Building Type W1: Wood Light Frames and W1A: Multi-Story, Multi-Unit Residential Wood Frame

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☐	☒	☐	☐	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the following values (Commentary: Sec. A.3.2.7.1. Tier 2: Sec. 5.5.3.1.1) Structural panel sheathing 1,000 plf Diagonal sheathing 700 plf Straight sheathing 100 plf All other conditions 100 plf	**DCR fails @ quickcheck ~180% worst case direction** **May not be a life safety issue with further analysis. Preliminary check indicates walls are very near their capacity.** **The drag connections at the glulams appear overstressed to transfer their tributary mass, so further analysis of the collectors and alternate load paths should be explored.**
☒	☐	☐	☐	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic force-resisting system. (Commentary: Sec. A.3.2.7.2. Tier 2: Sec. 5.5.3.6.1)	**Plywood walls provided.**
☒	☐	☐	☐	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard are not used as shear walls on buildings over one story in height with the exception of the uppermost level of a multi-story building. (Commentary: Sec. A.3.2.7.3. Tier 2: Sec. 5.5.3.6.1)	**Plywood walls provided.**
☒	☐	☐	☐	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces. (Commentary: Sec. A.3.2.7.4. Tier 2: Sec. 5.5.3.6.1)	**Narrow piers at the perimeter of the structure are 4' wide and have a 6' clear height. They also are detailed with holdowns which will increase their effectiveness.**
☒	☐	☐	☐	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor. (Commentary: Sec. A.3.2.7.5. Tier 2: Sec. 5.5.4.6.2)	**1 story building**
☐	☐	☒	☐	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story due to a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1. (Commentary: Sec. A.3.2.7.6. Tier 2: Sec. 5.5.3.6.3)	
☐	☐	☒	☐	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels. (Commentary: Sec. A.3.2.7.7. Tier 2: Sec. 5.5.3.6.4)	

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ASCE 41-13 Life Safety Structural Checklist for Building Type W1: Wood Light Frames and W1A: Multi-Story, Multi-Unit Residential Wood Frame

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
☒	☐	☐	☐	OPENINGS: Walls with openings greater than 80 percent of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces. (Commentary: Sec. A.3.2.7.8. Tier 2 Sec. 5.5.3.6.5)	**No large diaphragm openings**
				Connections	
☒	☐	☐	☐	WOOD POSTS: There is a positive connection of wood posts to the foundation. (Commentary: Sec. A.5.3.3. Tier 2: Sec. 5.7.3.3)	**Holdowns provided at wood posts at wall boundaries.**
☒	☐	☐	☐	WOOD SILLS: All wood sills are bolted to the foundation. (Commentary: Sec. A.5.4.3.4. Tier 2: Sec. 5.7.3.3)	**5/8" sill bolts provided at least @ 4' O.C.**
☒	☐	☐	☐	GIRDER/COLUMN CONNECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1	**Plates and thru bolts provided.**
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Connections	
☒	☐	☐	☐	WOOD SILL BOLTS: Sill bolts are spaced at 6 feet or less with proper edge and end distance provided for wood and concrete. (Commentary: Sec. A.5.3.7 Tier 2: Sec. 5.7.3.3)	**5/8" sill bolts provided at least @ 4' O.C.**
				Diaphragms	
☐	☒	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	**Mansard roof is creates a stepped diaphragm at the roof. The back side has plywood so the load transfer in this direction appears sufficient. In the direction normal to the roof edge, the load has to transfer weak-way through the edge gluelam beams. This should be studied further**
☒	☐	☐	☐	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation. (Commentary: Sec. A.4.1.3 Tier 2: Sec. 5.6.1.1)	**"Drag" connections are detailed at the roof boundaries so the glulams can serve as chords.**
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.1)	

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ASCE 41-13 Life Safety Structural Checklist for Building Type W1: Wood Light Frames and W1A: Multi-Story, Multi-Unit Residential Wood Frame

C	NC	N/A	U		Comments
☒	☐	☐	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☒	☐	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and shall have aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	**Maximum span is ~60'. This should be investigated further but is not likely a life-safety concern provided that the drag connections at the glulam beams are found to be sufficient.**
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**There is a significant amount of piping entering the building without flexible couplings. The adequacy of this piping needs to be evaluated further.** **The below grade portion of the lift station is directly adjacent but separated from the above grade portion. The seismic surcharge from the small reinforced masonry portion of the structure is not judged to be a life-safety hazard with respect to the embedded lift station.**
☐	☐	☒	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☐	☐	☒	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	**1 story**
☐	☐	☒	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	**1 story**
☐	☐	☒	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	**1 story**
☐	☐	☒	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	**1 story**
☐	☐	☒	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	**1 story**
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations | By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U	Item	Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	**A deficiency per ASCE 41 because the shear walls are also bearing walls. Very likely not a deficiency with more advanced analysis. Not judged to be a life-safety issue at this time.**
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**Max stress OK by inspection**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	**#6@12 EW, EF in 12” wall typ.**
				Connections	
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	
☒	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
				HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**	
				Seismic-Force-Resisting System	
☐	☐	☒	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	
☒	☐	☐	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	**Flat slabs terminated into beams.**

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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**	
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☒	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☐	☒	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	**The embedded lift station is a completely open structure except for the landing near the base of the stairs (which itself is open). The minimal diaphragm continuity at the base is very likely not a lift-safety issue with further analysis. Also, the walls did not appear to under structural distress at the time of site visit so they are likely sufficient for additional seismic induced soil pressures. However, this should be studied further to confirm.**
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	

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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations | By: CAH Checked:

ASCE 41-13 RM1 / RM1A Life Safety Structural Checklist: Reinforced Masonry Bearing Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	**Solid walls on all 4 sides.**
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than 70 psi. (Commentary: Sec. A.3.2.4.1. Tier 2: Sec. 5.5.3.1.1)	**In-plane stress in walls ~5 psi max.**
☒	☐	☐	☐	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 in either of the two directions; the spacing of reinforcing steel is less than 48 inches and all vertical bars extend to the top of the walls. (Commentary: Sec. A.3.2.4.2. Tier 2: Sec. 5.5.3.1.3)	**Sum of total reinforcing is 0.0021. However, the spacing of horizontal reinforcing is 48" so this requirement is technically not satisfied. Note that as long as walls are well tied into the roof, the relative lack of reinforcing is not judged to be a life safety issue for such a small structure.**
				Stiff Diaphragms	
☐	☐	☒	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	
				Connections	
☐	☒	☐	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	**Ledger anchors are sufficient to resist the out-of-plane forces with an 80% DCR.** **However, this induces cross-grain tension in the ledger plate. Also out-of-plane action can potentially roll rim joist.** **Preliminary analysis indicates that joists are strong enough for out-of-plane loading if anchors are places @ 4' O.C.**
☐	☒	☐	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	**Cross Grain Tension in Top Plate. Potential Cross Grain Bending (or Rolling) of Rim Joist.**
☒	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	**5/8" bolt @ 32" O.C.**
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.6.4)	

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Building Name: Paseo Padre Lift Station   Date: 6/24/14
Building Address: Page: 2 of 3
Job Number: B3215013.00   Job Name: Union Sanitary Seismic Evaluations   By: CAH   Checked:

ASCE 41-13 RM1 / RM1A Life Safety Structural Checklist: Reinforced Masonry Bearing Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
☐	☐	☒	☐	GIRDER / COLUMN CONNECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

Stiff Diaphragms

C	NC	N/A	U		Comments
☐	☐	☒	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
☐	☐	☒	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 feet long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)	

Flexible Diaphragms

C	NC	N/A	U		Comments
☐	☒	☐	☐	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	**No cross ties in blocking direction. Joists can serve as ties in joist direction.**
☒	☐	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
☒	☐	☐	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 feet long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

Connections

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Building Name: Paseo Padre Lift Station Date: 6/24/14
Building Address: Page: 3 of 3
Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 RM1 / RM1A Life Safety Structural Checklist: Reinforced Masonry Bearing Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
☐	☐	☒	☐	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 inch prior to engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)	**No anchors provided.**

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Building Name: Generator Building #2 Date: 6/24/14

Building Address: Page: 1 of 2

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☒	☐	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**

Building Name: Generator Building #2 Date: 6/24/14

Building Address: Page: 2 of 2

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	

HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**

FOUNDATION CONFIGURATION

C	NC	N/A	U		Comments
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	**Strip footings provided.**

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Building Name: Generator Building #2 Date: 6/24/14

Building Address: Page: 1 of 2

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				Connections	
☐	☐	☐	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1)	**See C2 checklist**
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				Seismic-Force-Resisting System	
☐	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	**See C2 checklist**
☐	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.3.1. Tier 2: Sec. 5.5.3.1.1)	**See C2 checklist**
☐	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.3.2. Tier 2: Sec. 5.5.3.1.3)	**See C2 checklist**
				Diaphragms	
☒	☐	☐	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	**2 ½" topping slab provided**
				Connections	
☐	☐	☒	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	
☐	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)	**See C2 checklist**
☐	☒	☐	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2 Sec. 5.6.1)	**Topping slab not doweled into E-W Running Concrete Walls.**
☐	☐	☒	☐	GIRDER/COLUMN CONECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	

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Building Name: Generator Building #2 Date: 6/24/14
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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Seismic-Force-Resisting System	
☐	☐	☐	☐	DEFLECTION COMPATIBILITY FOR RIGID DIAPHRAGMS: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**See C2 checklist**
☐	☐	☐	☐	WALL OPENINGS: The total width of openings along any perimeter wall line constitute less than 75 percent of the length of any perimeter wall with the wall piers having aspect ratios of less than 2-to-1. (Commentary: Sec. A.3.2.3.3. Tier 2: Sec. 5.5.3.3.1)	**See C2 checklist**
				Diaphragms	
☐	☐	☐	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	**See C2 checklist**
☐	☐	☐	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	**See C2 checklist**
☐	☐	☐	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	**See C2 checklist**
☐	☐	☐	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	**See C2 checklist**
☐	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	**See C2 checklist**
				Connections	
☐	☐	☒	☐	MINIMUM NUMBER OF WALL ANCHORS PER PANEL: There are at least two anchors from each precast wall panel into the diaphragm elements. (Commentary: Sec. A.5.1.3. Tier 2: Sec. 5.7.1.4)	
☐	☐	☒	☐	PRECAST WALL PANELS: Precast wall panels are connected to the foundation. (Commentary: Sec. A.5.3.6 Tier 2: Sec. 5.7.3.4)	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
☐	☐	☒	☐	GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts. (Commentary: Sec. A.5.4.2. Tier 2: Sec. 5.7.4.2)	

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Building Name: **Generator Building #2** Date: **6/24/2014**

Building Address: Page: **1** of **3**

Job Number: **B3215013.00** Job Name: **Union Sanitary Seismic Evaluations** By: **CAH** Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

LOW AND MODERATE SEISMICITY

Seismic-Force-Resisting System

C	NC	N/A	U		Comments
☐	☒	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	**A deficiency per ASCE 41 because the shear walls are also bearing walls. Very likely not a deficiency with more advanced analysis. Not judged to be a life-safety issue at this time.**
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**Max stress in wall ~20 psi.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	**#4 @ 12 E.W., E.F. – p = 0.0028**

Connections

C	NC	N/A	U		Comments
☐	☒	☐	☒	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	**Tier 2 procedure used because diaphragm is rigid. Anchorage and doweling is OK.** **N-S walls are attached to the double t stem which is likely not detailed to transfer out-of-plane loads into the diaphragm.** **Middle wall could cantilever but is attached to bottom stem of t which could be deficient.**
☐	☒	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	**Diaphragms are connected for shear transfer, but this load path is deficient.** **300% DCR in E-W Direction** **150% DCR in N-S Direction**

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Building Name: Generator Building #2 Date: 6/24/2014

Building Address: Page: 2 of 3

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

Seismic-Force-Resisting System

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**Gravity elements typically have shear capacity to develop flexural hinges. They will also be protected by the small deflection of the structure.**
☐	☐	☒	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	
☒	☐	☐	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	**Coupling beams have #3 ties @ 4.5" O.C with 135 degree hooks.**

Connections

C	NC	N/A	U	Item	Comments
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	

Diaphragms (Flexible or Stiff)

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☒	☐	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	

Flexible Diaphragms

C	NC	N/A	U	Item	Comments
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	

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Building Name: Generator Building #2 Date: 6/24/2014
Building Address: Page: 3 of 3
Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

Building Name: **Alvarado Pump Sation** Date: **3/25/14**

Building Address: ______ Page: **1** of **2**

Job Number: **B3215013.00** Job Name: **Union Sanitary Seismic Evaluations** By: **CAH** Checked: ______

ASCE 41-13 Life Safety Basic Configuration Checklist

LOW SEISMICITY

BUILDING SYSTEM

C	NC	N/A	U	Item	Comments
☐	☒	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	**See other checklists for potential deficiencies in the load path.**
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Large tower adjacent to the pump station is supported off one of the corbels of the station. The movement of the structures should be minor but further analysis is warranted.**
☐	☒	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	**New mezzanines are not braced.**

BUILDING CONFIGURATION

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☐	☒	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	**Middle East-West running wall is offset at the both the 2nd and 3rd floors. The wall runs the entire width of the structure so this is very likely not a deficiency with further analysis. The diaphragms should be sufficient to distribute the transfer forces.**
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	

MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)

GEOLOGIC SITE HAZARDS

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C	NC	N/A	U		Comments
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	

HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**

FOUNDATION CONFIGURATION

C	NC	N/A	U		Comments
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				Connections	
☒	☐	☐	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1)	**New anchorage is deficient for North-South seismic. Appears no positive attachment is provided at the West wall. Precast panels are sufficient to span to the columns, which can cantilever and are also supported by new roof framing.**
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.3.1. Tier 2: Sec. 5.5.3.1.1)	**Max stress ~30 psi. Note that precast wall panel bars are weld connected at the base, which creates a brittle connection. Checking this connection as a force controlled action DCR ~ 30%, so there appears to be enough wall provided.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.3.2. Tier 2: Sec. 5.5.3.1.3)	

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C	NC	N/A	U		Comments
				Diaphragms	
☐	☒	☐	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	**New steel bracing diaphragm is only connected at the walls. There is a load path for the out-of-plane wall weight to get into the new steel diaphragm (however the adequacy of this load path requires further analysis), therefore check the diaphragm ties for only the self-weight of the diaphragm @ 60psf.** **Therefore, the precast double-t and the diaphragm ties still have to serve as the mechanism to transfer at least the self-weight of the roof. Based on this analysis, the diaphragm ties are slightly overstressed.** **Further analysis is necessary to confirm that actual accelerations at the roof level and a condition assessment should be conducted to confirm the condition of the diaphragm ties. Based on observations from Primary Clarifiers 1-4, it is possible that the diaphragm ties are corroded and thus largely ineffective.**
				Connections	
☐	☐	☒	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	

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C	NC	N/A	U		Comments
☐	☒	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)	**The retrofit connections at the existing walls appear to be intended for in-plane seismic transfer but are deficient, especially at the North Wall. The new connections provided at the North Wall have a 700% DCR only considering the force from the diaphragm self-weight and the lowest possible roof acceleration. Note that the connection at the middle wall appears to be different from what is shown on the retrofit drawings and requires further study.** **New connections to the East and West walls were only provided at the East wall. From the original drawings the East wall is the end where the girders are positively attached, so there appears to be no positive attachment at the West wall. It is possible that the side where the original positive attachment is located was switched from the original drawings. It is also possible that Carollo Engineers mistook the original drawings. Finally it is possible that Carollo Engineers did not want to lock the beams in at the West wall and intended for the beams to cantilever. Field exploration and testing are necessary to confirm that actual condition. Still, even if the diaphragm were attached at both ends, the load path is almost certainly deficient because it relies on weak-way bending of the precast beam webs which will induce flexure into the thin diaphragm of the double-t beams**
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2 Sec. 5.6.1)	
☐	☐	☒	☐	GIRDER/COLUMN CONECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	**NG at one end of the precast roof beams.**

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

Seismic-Force-Resisting System

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	DEFLECTION COMPATIBILITY FOR RIGID DIAPHRAGMS: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**Checked weak-way for intermediate columns (only columns/direction that will undergo typical column seismic deformation)**
☒	☐	☐	☐	WALL OPENINGS: The total width of openings along any perimeter wall line constitute less than 75 percent of the length of any perimeter wall with the wall piers having aspect ratios of less than 2-to-1. (Commentary: Sec. A.3.2.3.3. Tier 2: Sec. 5.5.3.3.1)	

Diaphragms

C	NC	N/A	U	Item	Comments
☐	☒	☐	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	**Diaphragm is not technically flexible so not technically a deficiency. It is marked here, however, to flag the reliance on the diaphragm ties as chord elements. Further analysis required.**
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

Connections

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	MINIMUM NUMBER OF WALL ANCHORS PER PANEL: There are at least two anchors from each precast wall panel into the diaphragm elements. (Commentary: Sec. A.5.1.3. Tier 2: Sec. 5.7.1.4)	
☐	☐	☒	☐	PRECAST WALL PANELS: Precast wall panels are connected to the foundation. (Commentary: Sec. A.5.3.6 Tier 2: Sec. 5.7.3.4)	

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ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
☐	☐	☒	☐	GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts. (Commentary: Sec. A.5.4.2. Tier 2: Sec. 5.7.4.2)	

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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**See PC1A checklist.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	
				Connections	
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	
☐	☒	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	**See PC1A checklist.**
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**See PC1A checklist.**
☐	☒	☐	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	**Appears that CIP slabs bottom reinforcing is not hooked into walls. Not likely a life-safety issue.**

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**	
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☒	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☐	☒	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	**There are a series of openings on both sides of the second floor diaphragm. Given the inertial loads in the diaphragm, this shouldn't be a life-safety issue for seismic load transfer with additional analysis. However, the slabs tie the walls of the structure together to resist incremental soil pressures during a seismic event and the slab may be overstressed to serve this purpose. Further analysis is recommended.**
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	

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C	NC	N/A	U		Comments
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Building Name: Aeration Tanks 1-4 Date: 3/21/14

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C NC N/A U **Comments**

LOW SEISMICITY

BUILDING SYSTEM

☐ ☒ ☐ ☐ LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)

There is deficient lateral load path for the precast roof elements. For East-West seismic action, all of the load must be taken out at the end the precast beams are attached to. For North-South seismic action, the roof diaphragm must cantilever from the end of attachment. The diaphragm elements are not positively connected to one another, so each double-t precast beam must cantilever its entire length.

While this is not a life-safety hazard, it is possible the beams are damaged to the point where the fall into the tanks. Also localized pounding damage can be expected. The level of expected impact damage, however, is velocity dependent. The actual gap of the pre-cast roof elements to the wall is only 1-2" based on site observations, which will limit velocities of the roof beams after the connections fail. Therefore, with further analysis it may be possible to show that impact related damage will be minimal.

☐ ☒ ☐ ☐ ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)

Aeration basin is connected to lift station #1. With more advanced analysis this is likely not a deficiency, as both structures appear to be stiff structures.

Aeration basin also connected to east aeration blower room, which has a un-retrofitted precast concrete roof that could collapse and damage the aeration basins.

Piping in and out of structure some without flexible couplings. This needs to be studied further.

☒ ☐ ☐ ☐ MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

BUILDING CONFIGURATION

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C	NC	N/A	U		Comments
				LOW SEISMICITY	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☐	☒	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	**No torsion irregularity for the walls. However, for N-S seismic action the center of stiffness for the diaphragm is at the West wall where the diaphragm is positively attached. The center of mass is at the center of the diaphragm, which means there is a 50% difference between the center of mass and center of stiffness.**
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	

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C	NC	N/A	U		Comments
				LOW SEISMICITY	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	**A deficiency per ASCE 41 because the shear walls are also bearing walls. Very likely not a deficiency with more advanced analysis. Not judged a life-safety issue at this time.**
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**Shear stress ~30 psi assuming the in-plane walls can take out the shear (or ~60 psi at middle walls).** **Actual behavior is likely cantilever wall behavior. Based on preliminary analysis the maximum stress load case is ~70 psi in out of plane shear and 85% DCR in flexure.** **Freeboard in the tank is sufficient at a life-safety level.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	
				Connections	
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	**Walls are sufficient to cantilever**
☐	☒	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	**See comment in basic life-safety checklist**
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	

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Building Name:	Aeration Tanks 1-4	Date:	3/21/2014
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By:	CAH	Checked:	

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C NC N/A U **Comments**

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

Seismic-Force-Resisting System

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Building Name:	Aeration Tanks 1-4	Date:	3/21/2014
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By:	CAH	Checked:	

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C NC N/A U — **Comments**

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

C	NC	N/A	U	Item	Comments
☒	☒	☐	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**See calculations which show that typical columns supporting effluent channels can likely develop the flexural capacity of the columns.** **A rigorous analysis of the forces on and the behavior of the influent channel are beyond the scope of this report, and further analysis is required.** **There are 2 load paths for the center influent channel to resist East-West seismic action: the bents that support the channel can cantilever from the foundation, and the two diaphragms that comprise the channel can span the load to the East-West running walls. Preliminary analyses of these actions show that some level of ductility will be required for either load path.** **Our preliminary analysis indicates that diaphragm action is the primary (stiffer) load path. Based this preliminary analysis, a potential deficiency in the load path is the lack of connection between the diaphragms and the East-West running walls. This load transfer requires further study, but will likely require additional doweled connections or steel plate connections. Furthermore, if the diaphragm load path is stiff enough to force double curvature into the bent supports, the bent supports are likely shear controlled, meaning that the diaphragm action must be stiff enough to prevent the columns from drifting far enough to fail. Our preliminary analysis indicates that the diaphragm is stiff enough to force the bents into double curvature, but the deflection of the diaphragms is small enough to protect the columns.**

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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
☐	☐	☒	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☐	☒	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	**See comments RE precast beams @ roof level.** **See comments RE deflection compatibility for the influent channel.**
☒	☐	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☒	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	**See comments RE precast beams @ roof level.**

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments

Seems that no connection is present between double-t precast elements.

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Building Name: Control Building Date: 3/11/14
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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☐	☒	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	**Load path relies on many transfers through diaphragms and many load path elements that were likely never intended to serve as lateral load path elements (e.g. low sloping roof through exterior stucco up to 2nd floor diaphragm; see 4/201). Judged to be non-conforming pending a rigorous tier 2/3 analysis.**
☒	☐	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☐	☒	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	**All 2nd story metal strap walls do not align with walls below. Holdowns etc. from these 2nd floor walls will not develop the strength of the straps.**
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☐	☐	☒	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	**Diaphragm is flexible and there are many walls so no "torsion" deficiency. See W1 checklist for deficiencies related to diaphragm and load path.**

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Building Name: Control Building Date: 3/11/14

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ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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Building Name: Control Building | Date: 3/6/14
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ASCE 41-13 Life Safety Structural Checklist for Building Type W1: Wood Light Frames and W1A: Multi-Story, Multi-Unit Residential Wood Frame

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☐	☒	☐	☐	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the following values (Commentary: Sec. A.3.2.7.1. Tier 2: Sec. 5.5.3.1.1) Structural panel sheathing 1,000 plf Diagonal sheathing 700 plf Straight sheathing 100 plf All other conditions 100 plf	**DCRs for estimated amount of gypboard ~3-5** **DCRs for exterior stucco only ~2-3 (considering 350 plf for stucco and the same quickcheck m-factor of 4.0 which is unconservative)** **DCRs for exterior stucco and interior gypboard ~1.5-2.5** **DCRs for metal strapping only 2-4**
☒	☐	☐	☒	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic force-resisting system. (Commentary: Sec. A.3.2.7.2. Tier 2: Sec. 5.5.3.6.1)	**Conforming because there are metal straps. However, the exterior stucco is much stiffer than the straps and may try to take the majority of the lateral load. Need to confirm that exterior walls are not sheathed with plywood (it does not appear that way from the drawings).**
☒	☐	☐	☐	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard are not used as shear walls on buildings over one story in height with the exception of the uppermost level of a multi-story building. (Commentary: Sec. A.3.2.7.3. Tier 2: Sec. 5.5.3.6.1)	**See above.**
☐	☒	☐	☐	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces. (Commentary: Sec. A.3.2.7.4. Tier 2: Sec. 5.5.3.6.1)	**There are some very short strap walls on the north side of the building.**
☒	☐	☐	☐	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor. (Commentary: Sec. A.3.2.7.5. Tier 2: Sec. 5.5.4.6.2)	**There is a load path out of the 2nd floor transfer walls.**
☐	☐	☒	☐	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story due to a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1. (Commentary: Sec. A.3.2.7.6. Tier 2: Sec. 5.5.3.6.3)	
☐	☐	☒	☐	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels. (Commentary: Sec. A.3.2.7.7. Tier 2: Sec. 5.5.3.6.4)	

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Building Name: Control Building | Date: 3/6/14
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ASCE 41-13 Life Safety Structural Checklist for Building Type W1: Wood Light Frames and W1A: Multi-Story, Multi-Unit Residential Wood Frame

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
☐	☐	☐	☒	OPENINGS: Walls with openings greater than 80 percent of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces. (Commentary: Sec. A.3.2.7.8. Tier 2 Sec. 5.5.3.6.5)	
				Connections	
☒	☐	☐	☐	WOOD POSTS: There is a positive connection of wood posts to the foundation. (Commentary: Sec. A.5.3.3. Tier 2: Sec. 5.7.3.3)	
☒	☒	☐	☐	WOOD SILLS: All wood sills are bolted to the foundation. (Commentary: Sec. A.5.4.3.4. Tier 2: Sec. 5.7.3.3)	**Non strap walls only have shot pins into the foundation. This is likely not a life-safety hazard for the interior gypboard walls, but could be a life-safety hazard if the exterior stucco walls are taking most of the seismic load.**
☒	☐	☐	☐	GIRDER/COLUMN CONNECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Connections	
☒	☒	☐	☐	WOOD SILL BOLTS: Sill bolts are spaced at 6 feet or less with proper edge and end distance provided for wood and concrete. (Commentary: Sec. A.5.3.7 Tier 2: Sec. 5.7.3.3)	**At strap walls this provision is met. At non-strap walls, see above.**
				Diaphragms	
☐	☒	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	**Mansard roof and steps in the diaphragm create discontinuities. The back of the Mansard roof typically has plywood so the roof seems OK in this direction.** **In the direction normal to the edge of the roof, some of the Mansard sections seem like they have to cantilver or span weak way where the roof framing is parallel to the roof edge.**
☐	☒	☐	☐	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation. (Commentary: Sec. A.4.1.3 Tier 2: Sec. 5.6.1.1)	**Chord on the south side of the structure does not appear to be continuous.**

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Building Name: **Control Building** Date: **3/6/14**

Building Address: ______ Page: **3** of **3**

Job Number: **B3215013.00** Job Name: **Union Sanitary Seismic Evaluations** By: **CAH** Checked: ______

ASCE 41-13 Life Safety Structural Checklist for Building Type W1: Wood Light Frames and W1A: Multi-Story, Multi-Unit Residential Wood Frame

C	NC	N/A	U		Comments
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.1)	
☒	☐	☐	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☒	☐	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and shall have aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	**Bridging as shown on plan does not appear to be able to align with panel edges. Spans are generally less than 40 feet but aspect ratios are not always 4:1 and some portions of the diaphragm will try to cantilever.**
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Building Name: **FMC Maintenance Building/Generator Building 1** Date: **3/11/14**

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Job Number: **B3215013.00** Job Name: **Union Sanitary Seismic Evaluations** By: **CAH** Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**There is a significant amount of piping entering the building without flexible couplings. The adequacy of this piping needs to be evaluated further.**
☒	☒	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	**Main mezzanine in shops area is tied into wall 2 on one side and the bottom chord of the truss on the other side. Truss should be evaluated to transfer this load, but trusses appear robust and should be able serve as a transfer.** **Small offices appear to have been constructed after the original construction. These are typically partial height and are not tied into structure. These small offices need further study and are very likely deficient.**
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	

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ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
☐	☐	☒	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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Building Name: FMC Maintenance Building/Generator Building 1 Date: 3/5/14
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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 RM1 / RM1A Life Safety Structural Checklist: Reinforced Masonry Bearing Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than 70 psi. (Commentary: Sec. A.3.2.4.1. Tier 2: Sec. 5.5.3.1.1)	**In-plane stress in walls ~25 psi max.**
☒	☐	☐	☐	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 in either of the two directions; the spacing of reinforcing steel is less than 48 inches and all vertical bars extend to the top of the walls. (Commentary: Sec. A.3.2.4.2. Tier 2: Sec. 5.5.3.1.3)	
				Stiff Diaphragms	
☐	☐	☒	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	
				Connections	
☐	☒	☐	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	**Exterior walls are directly anchored to the trusses/framing beams. The North/South walls meet life safety requirements and the East/West walls are slightly deficient.** **Wall 1 (windows @ shops building) can cantilever to window height. Lintel appears adequately braced.** **Interior walls are well anchored to bracing and/or studs at the roof level. However, load path appears to rely on bending of 10 Ga plates (see 2/172) which is not adequate.**
☐	☐	☒	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	

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Building Name: FMC Maintenance Building/Generator Building 1 Date: 3/5/14

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ASCE 41-13 RM1 / RM1A Life Safety Structural Checklist: Reinforced Masonry Bearing Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
☐	☒	☐	☒	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	**Interior walls rely on strong way bending of 10 Ga plate which may be deficient, but are otherwise well anchored.** **East/West walls have 18 gauge closures to transfer in-plane load, but these rely on 1/8" welds that could have burned through the gauge material. This needs to be investigated further.** **From the drawings (6/172), the North/South walls are only connected at the trusses, with no blocking/closures between them. This is inadequate and could lead to the joists rolling. However, blocking/closures is shown on Section A-A on sheet 169. Further site investigation is necessary to determine the actual condition and determine the condition of the welds.**
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.6.4)	
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
☐	☐	☒	☐	GIRDER / COLUMN CONNECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

Stiff Diaphragms

C	NC	N/A	U		Comments
☐	☐	☒	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
☐	☐	☒	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 feet long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)	

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ASCE 41-13 RM1 / RM1A Life Safety Structural Checklist: Reinforced Masonry Bearing Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				Flexible Diaphragms	
☒	☐	☐	☐	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	**Trusses can serve as cross ties in the short direction. In the long direction walls are anchored directly to steel members which appear to be developed into the diaphragm. Those members and the diagonal roof trusses can form a sub-diaphragm (adequacy needs to be calculated in a tier 2 type analysis). Middle cross walls are not developed into the diaphragm so they technically do not have adequate “cross ties” but this is really covered in a separate checklist item.**
☒	☐	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
☐	☒	☐	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 feet long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)	**Large dormer at south wall does not appear to be connected to wall.**
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☒	☐	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	
				Connections	
☒	☐	☐	☐	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 inch prior to engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)	

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LOW SEISMICITY

BUILDING SYSTEM

C	NC	N/A	U	Item	Comments
☐	☒	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	**See other checklists for potential deficiencies in the load path.**
☒	☐	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	
☐	☐	☒	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	**No mezzanines**

BUILDING CONFIGURATION

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	**The bottom story is stronger.**
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	**The bottom story is stiffer.**
☐	☒	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	**The last 17 feet of the East North-South running wall is discontinuous. The wall runs the entire width of the structure so this is very likely not a deficiency with further analysis. The diaphragms should be sufficient to distribute the transfer forces.**
☐	☒	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	**Length of wall on base story longer than upper story. Likely not a life-safety concern with additional analysis.**
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	**Roof is lighter**
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	

MODERATE SEISMICITY (**Complete the following items in addition to the items for Low Seismicity)**

GEOLOGIC SITE HAZARDS

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C	NC	N/A	U		Comments
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	

HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**

FOUNDATION CONFIGURATION

C	NC	N/A	U		Comments
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				Connections	
☒	☒	☐	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1)	**Original inserts are not sufficient for out-of-plane load. However, wall panels are either sufficient to span horizontally to columns or horizontally to base shear lugs that were provided as a part of the retrofit.** **New retrofit connections at the top of the walls are sufficient to resist the out-of-plane load from the panels but may not be sufficient to resist the in-plane forces that result from the out-of-plane load (shear flow into chord) depending on how much load is transferred into angle chord and how much load directly transfers into plate. Further analysis recommended.**
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.3.1. Tier 2: Sec. 5.5.3.1.1)	**Max stress ~25 psi. Note that precast wall panel bars are weld connected at the base, which creates a brittle connection. This connection has been supplemented by shear blocks. Checking both connections as a force controlled action the max DCR ~ 130%. However, there is some ductility in these connections (and side face blowout of welded inserts is very conservative), so this is not judged to be a life-safety issue.**

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C	NC	N/A	U		Comments
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.3.2. Tier 2: Sec. 5.5.3.1.3)	**@ Precast Panels** **#5 @ 10 Horz. – p = 0.0034** **#6 @ 12 Vert. – p = 0.0041** **Below Grade** **#7 @ 12 E.W. E.F Typ.**
				Diaphragms	
☐	☒	☐	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	**New steel bracing diaphragm is only connected at the walls to take the out-of-plane wall weight. The DCR for diaphragm diagonals is ~70-100% at a force controlled level. Further analysis of all connections and members is warranted.** **Precast double-t and the diaphragm ties have to serve as the mechanism to transfer the self-weight of the roof. Note that the diaphragm is not tied into the wall at Grid C, so the diaphragm ties along this wall have to transfer shear and moment. Based on this analysis, the diaphragm ties are slightly overstressed to span the load over the wall on gridline C.** **The connection between the double t's is not indicated on the drawings, but for the diaphragms to act together (VQ/I) for E-W seismic, additional shear transfer will be required even if there are diaphragm ties. The double t-s are sufficient to span out-of-plane without interconnectivity.**

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C	NC	N/A	U		Comments
				Connections	
☐	☐	☒	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	
☐	☒	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)	**There is no connection of the precast diaphragm to the wall on gridline C.** **There is no connection of the precast diaphragm to the walls on gridlines 1 and 2 (bond breaker provided where ts cross exterior columns. Note that the beams can act as struts, transfer through the eve beams and transfer through the diaphragm bracing, but this load path appears slightly deficient. It may be possible that this load path is sufficient if the diaphragm is tied into the center wall for out-of-plane load transfer.**
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2 Sec. 5.6.1)	
☐	☐	☒	☐	GIRDER/COLUMN CONECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	**NG at seat on wall at gridline C.**

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

C	NC	N/A	U		Comments
				Seismic-Force-Resisting System	
☒	☐	☐	☐	DEFLECTION COMPATIBILITY FOR RIGID DIAPHRAGMS: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**Columns will be protected from excessive deformation by added steel grid.**
☒	☐	☐	☐	WALL OPENINGS: The total width of openings along any perimeter wall line constitute less than 75 percent of the length of any perimeter wall with the wall piers having aspect ratios of less than 2-to-1. (Commentary: Sec. A.3.2.3.3. Tier 2: Sec. 5.5.3.3.1)	
				Diaphragms	

ASCE 41-13 PC1 / PC1A Life Safety Structural Checklist: Precast/ Tilt-up Concrete Shear Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
☐	☒	☐	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	**Diaphragm is not technically flexible so not technically a deficiency. It is marked here, however, to flag the reliance on the diaphragm ties as chord elements. Further analysis required.**
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	
				Connections	
☒	☐	☐	☐	MINIMUM NUMBER OF WALL ANCHORS PER PANEL: There are at least two anchors from each precast wall panel into the diaphragm elements. (Commentary: Sec. A.5.1.3. Tier 2: Sec. 5.7.1.4)	**When horizontal spanning action considered this is compliant.**
☐	☐	☒	☐	PRECAST WALL PANELS: Precast wall panels are connected to the foundation. (Commentary: Sec. A.5.3.6 Tier 2: Sec. 5.7.3.4)	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
☐	☐	☒	☐	GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts. (Commentary: Sec. A.5.4.2. Tier 2: Sec. 5.7.4.2)	

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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**See PC1A checklist.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	**See PC1A checklist.**
				Connections	
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	
☒	☒	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	**See PC1A checklist.** **Compliant for cast-in-place portion of the structure**
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
				HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**See PC1A checklist.**

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C	NC	N/A	U		Comments
				HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**	
☐	☒	☐	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	**Appears that CIP slabs bottom reinforcing is not hooked into walls. Not likely a life-safety issue.**
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☒	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☐	☒	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	**There are a series of openings on both sides of the second floor diaphragm. Given the inertial loads in the diaphragm, this shouldn't be a life-safety issue for seismic load transfer with additional analysis.**
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	

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Building Name: EBDA Pump Station Date: 6/26/2014

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Building Name: Covered Storage | Date: 3/11/14

Building Address: | Page: 1 of 2

Job Number: B3215013.00 | Job Name: Union Sanitary Seismic Evaluations | By: CAH | Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**At CMU structure between gridlines 1 and 2, only ~1 ½" separation provided between CMU wall and building column on gridline B. Expected drift at middle column line is ~3+" in each direction so pounding is expected. Also, column goes through diaphragm on mezzanine, so it is likely that column will get hung up on the diaphragm.**
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☐	☒	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	**Diaphragm is flexible so torsion deficiencies do not technically apply. However, CMU walls are MUCH stiffer than cantilever column system. Therefore, if columns get hung up on diaphragm large differential deformations will be induced in the roofing members, which could lead to damage.**

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Building Name: Covered Storage		Date: 3/11/14	
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Job Number: B3215013.00	Job Name: Union Sanitary Seismic Evaluations	By: CAH	Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	**Poll foundations are doweled into the slab-on-grade.**

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Building Name: Covered Storage | Date: 3/10/14
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Job Number: B3215013.00 | Job Name: Union Sanitary Seismic Evaluations | By: CAH | Checked:

ASCE 41-13 S1 / S1A Life Safety Structural Checklist: Steel Moment Frames with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.5.3.1, is less than 0.025. (Commentary: Sec. A.3.1.3.1. Tier 2: Sec. 5.5.2.1.2)	**Drift ratio is very close to 2.5%. Pounding with mezzanine is possible.**
☒	☐	☐	☐	AXIAL STRESS CHECK: The axial stress due to gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress due to overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than $0.30F_y$. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3)	
☒	☐	☐	☐	FLEXURAL STRESS CHECK: The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.5.3.9 is less than F_y. Columns need not be checked if the Strong Column/Weak beam checklist item is Compliant. (Commentary: Sec. A.3.1.3.3. Tier 2: Sec. 5.5.2.1.5)	**Average stress among all columns is less than Fy, but middle row of columns may be slightly overstressed in E-W seismic action.**
				Connections	
☐	☒	☐	☐	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames. (Commentary: Sec. A.5.2.2. Tier 2: Sec. 5.7.1)	**The rounded corrugated metal panel roof will not work well as a diaphragm. However, the diaphragm shears are very small. For the typical 25' span in N-S seismic action the diaphragm shear is ~125 plf. Similar 24 Ga roofs with flat edges (e.g. Verco "Vercor") have allowable diaphragm shears approaching 150 plf depending on the method of attachment. Further analysis is recommended, and it is necessary to assess the method of connecting the deck to the steel supports, as this is not shown in the drawings.** **If the deck fails the, diaphragm action in the N-S direction will rely on channels spanning weak-way, and channels are susceptible to rolling at truss locations between blocking members (see figure).**
☒	☐	☐	☐	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3.1)	

MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)

Seismic-Force-Resisting System

ASCE 41-13 S1 / S1A Life Safety Structural Checklist: Steel Moment Frames with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
☒	☐	☐	☐	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. The number of bays of moment frames in each line is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1)	
☐	☒	☐	☐	INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements. (Commentary: Sec. A.3.1.2.1. Tier 2: Sec. 5.5.2.1.1)	
☐	☒	☐	☒	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members based on the specified minimum yield stress of steel. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1). Note more restrictive requirements for High Seismicity.	**Base connection is not capable of developing moment strength of tube but it may not have to if load is adequately transferred to concrete pedestal. Poll foundation capacity to develop tube depends on allowable bearing pressure but does not work based on default values. Unclear if concrete pedestals are doweled into foundation or are positively connected to cantilever columns. This needs to be confirmed through testing.**

HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**

Seismic-Force-Resisting System

C	NC	N/A	U		Comments
☐	☒	☐	☒	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110 percent of the expected yield stress of the steel per AISC 341 Section A3.2. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1)	**See above.**
☐	☐	☒	☐	PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column. (Commentary: Sec. A.3.1.3.5. Tier 2: Sec. 5.5.2.2.2)	
☐	☐	☒	☐	COLUMN SPLICES: All column splice details located in moment-resisting frames include connection of both flanges and the web. (Commentary: Sec. A.3.1.3.6. Tier 2: Sec. 5.5.2.2.3)	
☐	☐	☒	☐	STRONG COLUMN / WEAK BEAM: The percentage of strong column / weak beam joints in each story of each line of moment frames is greater than 50 percent. (Commentary: Sec. A.3.1.3.7. Tier 2: Sec. 5.5.2.1.5)	
☐	☒	☐	☐	COMPACT MEMBERS: All frame elements meet section requirements set forth by AISC 341 Table D1.1 for moderately ductile members. (Commentary: Sec. A.3.1.3.8. Tier 2: Sec. 5.5.2.2.4)	

Diaphragms (Stiff or Flexible)

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Job Number: B3215013.00 | Job Name: Union Sanitary Seismic Evaluations | By: CAH | Checked:

ASCE 41-13 S1 / S1A Life Safety Structural Checklist: Steel Moment Frames with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
☒	☐	☐	☐	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 25 percent of the total frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3)	
				Flexible Diaphragms	
☒	☐	☐	☐	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	**Corrugated metal panels**
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☒	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Building Name: Covered Storage Date: 3/10/14

Building Address: Page: 1 of 3

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 RM1 / RM1A Life Safety Structural Checklist: Reinforced Masonry Bearing Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than 70 psi. (Commentary: Sec. A.3.2.4.1. Tier 2: Sec. 5.5.3.1.1)	**For CMU portion of the structure this is OK by inspection. No calculation required.**
☒	☐	☐	☐	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 in either of the two directions; the spacing of reinforcing steel is less than 48 inches and all vertical bars extend to the top of the walls. (Commentary: Sec. A.3.2.4.2. Tier 2: Sec. 5.5.3.1.3)	
				Stiff Diaphragms	
☐	☐	☒	☐	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)	
				Connections	
☐	☐	☒	☐	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	**Walls are sufficient to cantilever and/or span horizontally to cross walls.**
☐	☒	☒	☐	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)	**N/A because the walls can span horizontally or cantilever. Joists are face mounted but as soon as nails withdraw the ledger would be subject to cross-grain bending so also NC.**
☒	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	
☐	☐	☒	☐	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.6.4)	
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
☐	☐	☒	☐	GIRDER / COLUMN CONNECTION: There is a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)	

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 RM1 / RM1A Life Safety Structural Checklist: Reinforced Masonry Bearing Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**	
				Stiff Diaphragms	
☐	☐	☒	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
☐	☐	☒	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 feet long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)	
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	**N/A because walls can cantilever or span horizontally.**
☒	☐	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
☒	☐	☐	☐	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 feet long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Building Name: Covered Storage | Date: 3/10/14
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Job Number: B3215013.00 | Job Name: Union Sanitary Seismic Evaluations | By: CAH | Checked:

ASCE 41-13 RM1 / RM1A Life Safety Structural Checklist: Reinforced Masonry Bearing Walls with Flexible or Stiff Diaphragms

C	NC	N/A	U		Comments
				Connections	
☐	☐	☒	☐	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 inch prior to engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)	**N/A because walls can cantilever or span horizontally.**

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Building Name: Primary Digester #5 Date: 3/20/14

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Primary Digester #5 has piping going to Heat Mix Building #3. The seismic movement of Digester #5 should be minimal, but piping connections and potential movement of Primary Digester #5 need to be evaluated further.**
☐	☒	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	**Not a “mezzanine” structure, but the anchorage of the center pipe is deficient.**
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	

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Building Name: Primary Digester #5 Date: 3/20/14

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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Building Name: Primary Digester #5 Date: 3/20/2014

Building Address: Page: 1 of 3

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	**A deficiency per ASCE 41 because the shear walls are also bearing walls. Very likely not a deficiency with more advanced analysis. Not judged a life-safety issue at this time.**
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☒	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**Shear stress ~105 psi. Based on preliminary analysis, overall moment on wall OK.** **Based on wall analysis digester #5 slightly exceeds its cracking hoop stress under a seismic event. However, the transverse reinforcement is likely sufficient to limit this cracking and prevent failure of the tank.** **Based on 3/28 site visit, there are large vertical cracks that have been repaired. These represent weak points where the tank is likely to crack in a seismic event. Furthermore, if the rebar was corroded when these cracks initially formed, it is possible that the rebar will be overstressed in a seismic event.** **Freeboard in the tank is not sufficient at a life-safety level.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	
				Connections	
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	**Not dependent for lateral support because circular walls.**

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Building Name: Primary Digester #5 Date: 3/20/2014
Building Address: Page: 2 of 3
Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
☒	☐	☐	☒	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	**See calculation. Based on this analysis and assuming the HSS is in good condition, the HSS and anchorage appear sufficient. However, based on 3/28 site visit, the anchorage attachments are significantly corroded, and therefore may not be able to withstand the additional stresses from a seismic event.**
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	

HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**

Seismic-Force-Resisting System

C	NC	N/A	U		Comments
☐	☐	☐	☒	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**The stability of the roof appears, in part, to be reliant on the solid bearing between the thrust apparatus at the edge of the roof and the inside face of the concrete wall. It is unknown how this connection will respond to either sloshing fluid, or deformations to the circular shape of the tank. This is not expected to be a life-safety concern because, in the case of vertical pressures due to sloshing, the roof is already under positive pressure from the methane gasses. Likewise, the tank walls are already under pressure from the hydrostatic weight of the fluid in the tank, and while the seismic forces nearly double the load on the tank walls, it seems unlikely that this added deformation will cause the roof to completely fail. However, further analysis is recommended.**
☐	☐	☒	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	

Building Name: Primary Digester #5 | Date: 3/20/2014

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Job Number: B3215013.00 | Job Name: Union Sanitary Seismic Evaluations | By: CAH | Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☐	☐	☐	☒	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☒	☐	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☒	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Building Name: Secondary Digester #1 Date: 4/3/14
Building Address: Page: 1 of 2
Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Lots of piping going into Secondary Digester #1 some without flexible couplings. The seismic movement of Digester #1 should be minimal, but piping connections and potential movement of Secondary Digester #1 need to be evaluated further.**
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	

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Building Name: Secondary Digester #1 Date: 4/3/14

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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Building Name: Secondary Digester #1 Date: 4/3/2014

Building Address: Page: 1 of 4

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	**A deficiency per ASCE 41 because the shear walls are also bearing walls. Very likely not a deficiency with more advanced analysis. Not judged a life-safety issue at this time.**
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☒	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**Shear stress ~110 psi. Based on preliminary analysis, overall moment on wall OK.** **Based on wall analysis digester #1 below cracking stress in the considered seismic event.** **Based on 3/28 site visit, there is a large horizontal crack about mid-height in the exposed portion of the tank. This crack has since been repaired. While the stresses are low enough this high on the tank, similar corrosion/cracking could be a concern lower in the tank, and further exploration and analysis is recommended.** **Freeboard in the tank is sufficient at a life-safety level.** **Note that the fluid height in Secondary Digester #1 varies, and the preliminary analysis assumed a fluid height of 35 feet above the base. This fluid height is significantly lower than the maximum fluid height indicated on the drawings, but matches the maximum fluid height reached in the digester during the month of March 2014. Because the probability of a significant seismic event occurring at the same time as the maximum fluid design height is very small, it is judged that designing for the maximum height at a particular month is sufficiently conservative.**

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Building Name: Secondary Digester #1 | Date: 4/3/2014
Building Address: | Page: 2 of 4
Job Number: B3215013.00 | Job Name: Union Sanitary Seismic Evaluations | By: CAH | Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	
				Connections	
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	**Not dependent for lateral support because circular walls.**
☐	☒	☐	☒	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	**See calculation. Based on this analysis and assuming the anchorage bolts are sufficiently torqued to prevent slippage, the anchor bolts could bend if the base plate slips relative to the grout pad below. In this case, the anchorage would be insufficient. More likely is that the bolts slip, which means that because the connection is slotted in both directions there is no mechanical means of transferring the seismic shear of the roof.** **Therefore, the roof is likely to push against the float control basin on one side and pull against it on the other side. This involves a bending of the side sheet plate, and interplay between the seismic reactions and the internal gas pressure. Further study on this mechanism is recommended.** **Additionally, anchorage is found to be overstressed for design internal pressure shown in the dome shop drawings.**
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U	Item	Comments
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Seismic-Force-Resisting System	
☐	☐	☐	☒	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**See transfer to shear walls comment.**
☐	☐	☒	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☐	☐	☐	☒	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☒	☐	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☒	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	**Solid welded plate diaphragm is very likely OK, but further analysis suggested to confirm it will not buckle.**

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Building Name: Secondary Digester #1 Date: 4/3/2014

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments

Building Name: EBDA Effluent Surge Tower | Date: 3/18/14
Building Address: | Page: 1 of 2
Job Number: B3215013.00 | Job Name: Union Sanitary Seismic Evaluations | By: CAH | Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Large pipe through the wall near its base. The structure is flexible but the foundation may rock**
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement. Although differential settlement is expected to be small and not expected to pose a serious life-safety hazard, this should be studied in greater detail for the Surge Tower because of its high height-to-footprint ratio.**

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Building Name: **EBDA Effluent Surge Tower** Date: **3/18/14**

Building Address: Page: **2** of **2**

Job Number: **B3215013.00** Job Name: **Union Sanitary Seismic Evaluations** By: **CAH** Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	

HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**

FOUNDATION CONFIGURATION

C	NC	N/A	U		Comments
☐	☒	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	**Based on preliminary analysis rocking is possible but may be OK if more advanced analysis is conducted. Foundation may have strength issues in a rocking event, as there is 15' of soil above foundation and top reinforcing is relatively light.**
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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Building Name: EBDA Effluent Surge Tower | Date: 3/18/2014

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Job Number: B3215013.00 | Job Name: Union Sanitary Seismic Evaluations | By: CAH | Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	**A deficiency per ASCE 41 because the shear walls are also bearing walls. Very likely not a deficiency with more advanced analysis. Not judged a life-safety issue at this time.**
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☐	☒	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**Deficiency depends on water level and specific consultation from a geotechnical engineer.** **Stress ~180 psi when water level is at the height of the grating and ~130 psi when the water level is at half height. A finite element analysis considering the large (5' diameter) pipe opening near the base of the wall shows the true max shear stress is closer to 300-400 psi, which is highly stressed even when the contribution from the rebar is considered (note the horizontal rebar also has to resist hoop stresses). This analysis also does not consider the potential beneficial effect of the passive pressure of the surrounding soil (~15 feet from the base). Further analysis that incorporates the specific recommendations of a geotechnical engineer is recommended.** **Detailed flexural analysis indicates the wall has enough vertical reinforcing to resist the overturning moment.** **Based on preliminary analysis hoop stress in concrete walls and freeboard meet life-safety requirements.**

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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	
				Connections	
☒	☐	☐	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	**Circular and small diameter/thickness ratio → will not rely on grating/grating support beams for out of plane support**
☐	☒	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	**Beams have horizontal slotted holes → no means of shear transfer in the direction longitudinal to the beams because the grating is also not connected to the edge angle. In the direction transverse to the beams, the beams and their anchorage are OK to serve as diaphragm based on preliminary analysis.**
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Seismic-Force-Resisting System	
☐	☐	☒	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	
☐	☐	☒	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	

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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				Diaphragms (Flexible or Stiff)	
☒	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☐	☐	☒	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☒	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	**Diaphragm relies on beams weak-way or grating (although grating is not tied into walls and is in multiple pieces so it cannot serve as a diaphragm). Can likely justify beams weak-way, but need strong way connection.**

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C NC N/A U Comments

Note: From 3/18/14 site visit, there is substantial cracking in the walls of thickener 1, particularly at the base of the wall. From the calculations contained in this report, this cracking does not appear to be the result of structural distress.

In 2004, a project was done to repair the cracking at the Thickeners. The repair to the concrete walls was limited to repairing the concrete and coating the rebar. This is an indication that the rebar was not significantly damaged. Additionally, the nature of the damage, the pattern of the cracking and corrosion of the concrete indicate that the cracking and damage was likely the result of the corrosive environment of the tank and not the result of structural distress. However, further study as to the condition of the reinforcing at the thickener may be warranted. Furthermore, the preliminary analysis that was conducted as part of this study assumed the tank walls are in good condition. If the concrete continues to spall/corrode and the rebar is found to be corroded, the seismic performance of the tank may be deficient.

LOW SEISMICITY

BUILDING SYSTEM

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Thickener #1 has piping going to the Thickener Control Building. The seismic movement of Thickener #1 should be minimal, but piping connections and potential movement of Thickener Control Building need to be evaluated further. Walkway leading to the thickener appears to be separated by a small caulked joint, but there may be dowels at this location. This should also be investigated further.** **Note that the center rake mechanism and associated piping were not specifically considered as part of this study and further analysis may be warranted .**
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	

BUILDING CONFIGURATION

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	

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C	NC	N/A	U		Comments
				LOW SEISMICITY	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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LOW AND MODERATE SEISMICITY

Seismic-Force-Resisting System

C	NC	N/A	U	Item	Comments
☐	☒	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	**A deficiency per ASCE 41 because the shear walls are also bearing walls. Very likely not a deficiency with more advanced analysis. Not judged a life-safety issue at this time.**
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**Stress ~75 psi. Based on wall analysis thickener #1 is also meets lift-safety requirements for freeboard, hoop stress, and overall wall moment.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	

Connections

C	NC	N/A	U	Item	Comments
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	
☒	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	**Diaphragm shear transfer meets life-safety requirements based on analysis.**
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

Seismic-Force-Resisting System

C	NC	N/A	U	Item	Comments
☒	☐	☐	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	
☐	☒	☐	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	**Roof bottom bars are not hooked into wall. Beam bars are not hooked into pilaster but are at least partially developed. Not likely to be a life-safety issue.**

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C	NC	N/A	U	Item	Comments
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☒	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☐	☒	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	**Openings comprise ~35% of wall length. However, based on the shear transfer DCR, this is not likely to be a deficiency with more advanced analysis.**
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☐	☒	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	**The lateral load path, particularly for the large mass transfer of the pumps, requires further analysis and an accurate estimation of the mass of the pumps.** **The top roof slab must support part of the weight of the pumps, in addition to its own self weight, but is not directly connected along its length to the east-west running walls. There are drag bars to the far south wall (appear to be 2 #5) but otherwise there are no special defined drag elements in the slab.** **It is very possible that back 15” wall can cantilever some of the load to the 2nd floor diaphragm, but this should be confirmed with a tier 2 type analysis. Even so, this is a much more flexible load path than loading the cross wall(s) directly and could lead to localized diaphragm damage.** **Also, short roof diaphragms must cantilever for North-South seismic action. This is likely OK with further analysis, but needs further study because of the large mass of the pumps. Cantilever reactions are taken out by small amount of reinforcing to cross walls.**
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Lift station 1 is physically connected to aeration basins 1-4. With more advanced analysis this is likely not a deficiency, as both structures appear to be stiff structures.** **Lots of piping in and out of structure without flexible couplings. This needs to be studied further.**
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	

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C	NC	N/A	U		Comments
				LOW SEISMICITY	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☐	☒	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	**Force that is taken by the middle fin may have to go through the diaphragm to the end walls. This is likely not a deficiency with further analysis.**
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	**A deficiency per ASCE 41 because the shear walls are also bearing walls. Very likely not a deficiency with more advanced analysis. Not judged a life-safety issue at this time.**
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**Max In-Plane Stress ~40 psi. Note that this analysis does not consider mass of fluid in the channels. Also assumes 200 psf equivalent weight for large pump units.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	
				Connections	
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	
☐	☒	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	**Diaphragms are connected, but may not be sufficiently connected. See comments RE load path.**
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	

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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U	Item	Comments
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Seismic-Force-Resisting System	
☒	☒	☐	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**Columns (see sheet 108) have the shear strength to develop the flexural strength of the column provided that concrete capacity can be relied upon for shear capacity. Also columns will be protected from excessive drifts with this very stiff embedded structure. However, because the columns carry very heavy gravity loads and because tie spacing is large this cannot be dismissed as a potential life-safety issue without further analysis.**
☐	☒	☐	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	**Slab bottom bars are not hooked into walls. This is very likely not a life-safety issue with further analysis.**
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☐	☒	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	**See comments RE load path**
☐	☒	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	**See comments RE load path. Openings on slabs adjacent to end walls.**
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	

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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Piping and walkway to the building with no seismic joints. Movement of the structure should be minor but the adequacy of the piping needs to be investigated further.**
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**

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C	NC	N/A	U		Comments
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

FOUNDATION CONFIGURATION

C	NC	N/A	U		Comments
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	**A deficiency per ASCE 41 because the shear walls are also bearing walls. Very likely not a deficiency with more advanced analysis. Not judged to be a life-safety issue at this time.**
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**Max stress in wall ~50 psi.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	
				Connections	
☒	☐	☐	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	**Pre-cast panels were placed in the open portions of the structure on the east and west side during the 1978 phase of construction. The 1978 drawings indicate that the panels are dowelled into the “existing concrete” (we take this to mean into the top spandrel above and concrete foundation below), and they indicate that the jamb bars are dowelled top and bottom.**
☒	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	

HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**

Seismic-Force-Resisting System

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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U	Item	Comments
				HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**	
☒	☐	☐	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	
☐	☒	☐	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	**Bottom slab reinforcing is not hooked into the wall. While this is technically a deficiency, it is likely not a life-safety hazard with further analysis.**
☐	☒	☐	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	**Coupling beams (above what are now precast panels) are likely flexurally controlled and are not likely a life-safety hazard with further analysis.**
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☐	☒	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	**Small low diaphragm sections above pre-cast panels are not judged to be a life-safety hazard.**
☒	☐	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	

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C	NC	N/A	U		Comments
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Building Name: Chlorine Contact Tank Date: 3/18/14

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**Chlorine contact tank is physically connected to EBDA pump station and secondary clarifiers. With more advanced analysis this is likely not a deficiency, as all 3 structures are stiff structures.** **Note that EBDA pump station roof is a precast roof which as originally constructed was very likely deficient and had the possibility of collapsing and damaging the chlorine contact tank. This roof has been retrofit after the original construction so this risk has likely been mitigated. However, specific study of the EBDA pump station and the retrofit work is beyond the scope of this report, and further study is necessary to confirm this potential hazard.**
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	

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C	NC	N/A	U		Comments
				LOW SEISMICITY	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☐	☒	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**Stress ~190 psi. Per ACI 350.3 S5.2.1, "in-plane" walls checked as shearwalls for entire base shear. However, based on aspect ratio, walls will try to cantilever.** **Max out-of-planes shear stress on cantilever wall ~50 psi and walls are strong enough to cantilever. Therefore, "in-plane" shear stress deficiency is likely not a life safety hazard.** **Freeboard is sufficient at a life-safety level.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	
				Connections	
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	
☐	☐	☒	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
				HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**	
				Seismic-Force-Resisting System	

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C	NC	N/A	U		Comments
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
☐	☐	☒	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	
☐	☐	☒	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☐	☐	☒	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☐	☐	☒	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Building Name: Secondary Clarifiers 1-4 Date: 3/25/14

Building Address: Page: 1 of 2

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**The secondary clarifiers are physically connected to EBDA pump station, chlorine contact tank, Sludge Pump Room #2 and Control Box #3. With more advanced analysis this is likely not a deficiency, as all 5 structures are stiff structures.** **Control Box #3 and Sludge Pump Room #2 have precast concrete roofs which are almost certainly deficient.** **The EBDA pump station roof is a precast roof which as originally constructed was very likely deficient and had the possibility of collapsing and damaging the clarifiers. This roof has been retrofit after the original construction so this risk has likely been mitigated. However, specific study of the EBDA pump station and the retrofit work is beyond the scope of this report, and further study is necessary to confirm this potential hazard.**
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	

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Building Name: Secondary Clarifiers 1-4 Date: 3/25/14
Building Address: Page: 2 of 2
Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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Building Name: Secondary Clarifiers 1-4 Date: 3/25/2014
Building Address: Page: 1 of 2
Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**Shear stress ~30 psi assuming the in-plane walls can take out the shear.** **Max out-of plane shear stress on a cantilever wall (more likely load path) is ~70 psi. Walls have sufficient flexural capacity to cantilever.** **Freeboard in the tank is sufficient at a life-safety level, but the DCR is approaching 1. Therefore, there is the potential for spillage in a major seismic event.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	
				Connections	
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	
☐	☐	☒	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	

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Building Name: Secondary Clarifiers 1-4 Date: 3/25/2014
Building Address: Page: 2 of 2
Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**	
				Seismic-Force-Resisting System	
☐	☐	☒	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	
☐	☐	☒	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☐	☐	☒	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☐	☐	☒	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Building Name: Control Box 3 Date: 6/27/14

Building Address: Page: 1 of 2

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	**See other checklists for potential deficiencies in the load path.**
☒	☐	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☐	☐	☒	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	**One story structure.**
☐	☐	☒	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	**One story structure.**
☐	☐	☒	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	**One story structure.**
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	**One story structure.**
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	**One story structure.**
☐	☒	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	**The series of cross walls are interrupted by a series of large diameter pipes that run in the long direction of the structure. Consequently, there are large openings in these walls, with the furthest east wall being completely interrupted at the base by pipes. This is not expected to be a life-safety concern with additional analysis because the structure is nearly completely embedded.**

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Building Name: Control Box 3 Date: 6/27/14

Building Address: Page: 2 of 2

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				FOUNDATION CONFIGURATION	
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	**l/h ~ 0.62 < 0.6Sa = 0.65 but not judged to be a life-safety deficiency because structure is embedded.**
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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Building Name: Control Box 3 Date: 6/27/2014

Building Address: Page: 1 of 2

Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**15 psi max.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	**#7 @ 12 E.W. E.F Typ.**
				Connections	
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	**Diaphragm at ground floor level is rigid.**
☒	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	**Diaphragm connected to walls with dowels**
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Seismic-Force-Resisting System	
☐	☐	☒	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	**There are no secondary components.**
☐	☒	☐	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	**Appears that CIP slabs bottom reinforcing is not hooked into walls. Not likely a life-safety issue.**

Building Name: Control Box 3 Date: 6/27/2014
Building Address: Page: 2 of 2
Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☒	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☐	☒	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	**There are a series of openings on the ground floor diaphragm. Given the inertial loads in the diaphragm, this shouldn't be a life-safety issue for seismic load transfer with additional analysis.**
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Building Name: **Avarado WWTP Force Main Influent Valve Vault** Date: **3/21/14**

Building Address: ______ Page: **1** of **2**

Job Number: **B3215013.00** Job Name: **Union Sanitary Seismic Evaluations** By: **CAH** Checked: ______

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☐	☒	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	**There is lots of piping entering the structure which appears to not have any seismic joints. Movement of the structure should be minor but the adequacy of the piping needs to be investigated further.** **Structure is also 1” from adjacent vent structure.**
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building’s seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**

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Building Name: **Avarado WWTP Force Main Influent Valve Vault** Date: **3/21/14**

Building Address: ______ Page: **2** of **2**

Job Number: **B3215013.00** Job Name: **Union Sanitary Seismic Evaluations** By: **CAH** Checked: ______

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	

HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)

FOUNDATION CONFIGURATION

C	NC	N/A	U		Comments
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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Building Name: **Avarado WWTP Force Main Influent Valve Vault** Date: **3/21/2014**

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Job Number: **B3215013.00** Job Name: **Union Sanitary Seismic Evaluations** By: **CAH** Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	**A deficiency per ASCE 41 because the shear walls are also bearing walls. Very likely not a deficiency with more advanced analysis. Not judged to be a life-safety issue at this time.**
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**Max stress in wall ~20 psi.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	
				Connections	
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	
☒	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Seismic-Force-Resisting System	
☐	☐	☒	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	

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Building Name: Avarado WWTP Force Main Influent Valve Vault | Date: 3/21/2014

Building Address: | Page: 2 of 3

Job Number: B3215013.00 | Job Name: Union Sanitary Seismic Evaluations | By: CAH | Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**	
☐	☒	☐	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	**Appears that roof slab reinforcing is hooked into wall. 2nd floor slab reinforcing appears stopped at wall. Not likely a deficiency with additional analysis.**
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☒	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☐	☒	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	**Openings in roof diaphragm are slightly more than 25%. Given the wall stresses this is very likely not a deficiency with additional analysis.**
				Flexible Diaphragms	
☐	☐	☒	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	

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Building Name: Avarado WWTP Force Main Influent Valve Vault | Date: 3/21/2014
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ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Building Name: Main Electrical Distribution		Date: 3/11/14	
Building Address:		Page: 1	of 2
Job Number: B3215013.00	Job Name: Union Sanitary Seismic Evaluations	By: CAH	Checked:

ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
				LOW SEISMICITY	
				BUILDING SYSTEM	
☒	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)	
☒	☐	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4 percent of the height of the shorter building. This statement shall not apply to the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	
☒	☐	☐	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure, or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)	
				BUILDING CONFIGURATION	
☒	☐	☐	☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)	
☒	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)	
☒	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)	
☒	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)	
☒	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)	
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)	
				MODERATE SEISMICITY (Complete the following items in addition to the items for Low Seismicity)	
				GEOLOGIC SITE HAZARDS	
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building. (Commentary: Sec. A.6.1.1)	**Possible liquefaction induced settlement, but unlikely a life-safety hazard with further analysis.**

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Building Name: Main Electrical Distribution | Date: 3/11/14
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ASCE 41-13 Life Safety Basic Configuration Checklist

C	NC	N/A	U		Comments
☒	☐	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2)	
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Commentary: Sec. A.6.1.3)	

HIGH SEISMICITY **(Complete the following items in addition to the items for Low and Moderate Seismicity)**

FOUNDATION CONFIGURATION

C	NC	N/A	U		Comments
☒	☐	☐	☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1 Tier 2: Sec. 5.4.3.3)	
☒	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)	

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Building Name: Main Electrical Distribution Date: 3/11/14

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Job Number: B3215013.00 Job Name: Union Sanitary Seismic Evaluations By: CAH Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				LOW AND MODERATE SEISMICITY	
				Seismic-Force-Resisting System	
☐	☒	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5)	**A deficiency per ASCE 41 because the shear walls are also bearing walls. Very likely not a deficiency with more advanced analysis. Not judged a life-safety issue at this time.**
☒	☐	☐	☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.1.1)	
☒	☐	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 psi or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	**Max stress in any wall ~15 psi. Very low in-plane shear stress. Walls also have sufficient out-of-plane strength to span from foundation to roof level based on Tier 2 type analysis.**
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	
				Connections	
☒	☐	☐	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	**Wall anchorage is robust, but has a 107% DCR based on quick check procedure. Judged to meet life-safety with this slight overstress.**
☒	☐	☐	☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: 5.7.2)	**16 GA deck diaphragms connected to continuous ledger angle with expansion anchors at 2' O.C.**
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
				Seismic-Force-Resisting System	
☒	☐	☐	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	
☐	☐	☒	☐	FLAT SLABS: Flat slabs / plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	

Building Name: **Main Electrical Distribution** Date: **3/11/14**

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Job Number: **B3215013.00** Job Name: **Union Sanitary Seismic Evaluations** By: **CAH** Checked:

ASCE 41-13 C2 / C2A Life Safety Structural Checklist: Concrete Shear Walls with Stiff or Flexible Diaphragms

C	NC	N/A	U		Comments
				HIGH SEISMICITY (Complete the following items in addition to the items for Low and Moderate Seismicity)	
☐	☐	☒	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135° or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads due to overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	
				Connections	
☐	☐	☒	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	
				Diaphragms (Flexible or Stiff)	
☐	☐	☒	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	
☒	☐	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25 percent of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)	
				Flexible Diaphragms	
☒	☐	☐	☐	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 feet consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	
☐	☐	☒	☐	UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	
☒	☐	☐	☐	OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)	

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Union Sanitary District

Detailed Seismic Assessments & Conceptual Strengthening Schemes
Union City, California

FINAL REPORT

April 22, 2016
Degenkolb Job Number B3215013.00

REGISTERED PROFESSIONAL ENGINEER
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STRUCTURAL
STATE OF CALIFORNIA

Roger S. Parra

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Executive Summary

This report summarizes the findings from detailed seismic assessments and outlines the conceptual strengthening schemes for the Primary Clarifiers 1-4, Administration Building, Field Operations Building, and Control Building. Note that this work expands upon the preliminary seismic assessment report "Union Sanitary District Seismic Vulnerability Assessment," and should be read in concert with that report. Based on the detailed assessments performed for the preparation of this report, the nature of the seismic deficiencies and scope of retrofit work required to mitigate those deficiencies are in line with the findings of the initial screening report.

The Primary Clarifiers 1-4 is a concrete structure which has precast concrete roof beams and precast concrete wall panels supported by a cast-in-place concrete structure below. The major seismic deficiencies in the building are the inadequate inter-connection between adjacent precast roof beams and the connection between the precast walls to the roof and cast-in-place concrete walls below. The deficiency can be mitigated by improving the connections of precast beams and precast wall panels.

The Administration Building is a two-story structure which consists of steel and wood framing and exterior precast concrete panels. The major seismic deficiencies are non-ductile braced frames and inadequately braced precast panels. The deficiencies can be mitigated by replacing the existing braces with new buckling restrained braces and bracing the precast panels.

The Field Operations Building is comprised of two structurally separate one-story buildings. One is a wood framed structure with plywood walls and the other has a wood framed roof supported by precast concrete walls. The major seismic deficiencies are inadequate connections between the roof and the precast panels and the potential for pounding between the structures. The deficiencies can be mitigated by improving the diaphragm to precast panel connection and reducing the anticipated displacement between the structures with new exterior buttresses.

The Control Building is a light cold-formed steel framed two story structure. The major seismic deficiencies are inadequate shear walls, discontinuous shear walls and diaphragms. The deficiencies can be mitigated by strengthening the existing shear walls with plywood, strengthening the diaphragm with plywood, and strengthening the connections at the discontinuous walls and diaphragms.

Based on our understanding of building operations their use and seismic risk it would be reasonable to phase the seismic upgrades of the four buildings as follows: 1) Administration Building, 2) Field Operations Building, 3) Control Building, 4) Primary Clarifiers 1-4.

The retrofit work outlined in this report is shown in greater detail on conceptual strengthening drawings which accompany this report in Appendix A.

Cumming has prepared a rough order of magnitude cost estimate for the work outlined in Appendix B of this report and findings are summarized in the table below.

Seismic Strengthening Construction Costs

Building	Cost
Primary Clarifiers 1-4	$ 3.4 Million
Control Building	$ 1.9 Million
Field Operations Building	$ 1.5 Million
Administration Building	$ 4.7 Million
Total	$ 11.5 Million

1.0 Tier-3 Detailed Seismic Analysis Procedure

We have completed detailed analyses and developed conceptual strengthening schemes for the Primary Clarifiers 1-4, Administration Building, Field Operations Building, and Control Building using the ASCE 41-13 Tier-3 detailed linear static and linear dynamic analysis procedure. The Tier-3 procedure uses detailed calculations and computer models of the building to determine seismic demands on the various elements that comprise a building's structure – each individual element that resists seismic forces is subsequently evaluated to determine whether or not it has sufficient strength to resist the earthquake demands. The Tier-3 analysis procedure differs from the Tier-1 checklist based screening procedure, which uses very simplified calculations on the aggregate of the lateral force resisting elements to flag potential deficiencies. In this manner, a Tier-3 analysis allows for targeted retrofit schemes, whereas a Tier-1 analysis merely identifies potential deficiencies.

The structural elements in a Tier-3 analysis procedure are divided into two categories: "force-controlled elements" which cannot sustain significant inelastic deformations without failing, and "deformation-controlled elements" which can undergo inelastic deformations without compromising the structural integrity of the structure. Consequently, the brittle force-controlled elements are checked to ensure they remain essentially elastic, and the ductile deformation-controlled elements are checked to ensure that they do not exceed an acceptable level of inelastic deformation.

The acceptance of structural elements hinges on two major variables; the force in the elements, which is governed by the earthquake magnitude considered, and the target performance level for the structure. For this study, the earthquake considered is the same earthquake that a new building would be designed for per the 2013 California Building Code. This earthquake is commonly referred to as the "DBE" ("Design Basis Earthquake") throughout this report, and is the same earthquake used for the initial screening study. This earthquake represents approximately a M6.3 magnitude earthquake on the nearby Hayward Fault, assuming that the fault ruptures nearby the Union Sanitary District Site. The shaking from this event at the site is expected to occur roughly every 200 years, and therefore has a significant chance of occurring within the lifetime of the structures.

Where brittle force-controlled elements are shown to exceed their elastic capacity in the DBE earthquake, retrofit measures have been introduced to protect that element. The amount of deformation that deformation-controlled elements can sustain depends on the targeted performance for the structure; the better the target performance, the lower the amount of allowed inelastic action. Seismic retrofits usually target one of two seismic performance levels: "life-safety" performance or "immediate occupancy" performance. An "immediate occupancy" performance retrofit targets a lower amount of inelastic action than a "life-safety" performance retrofit. An "immediate occupancy" seismic performance translates to expected "green tagged" seismic performance where the extent of seismic damage would not preclude occupancy of the structure before repairs are made. "Immediate Occupancy" seismic performance does not necessarily mean that the structure will be fully functional directly following an earthquake, because non-structural systems could be damaged during an earthquake and may be in need of repair.

"Life-safety" performance endeavors to keep the major structural integrity intact during an earthquake. For structures that are normally occupied like the Administration Building, meeting "life-safety" performance indicates that there is a very low probability that earthquake damage to the structure will result in the loss of life. For unoccupied structures, meeting "life-safety" performance indicates that there is a very low probably that earthquake damage will cause partial or complete collapse of the structure. For an unoccupied tank, for example, "life-safety" performance would indicate that the tank has not been damaged to the point where it "partially collapses" and rapidly loses its contents. It's important to note that "life-safety" performance does not indicate that a given structure will not be damaged to the point where it is immediately usable following an earthquake. Furthermore, structures can be considered to have met "life-safety" performance even though they have been damaged to the point where repair is not feasible and they may need to be demolished following an earthquake. Where deformation-controlled elements are shown to exceed their inelastic capacity for life-safety performance in the DBE earthquake, additional deformation-controlled elements have been added to protect that element. Except where noted otherwise, each of the structures included in this study has been targeted for "life-safety" performance level per ASCE 41-13.

Note that the scope of this report is limited to the seismic assessment of the *structural systems* of the four structures included as part of this study. Evaluation of non-structural equipment and its seismic anchorage is not within the scope of this study. From our various site visits we noticed that some pieces of equipment seemed to be adequately anchored, and other pieces of equipment appeared to lack any seismic anchorage. Consequently, we recommend that a seismic assessment of important equipment at the site be done as a follow-up to this study.

2.0 Primary Clarifiers 1-4

The Primary Clarifiers 1-4 and the associated pump building are partially cast-in-place, partially precast concrete structures that were constructed during the 1978 phase of construction. The base slab, pilaster elements and tank walls are cast-in-place elements. The exterior walls above grade and roof structure are comprised of precast concrete elements. In the original 1978 construction, the precast concrete roof elements were not well tied into the walls below and were not well tied to each other. The clarifiers were seismically retrofitted in 1991 by Carollo Engineers to address some of the issues with the original 1978 construction. The plates added as part of the retrofit are shown in Figure 1. While these retrofits will improve the performance of the clarifiers in a significant earthquake, the Primary Clarifiers 1-4 are still deficient for the current evaluation criteria in many respects. These seismic deficiencies are discussed in greater detail below.

Figure 1: Portion of 1991 Structural Retrofit to Primary Clarifiers 1-4

The primary system that resists inertial and incremental fluid loads of the clarifiers and pump room are the concrete walls. The primary lateral force resisting system for the above grade structure consists of the existing Precast Double Tees acting as a horizontal diaphragm spanning to the perimeter precast concrete walls. The precast concrete wall panels directly under the precast roof are highlighted in Figure 2.

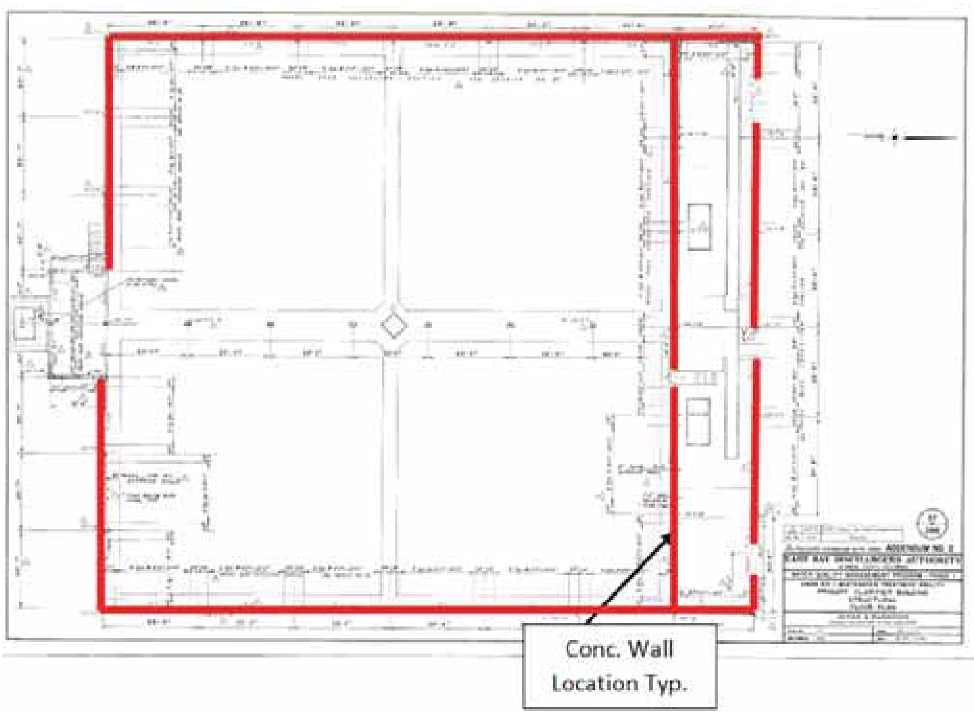


Figure 2: Primary Clarifiers 1-4 Precast Concrete Wall Locations

2.1 Deficiencies and Retrofit Measures

2.1.1 No Topping Slab at the Precast Concrete Roof

The diaphragm ties that connect the Precast Double Tee roof beams together are corroded and will be ineffective at keeping the diaphragm together in the DBE. Once the panels begin to break apart, the strength of the Double Tee roof beams will be significantly compromised and the precast double t-panels themselves could begin to break apart. To mitigate this deficiency, we propose strengthening the Precast Double Tee joint with Fiber-Reinforced Plastic (FRP) sheets. These FRP sheets can either be installed above or below the roof along the joints between the Double Tee beams. Installing the sheets from below would require scaffolding and would require sequencing in order to complete the work one quadrant at a time in the unfilled cells. Completing the work from above the concrete beams would require the removal and replacement of the existing roofing membrane but for the installation of the strips themselves it will be much more cost effective to install the FRP strips from above the roof. The diaphragm RFP strengthening is shown in Figure 1-1.

2.1.2 Load Transfer to Precast Shear Walls

The 1991 retrofit positively attached the roof diaphragm to the precast concrete walls, however, the connections are not sufficient for the demands in the DBE based on Tier-3 analysis procedures – it is possible that the steel bolts that were installed as part of the retrofit will rip through the two inch thick concrete diaphragm which could lead to a localized collapse. The connection at the bottom of the precast panels to the cast-in-place clarifier concrete walls was also found to be deficient. The bottom connection does not have adequate capacity to transfer seismic load, especially the force in in-plane direction of the panel. Therefore, we recommend supplementing these connections with a cast-in-place concrete connection that will effectively tie the roof into the precast concrete walls and connecting the precast concrete panels to the cast in place walls below. Strengthening is shown in Figures 1-2, 1-3, 1-4 and 1-5.

2.1.3 Out-Of-Plane Wall Anchorage

The roof level anchorage that was added as part of the 1991 retrofit is very flexible for resisting out-of-plane loads, and will not effectively serve as out of plane anchorage. Consequently, the precast concrete panels will have to span horizontally to the concrete pilasters for out-of-plane support. Based on our Tier-3 analysis, the panels are sufficiently strong to span to the concrete pilasters on either side of the wall panel and down to the cast-in-place concrete wall that supports the pilasters. Furthermore, the concrete wall panels are sufficiently strong to contend with the weight of the panel. However, the deflection at the top of the wall panels is estimated to be 6-8 inches in the DBE. This large displacement will lead to damage at the wall/roof interface, and it is possible that the concrete beams at the roof will unseat and lead to a localized collapse.

Therefore we recommend providing effective connections between the precast panels and the roof diaphragm for out-of-plane load transfer. This connection can be achieved by the cast-in-place concrete connection recommended for the shear transfer to the wall above. The connections will provide adequate capacity for seismic load transfer in both out-of-plane and in-plane directions. Strengthening is shown in Figures 1-2, 1-3 and 1-5.

2.1.4 Unbraced Mezzanine

The mezzanine in the pump room is not adequately braced based on Tier-3 analysis procedures, and could collapse in the DBE. We recommend installing tension rod bracing to limit the movement of the mezzanine and protect the structure.

2.1.5 Adjacent Structure

The clarifiers are adjacent to the expanded and re-built control box #1. The control box is partially integral and partially separated from the clarifiers. We recommend supplementing the connectivity between the control box structure and the clarifier structure. This can be accomplished with cast-in-place concrete curb shown in Figure 1-4 preventing pounding damage between the two structures in the DBE.

The conceptual strengthening drawings for Primary Clarifiers 1-4 are shown in Appendix A (Figures 1-1 through 1-5).

3.0 Administration Building

The Administration Building is a two-story structure built in 1999 designed under the 1994 Uniform Building Code. The structure is very geometrically and structurally irregular and complex; many different structural elements and materials are used to support the structure including rolled steel shape beams and columns, open web steel joists, dimensional timber beams, glue-laminated timber beams and precast concrete panels. The complex geometry can be seen from the exterior view of the Administration Building shown in Figure 3.

Figure 3: Exterior View of Administration Building

Concentrically braced frames serve as the structure's primary lateral force resisting system. The brace locations on the 1st story are shown in Figure 4. As noted in greater detail below, the concentrically braced frames do not have ductile detailing and are not expected to perform well in the DBE. Furthermore, the tall architectural precast concrete panels are not well tied into the structure and may collapse in the DBE. Retrofit strategies to address these deficiencies are discussed in greater detail below.

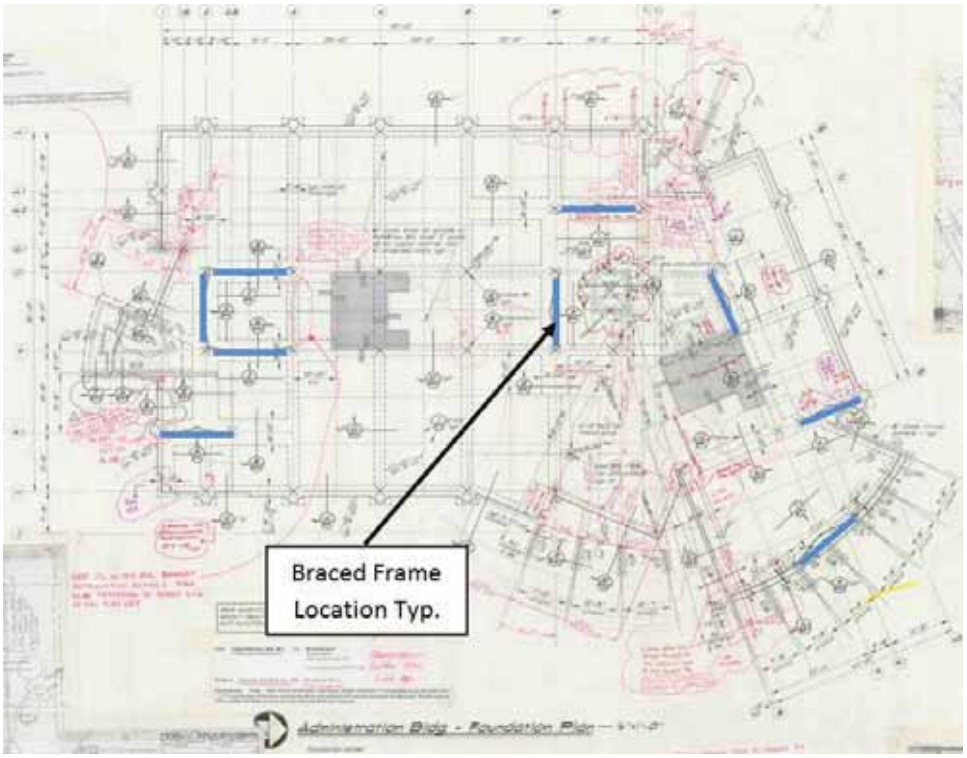


Figure 4: First Story Braced-Frame Locations in Administration Building

3.1 Deficiencies and Retrofit Measures

3.1.1 Braced Frames

The braced frame tube members themselves are relatively ductile members that are capable of sustaining inelastic action without failing. Based on the braced demands determined in the Tier 3 analysis, all of the tube brace members are sufficient to meet the life-safety performance criteria at the DBE. However, the Tier 3 analysis has shown that many of the brace connections, and many of the beams and columns that surround the bracing members will either fail prior to brace members themselves undergoing inelastic action, or will fail once the compression brace in the braced frame buckles. The brace configuration for many of the frames in the building consists of a chevron configuration which tends to perform very poorly. Prior to the compression brace buckling in a chevron braced frame configuration, the vertical component in the compression brace is balanced against the vertical component of the tension force in the tension braces such that there is no net vertical force on the beam at the brace intersection. However, once the compression brace buckles, it can no longer carry significant load, which then causes force to be transferred to the tension brace. Once this occurs, there is a very large downward force that must be resisted by the beam at the chevron brace intersection. This process is shown graphically in the figure below. In the Administration Building, the Tier 3 analysis has shown that once buckling occurs, the beams and beam connections are significantly deficient to resist the net vertical force, which could result in potential for localized collapse due to the failure in the beam.

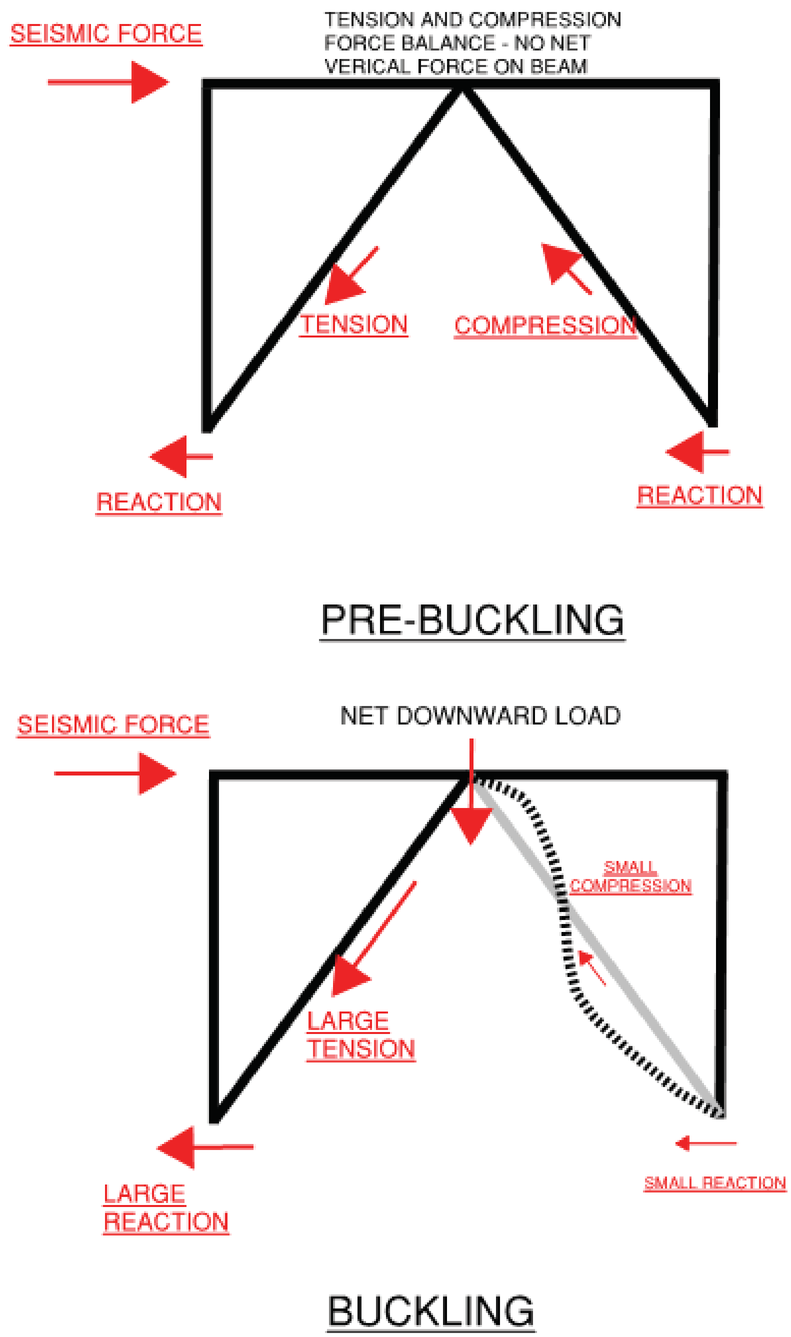


Figure 5: Chevron Brace Buckling Progression

Based on these analysis results, there are 2 possible courses of action to remedy the deficiencies associated with the braced frames. One course of action is to strengthen the beams, columns and connections at the braced frames. This course of action involves removing drywall to access the brace locations, and then involves intricate and complex strengthening of members and connections. The other course of action is to replace the steel tube braces with buckling-restrained braces, or "BRBs". BRBs are steel core braces encased in a concrete sleeve. The concrete sleeve prevents the steel core from buckling, which gives the braces essentially the same capacity in tension as in compression, and prevents the braces from buckling. Consequently, these highly ductile members do not have the same issue with unbalanced vertical forces as traditional tube braces. Furthermore, BRBs can be made to be very stiff relative to their strength. For use in the Administration Building, BRBs could be made stiff enough to match the existing stiffness of the existing tube braces, while being made weak enough to help protect the frame beams, columns and connections producing desirable and predictable ductile behavior.

For the retrofit of the Administration Building, we propose using a hybrid scheme between the two possible courses of action. Where braces are strong enough such that they are unlikely to buckle, the scope of strengthening to the frame beams, columns and connections is relatively minor – in these cases our recommendation is to simply correct the localized strength deficiencies. However, in the majority of cases where braces are not strong enough to avoid buckling, the strengthening required is very extensive, and our recommendation is to replace the existing braces with BRBs.

3.1.2 Precast Panel Out-Of-Plane Anchorage

The typical wall anchorage straps and their development of the loads into the diaphragm at roof level is overstressed based on Tier-3 detailed analysis procedures. Furthermore, the typical strap anchors provided have been cast into the precast panels meaning that there is no field tolerance to install the anchors. In many cases the straps are not installed as intended making them even less effective at bracing the panels.

Furthermore, the top of many panels is only connected at a single point. Based on our evaluation, the inherent torsion that this single-point bracing scheme produces makes the panels unstable and susceptible to collapse. This is particularly troublesome at the entrance of the building, where the collapse of the panels could not only endanger personnel, but could restrict egress following an earthquake.

In many cases, it would be possible to supplement the out-of-plane wall panel anchorage by installing bolts through the precast panels, and holdowns attached to the existing roof framing. However, in most cases this course of action would require adding supplemental framing and diaphragm strengthening to be able to develop this force into the roof diaphragms. Consequently, in lieu of this traditional wall anchorage scheme, we recommend introducing a steel bracing grid and BRB buttresses to anchor the walls. This scheme is stiff enough to protect the existing diaphragms and walls while reducing the strengthening costs and limiting the work that needs to be done within the occupied structure.

3.1.3 Vertical Irregularities

The brace on Line 8 is discontinuous at the second floor. At one end of the brace, the load is resisted by a transfer beam, which is insufficiently strong to take the vertical transfer reaction based on Tier-3 analysis results. Consequently, we recommend locally strengthening this transfer beam. Furthermore, the horizontal transfer connection into the concrete precast wall on Line 8 is also deficient and needs to be strengthened, as does the connection of this concrete wall into the foundation. Retrofits of these deficiencies are shown on Figures 2-1 and 2-2 in Appendix A at the end of this report.

The small brace on Line G that braces the back of the curved low roof portion is discontinuous, but the columns located at the ends of the small brace are sufficiently strong based on Tier-3 analysis procedures. However the ledger beam adjacent to the small brace is connected to a larger full-height brace at its mid length. Differential deformation along this drag induces enough bending at the full-height brace to cause failure based on Tier-3 analysis procedures. Consequently, we propose cutting this infill ledger free, and supporting the ledger framing member with a small post.

The Mansard roof is attached to the roof diaphragm for shear transfer parallel to the edge of the building. Perpendicular to the edge of the Administration Building, the Mansard Roof must span horizontally to the small cross-walls at the ends of the Mansard. Tier-3 analysis shows that these walls are insufficiently long. Consequently, to supplement the perpendicular seismic capacity of the Mansard, we recommend attaching each of the Mansard roof trusses to the diaphragm with framing clips.

3.1.4 Diaphragm Drags

Typically the diaphragm drags and connections into the framing members are sufficient based on Tier-3 analysis procedures. However, at the main roof diaphragm, the wood nailer connection that connects the plywood sheathing of the diaphragm into the steel beams on braced-frame lines, is much weaker than the plywood connections into the ledger. In order to adequately deliver load to the braced frames, this ledger connection needs to be supplemented in select locations.

The conceptual retrofit drawings for Administration Building are attached in Appendix A (Figures 2-1 through 2-10).

4.0 Field Operations Building

The Field Operations Building is comprised of two structurally separate, one-story buildings that were constructed in 1999 under the 1994 Uniform Building Code. One structure is a low-bay, wood framed structure with a plywood shear wall lateral system. The other structure is a high-bay wood framed structure with a precast concrete tilt-up wall lateral system. An exterior view showing both parts of the structure can be seen in Figure 6.

Figure 6: Exterior View of Field Operations Building

The locations of the plywood shear walls and precast concrete tilt-up walls are shown in Figure 7. As noted in greater detail below, the main deficiency of the Field Operations Building is the deficient connections between the precast concrete walls and the plywood diaphragms. These limited connections are not expected to perform well in the DBE. Furthermore, the structural separation between the two structures is inadequate based on Tier-3 analysis procedures which could lead to pounding between the two structures. Retrofit strategies to address these deficiencies are discussed in greater detail below

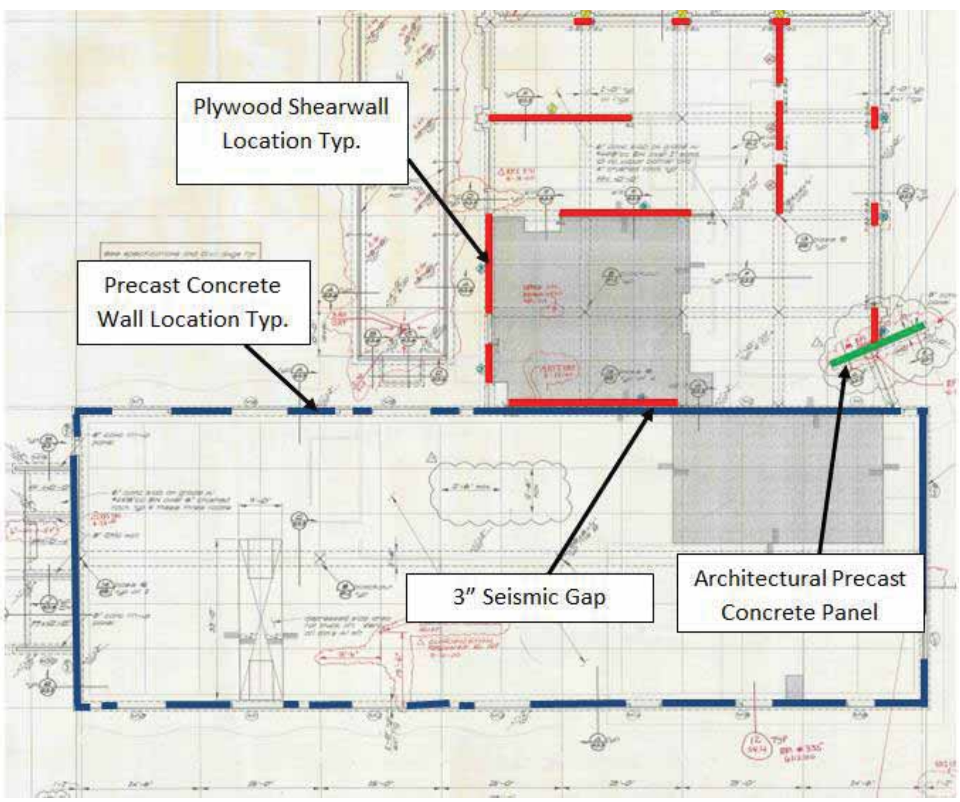


Figure 7: Wall Locations in Field Operations Building

4.1 Deficiencies and Retrofit Measures

4.1.1 Precast Panel Out-Of-Plane Anchorage

The typical embedded anchorage straps are overstressed, indicating that they could pull out and be ineffective in the DBE. We recommend connection strengthening between the roof framing and the precast concrete walls by installing new holdowns thru bolted to the wall. In addition to the deficient anchorage, the sub-diaphragm anchorage development in North-South direction is overstressed, which would lead to damage in the diaphragm. We recommend mitigating this deficiency by installing new crossties between the joists at the glulam ridge beam intersection.

The architectural concrete panel at the front entrance to the building, and at the back electrical room are not adequately anchored and represent a potential collapse hazard based on Tier-3 analysis procedures. At the front entrance to the building, the front of the panel is anchored to the diaphragm extension, but the diaphragm extension is too weak to support the front of the panel. Consequently, we recommend bracing the panel directly to the main structures drag line at Line D and Line 1.5 using a steel pipe brace. In addition, the shear wall at the back of the panel is too weak to effectively brace the panel. At this location, we propose introducing new holdown anchors and propose strengthening the existing plywood shear wall.

4.1.2 Adjacent Buildings

The clear distance between low-bay and high-bay portions of building is only 3 inches. Based on Tier-3 analysis, the gap needs to be approximately 6-8 inches to avoid pounding between the two structures in the DBE. There are two primary methods to address this deficiency: the first approach is to add stiffness to one or both of the structures to reduce the anticipated building displacement such that the existing gap is adequate, and the other approach is to increase the size of the gap. For the Field Operations Building, increasing the size of the gap would involve relocating columns, pouring new foundations, and replacing the existing spandrel framing and reframing the mansard roof. Additionally, this approach would require new waterproofing and a new joint be introduced, which is a challenge in an existing structure.

Based on Tier-3 analysis procedures, the alternate approach of adding stiffness to the structure can be achieved by introducing two BRB buttresses to the high bay portion of the building. Glulam drag elements need to also be added at the BRB lines to effectively drag load out of the roof diaphragm and into the buttress elements. This approach limits the work that needs to be done within the occupied structure, and doesn't risk the possibility of introducing a new waterproofing element that could leak in the future.

4.1.3 Inadequate Length of Plywood Shear Walls

The short plywood walls at the North end of the Low-Bay structure are overstressed based on Tier-3 analysis procedures. These walls need to be supplemented by adding an additional short segment of wall. We propose adding a short plywood wall in the Crew Room to address this deficiency.

4.1.4 Diaphragm Continuity

The mansard roof creates a stepped diaphragm at the roof. The back side of the mansard is plywood so the load transfer in this direction is sufficient based on Tier-3 analysis procedures. In the direction normal to the roof edge for the East and West mansard roof portions, the mansard diaphragm must cantilever from the North mansard portion which it is inadequate to do based on Tier-3 analysis procedures. Consequently, we recommend securing the South portion of the East and West mansards roofs directly to the roof diaphragm using short plywood shear walls.

4.1.5 Unbraced Mezzanines

The mezzanine in the warehouse portion of the structure is structurally independent of the precast concrete walls. This structure is a pre-engineered moment frame structure supporting relatively heavy repair parts. While the capacity of these types of frames is usually adequate, they typically experience excessive deflections in large earthquakes. The mezzanine in the Field Operations Building is directly adjacent to the exterior precast concrete wall and we would anticipate pounding damage. We recommend strengthening the mezzanine by adding steel rod X-bracing in each direction and increasing the gap between the mezzanine and the concrete walls.

The conceptual retrofit drawings for Field Operations Building are attached in Appendix A (Figures 3-1 through 3-7).

5.0 Control Building

The Control Building is a light cold-form steel framed partial two story structure. The structure has a complex pitched, Spanish Clay Tile clad roof. An exterior view of the Control Building is shown in Figure 8. This Spanish Clay Tile roof is heavy and adds a significant amount of seismic mass to the otherwise light structure.

Figure 8: Exterior View of Control Building

The building was constructed during the 1978 phase of construction, and has ¾" plywood diaphragms with gauge metal straps at discrete locations. Sheet metal X-bracing straps were originally designed to serve as the Control Building's seismic force resisting system. The locations of the sheet metal X-bracing straps are shown in Figure 9. Note how the bracing locations do not align between the two floors.

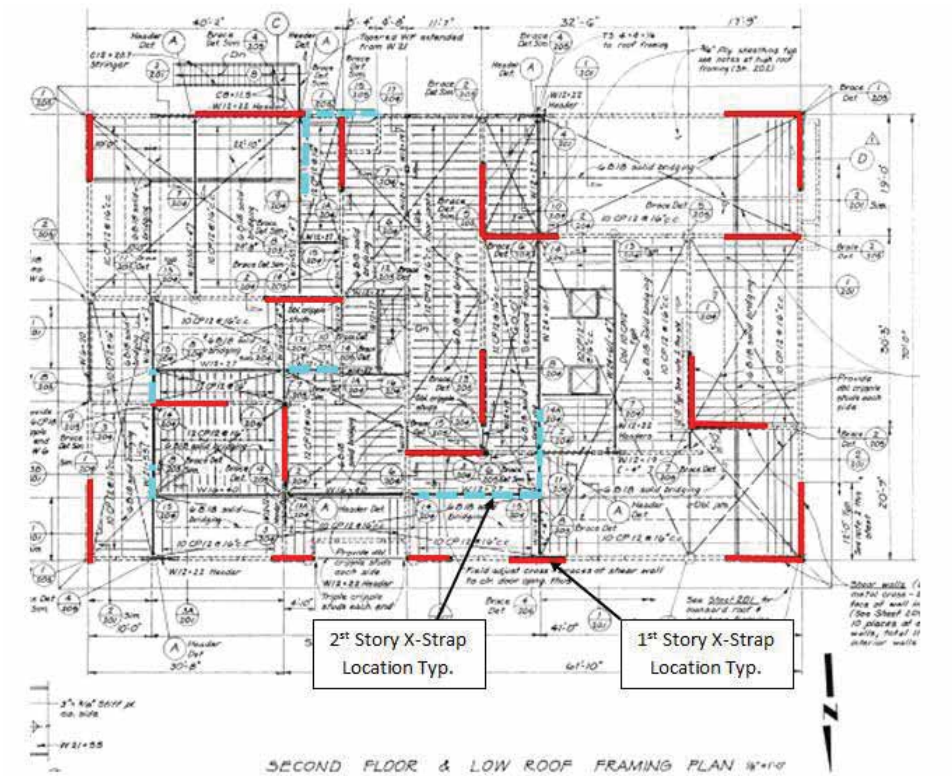


Figure 9: Control Building X-Strap Wall Locations

As noted below, these straps are much more flexible than the exterior stucco walls and interior gypsum board walls. Therefore, the strap walls will only be engaged after the other walls have sustained significant damage. In a seismic event, the plywood diaphragms will transfer the inertial load to the various walls in the Control Building, which will ultimately transfer the loads to the foundation. Typically, light-framed, low-rise structures like the Control Building perform well in seismic events. However, due to the many discontinuities in the lateral system, and because of its heavy roof, the Control Building will not perform as well in the design seismic event as other light-framed structures. These deficiencies and associated retrofit measures are discussed in greater detail below.

We understand that the Control Building is critical to continuing the operations of the Union Sanitary District. In the event of a major earthquake, the Control Building would need to be occupiable shortly after an earthquake in some capacity in order to control flows. Consequently, unlike the retrofit measures associated with Primary Clarifiers 1-4, the Administration Building, and the Field Operations Building, the retrofit measures listed below are designed for a targeted performance of "Immediate Occupancy." "Immediate Occupancy" is a higher targeted performance than "Life Safety" performance – it endeavors to keep the structure safe and usable shortly after an earthquake while repairs are made to the structure. "Immediate Occupancy" does not endeavor to keep the structure operational immediately following an earthquake, and focuses on structural related damage. Note that the retrofit measures for a “Life Safety” upgrade would be very similar to those listed below, but would be lesser in scope.

5.1 Deficiencies and Retrofit Measures

5.1.1 Shear Stress Check/Stucco Shear Walls/Gypsum Wallboard or Plaster Shear Walls

The shear in the Control Building will be distributed among the exterior stucco walls, interior gypboard walls and strap walls. While the lateral system does not exclusively rely on stucco or gypboard shear walls, these walls are much stiffer than the metal strap walls and will sustain significant damage before being engaged. Consequently, we recommend introducing a number of plywood shear walls with collectors throughout the Control Building. These walls will make the building stiff enough to protect the stucco and gypsum board walls, and help limit the damage to these elements so that they are still able to serve as structural bearing walls during and after the seismic event. Note that the vast majority of plywood walls that we are proposing are located in the same spot as current partition walls. Therefore the installation of these walls will not require a significant remodeling of the current space and function of the structure.

5.1.2 Deficient Sill Bolting

The gypboard and stucco walls are currently only connected to the foundation with shot pins or poorly detailed anchor bolts, which will be ineffective to transfer seismic force to the foundation in the DBE and render the walls ineffective at bracing the building. As part of adding plywood to the interior walls, we recommend anchoring these walls to the concrete foundation below with epoxy anchors.

5.1.3 Vertical Irregularities

Most of the walls on the second story do not align with walls below. Therefore, the second floor diaphragm will have to transfer load from the second floor walls into the ground floor walls in addition to its own mass. The plywood diaphragm is only connected to the framing with very small screws and therefore is inadequate to transfer these loads. Consequently, we propose adding plywood sheathing to the underside of the steel joists in select areas so that the diaphragm is adequate to transfer forces from the second floor walls to the ground floor walls.

5.1.4 Diaphragm Continuity

There are many diaphragm discontinuities created by the mansard roof, offsets at the roof, and complicated joist/steel beam framing. Although the vertical face of the mansard roof at the perimeter of the 2nd floor is sheathed with plywood, there is very little structural connection between the steel framing and the plywood sheathing. Consequently, adequate connection between the plywood will need to be added to make it effective to resist significant earthquake forces. This can be achieved by introducing bent steel gauge plates within the mansard.

Another discontinuity in the Control Building is the lack of connection between the roof and the walls. In order to make the new plywood shear walls effective and connect the roof and second floor into those walls, we propose introducing steel collector elements that are screwed to the roof diaphragm and into the shear walls. Finally, there is no connection between the diaphragm elements where the second floor steps from the interior to the exterior of the building. Thus, propose adding bent plates and screws to connect the diaphragm at these steps.

The conceptual retrofit drawings for Control Building are attached in Appendix A (Figures 4-1 through 4-6).

6.0 Statement of Probable Construction Costs

A conceptual design construction cost estimate for the seismic strengthening of the four buildings has been prepared by Cumming and is included in Appendix B.

The statement of probable construction costs was developed based on our conceptual seismic strengthening recommendations outlined in Sections 2 through 5 and Appendix A of this report. The construction costs for seismic strengthening of the Primary Clarifiers 1-4, Administration Building, Field Operations Building, and Control Building is $11.5 Million, including a 15% design contingency and a 10% construction contingency. The assumed start and end dates of the construction are January 2016 and January 2017, respectively. The cost estimate includes a 7.35% escalation to the assumed mid-point of construction, July 2016. A summary of construction costs is provided in the table below. Please see the cost estimate for a detailed list of inclusions and exclusions. The estimated cost of construction also assumes that the work will be performed during normal business hours but does include allowances for complexity of working within an occupied structure.

Construction Cost Summary

Building	Area (sf)	Cost / sf	Total
Primary Clarifiers 1-4 (Allowance for roof covering replacement is included)	26,430	$ 128	$ 3.4 Million
Control Building	11,855	$ 158	$ 1.9 Million
Field Operations Building	19,065	$ 81	$ 1.5 Million
Administration Building	28,328	$ 166	$ 4.7 Million
Total Estimated Construction Cost:			$ 11.5 Million

7.0 Conclusions

Based on our Tier-3 analysis and conceptual retrofit work, we have confirmed most of the deficiencies outlined in the initial screening study. These deficiencies could lead to substantial structural damage and potential loss of life in the DBE. We believe that it is possible to seismically retrofit the Primary Clarifiers 1-4, Administration Building, Field Operations Building, and Control Building, and have provided conceptual strengthening schemes to achieve this goal. From these conceptual strengthening scheme, Cumming has provided rough order of magnitude estimates for the strengthening work. The total Estimated Construction Cost for the recommended seismic upgrade schemes of the four buildings is $11.5 Million.

APPENDIX A - Conceptual Strengthening Figures

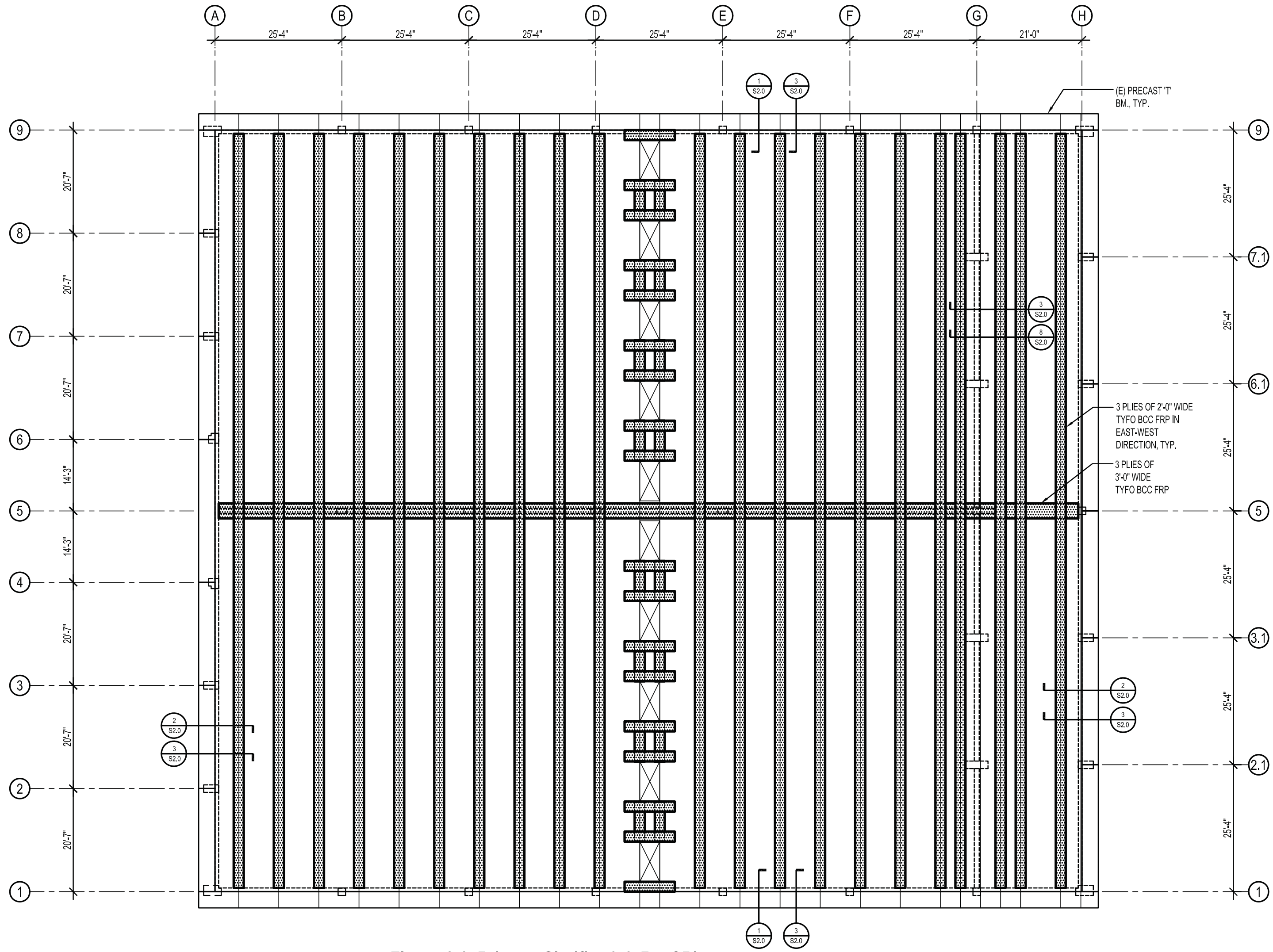


Figure 1-1: Primary Clarifier 1-4, Roof Plan

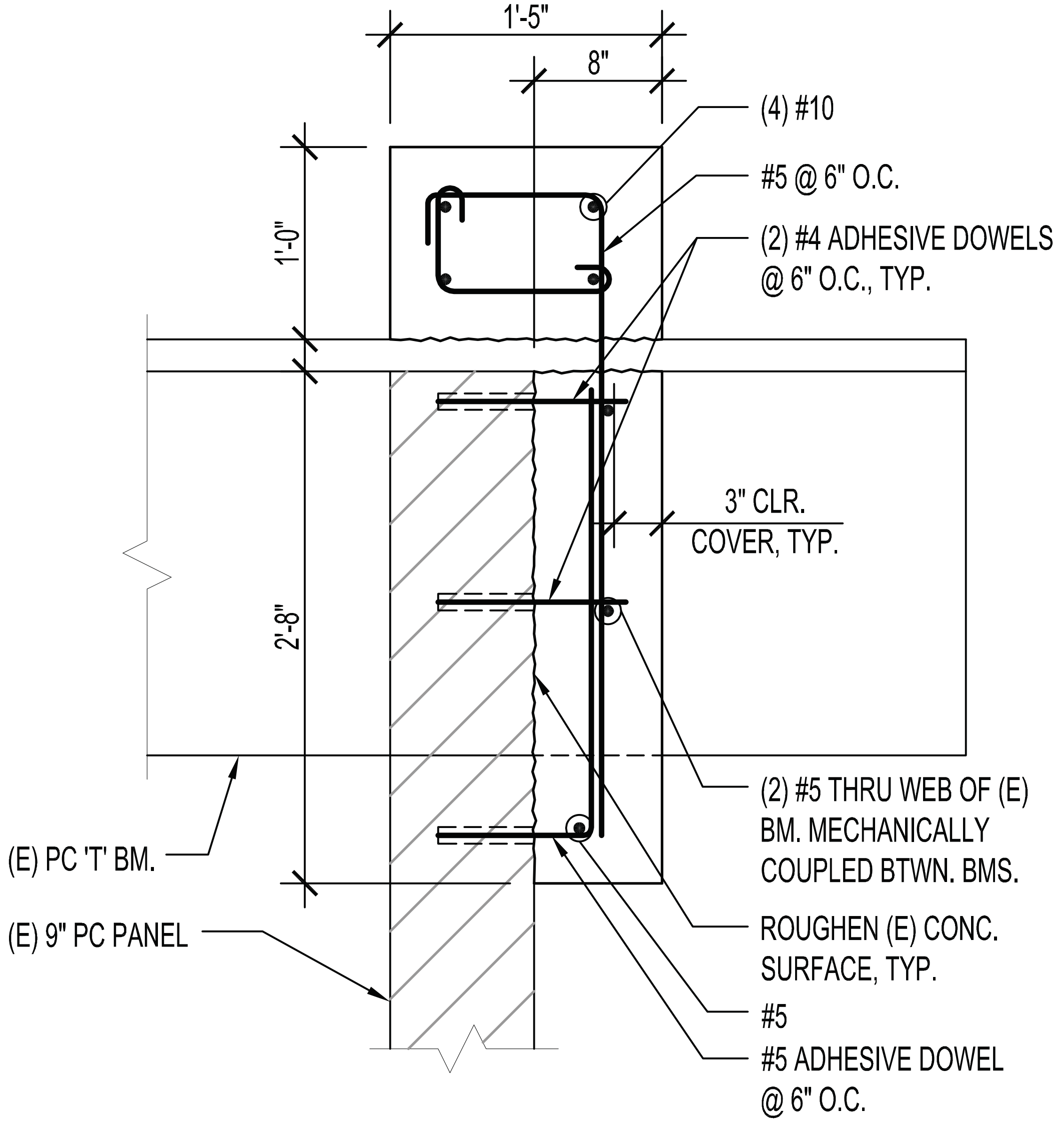


Figure 1-2: Primary Clarifier 1-4, East and West Wall

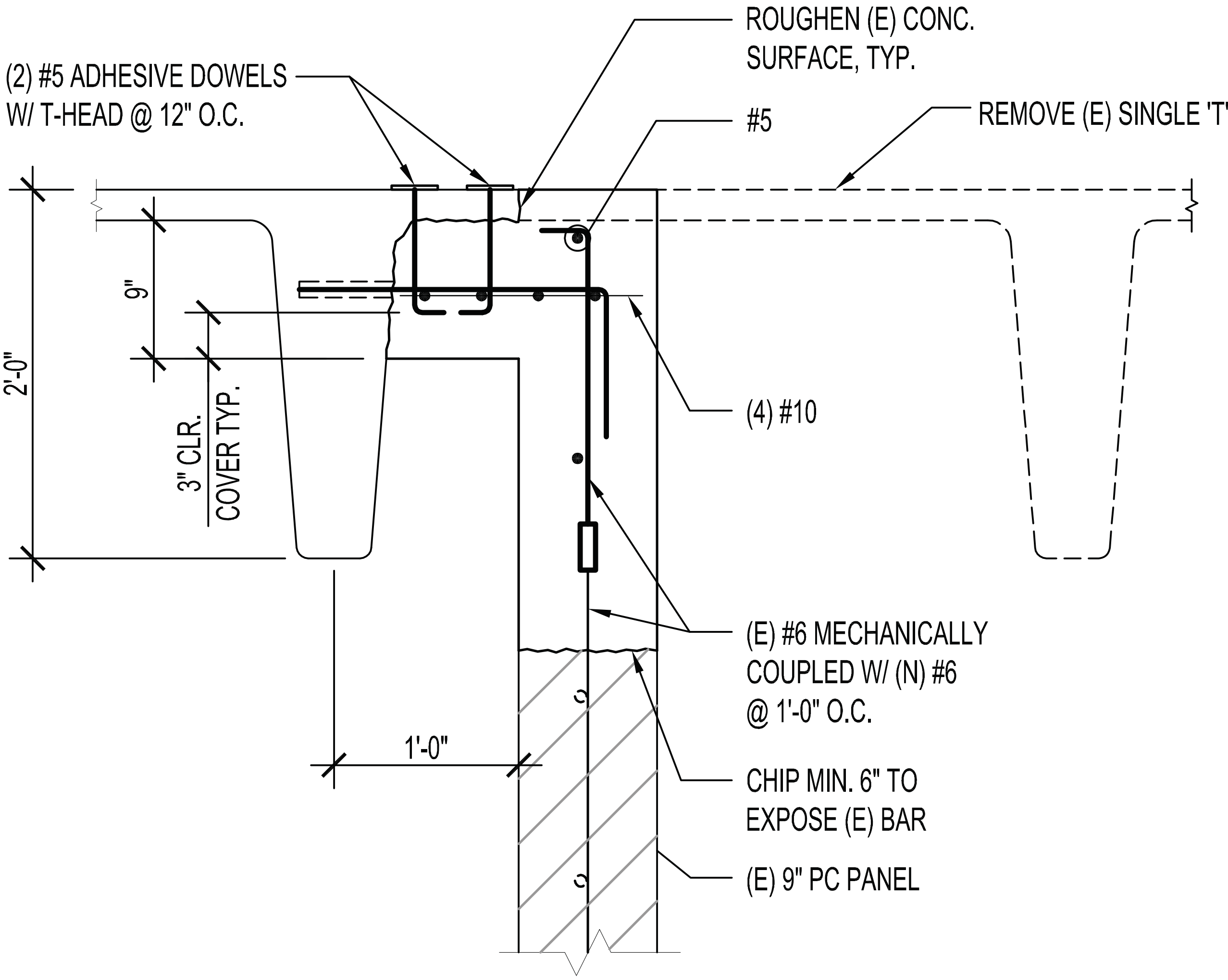


Figure 1-3: Primary Clarifier 1-4, North and South Wall

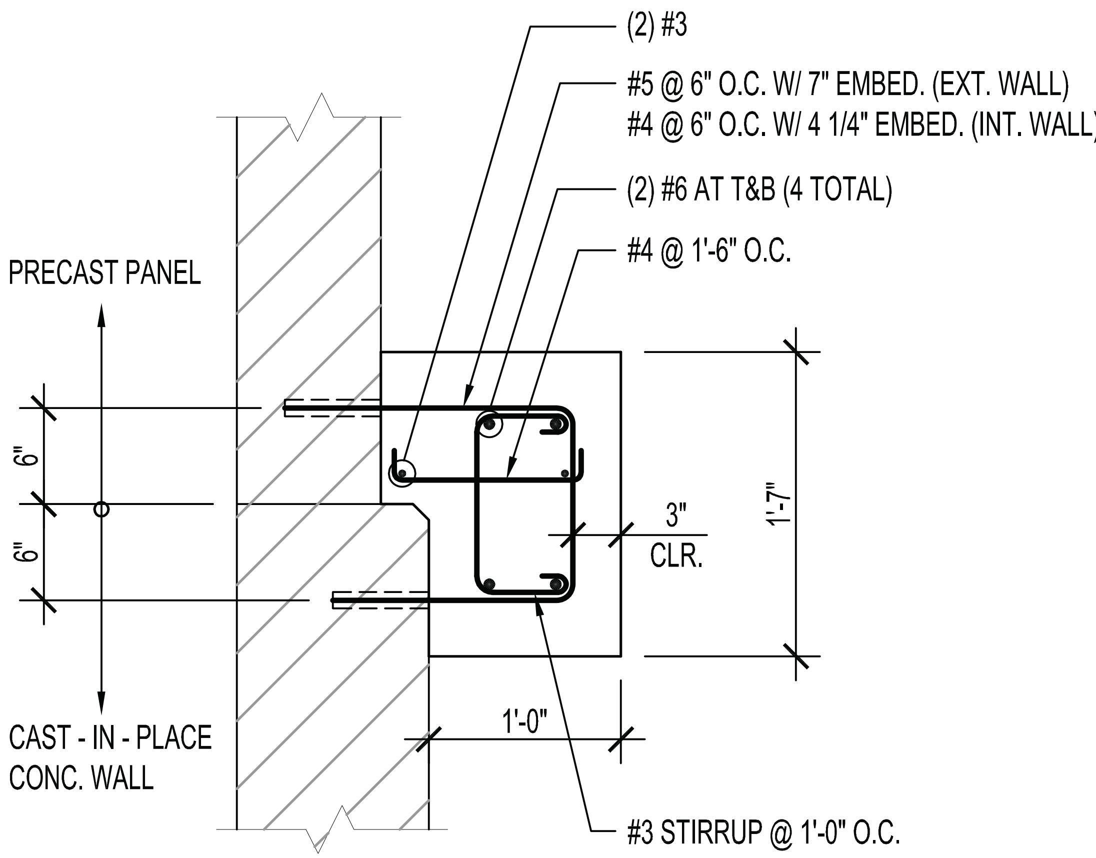


Figure 1-4: Primary Clarifier 1-4, Precast Panel Base

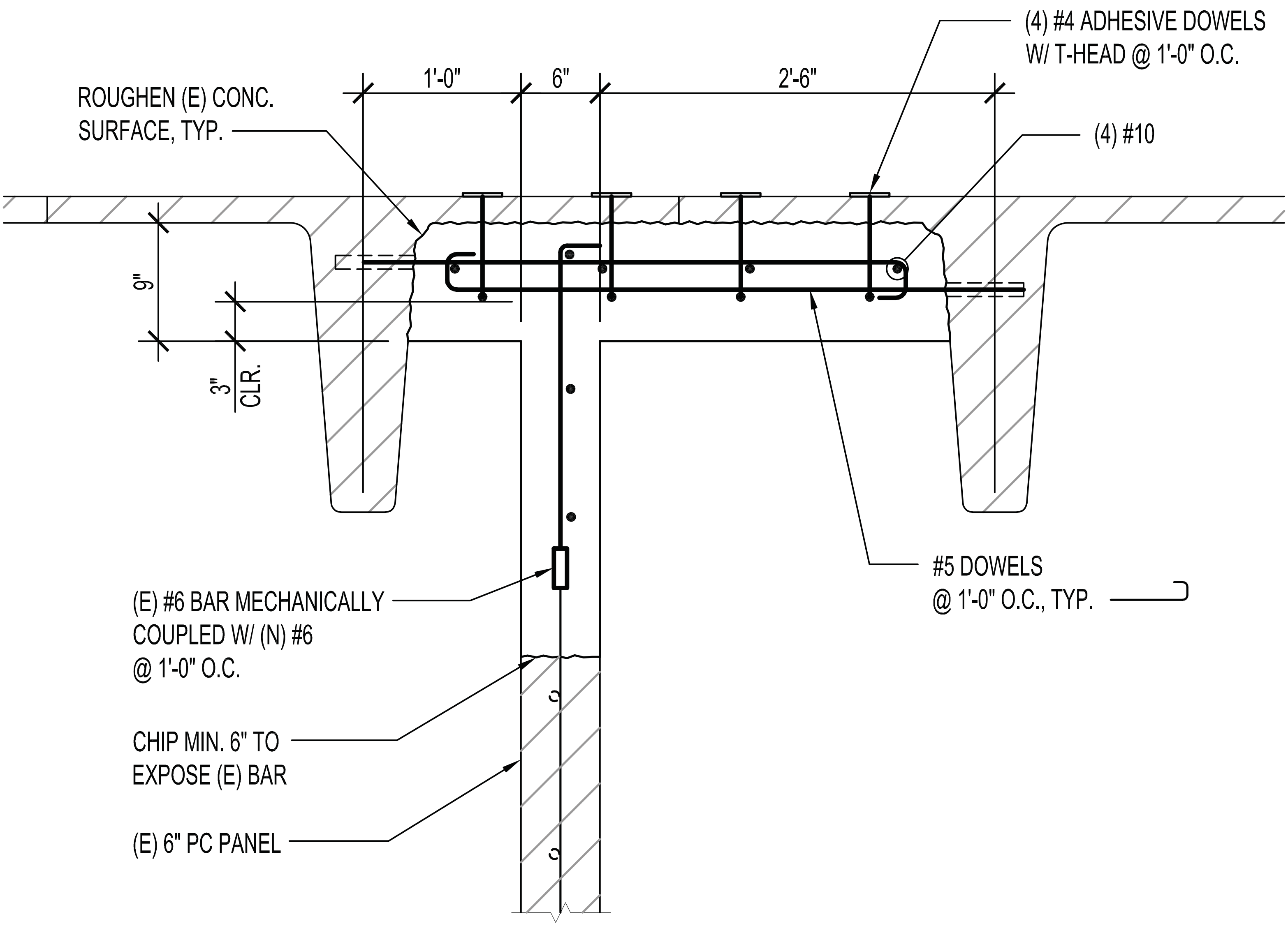


Figure 1-5: Primary Clarifier 1-4, Interior Wall

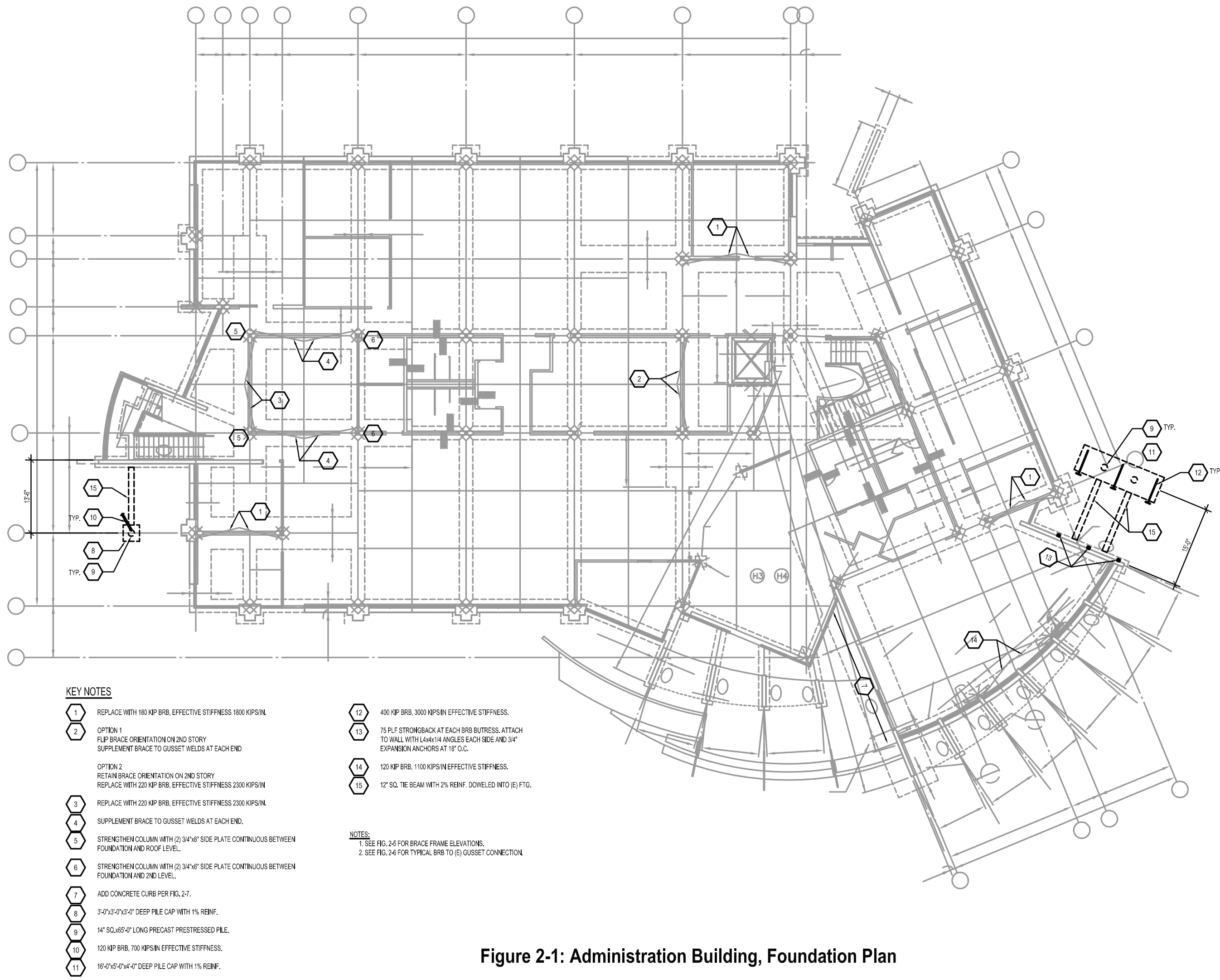


Figure 2-1: Administration Building, Foundation Plan

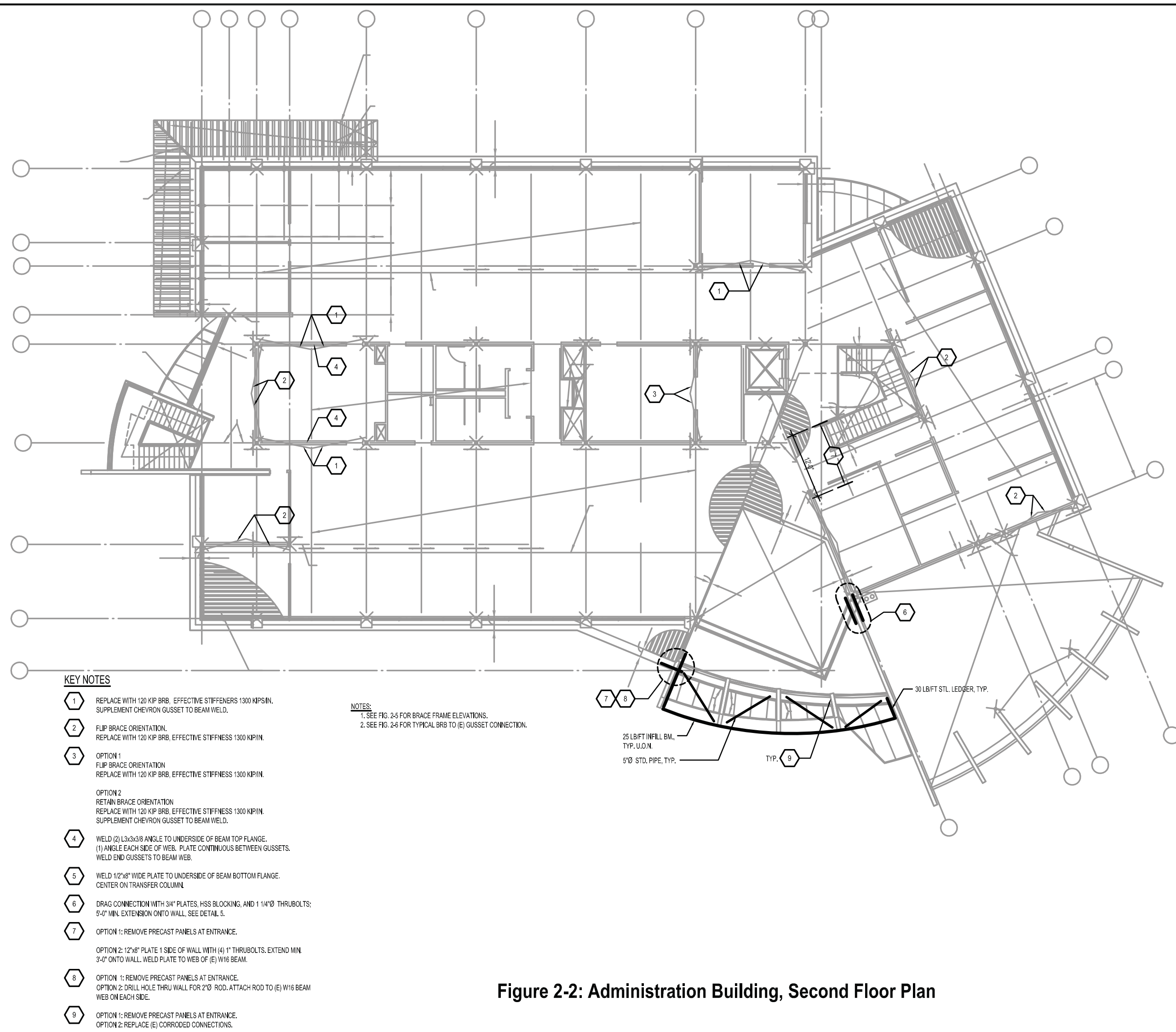


Figure 2-2: Administration Building, Second Floor Plan

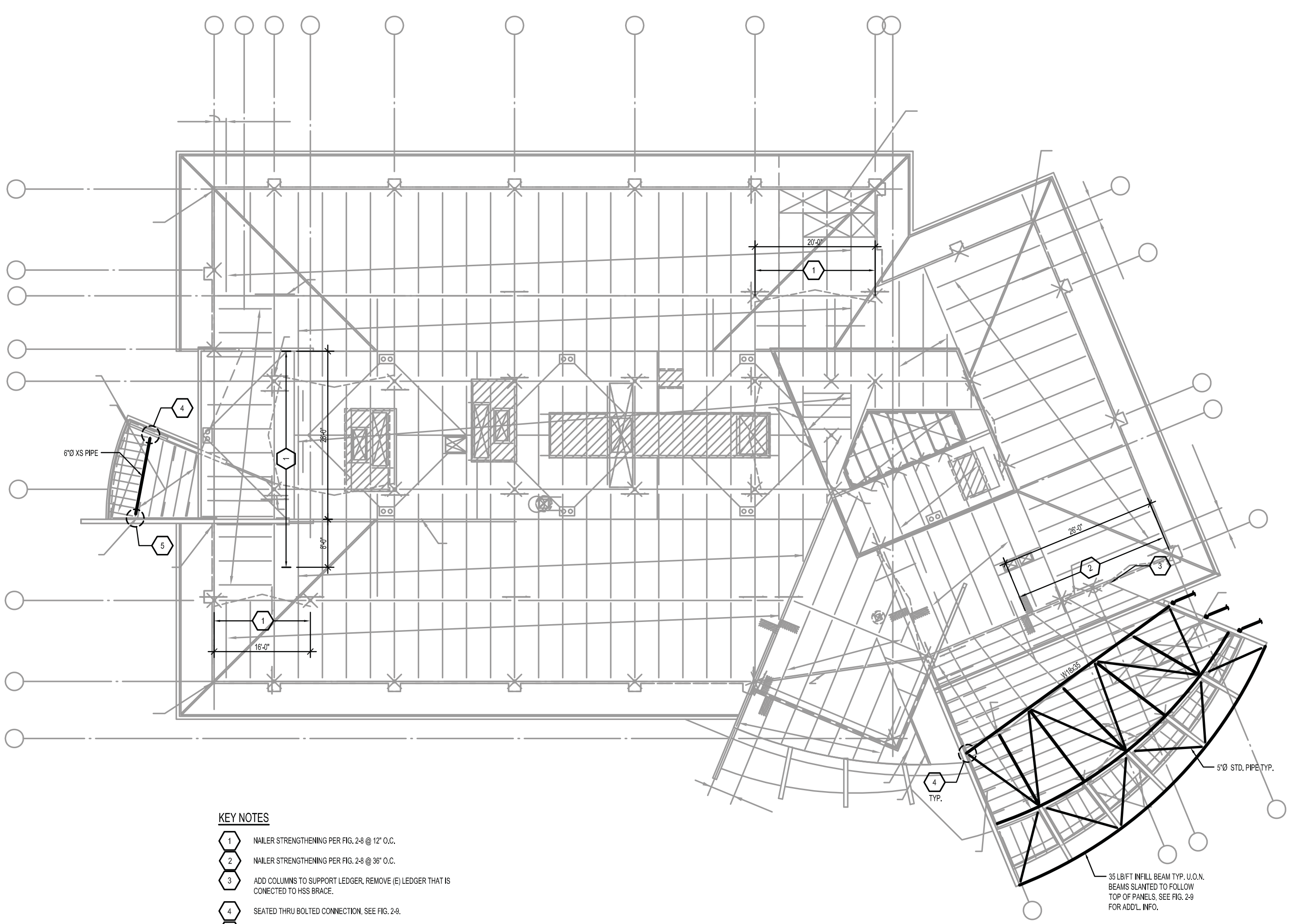


Figure 2-3: Administration Building, Roof Plan

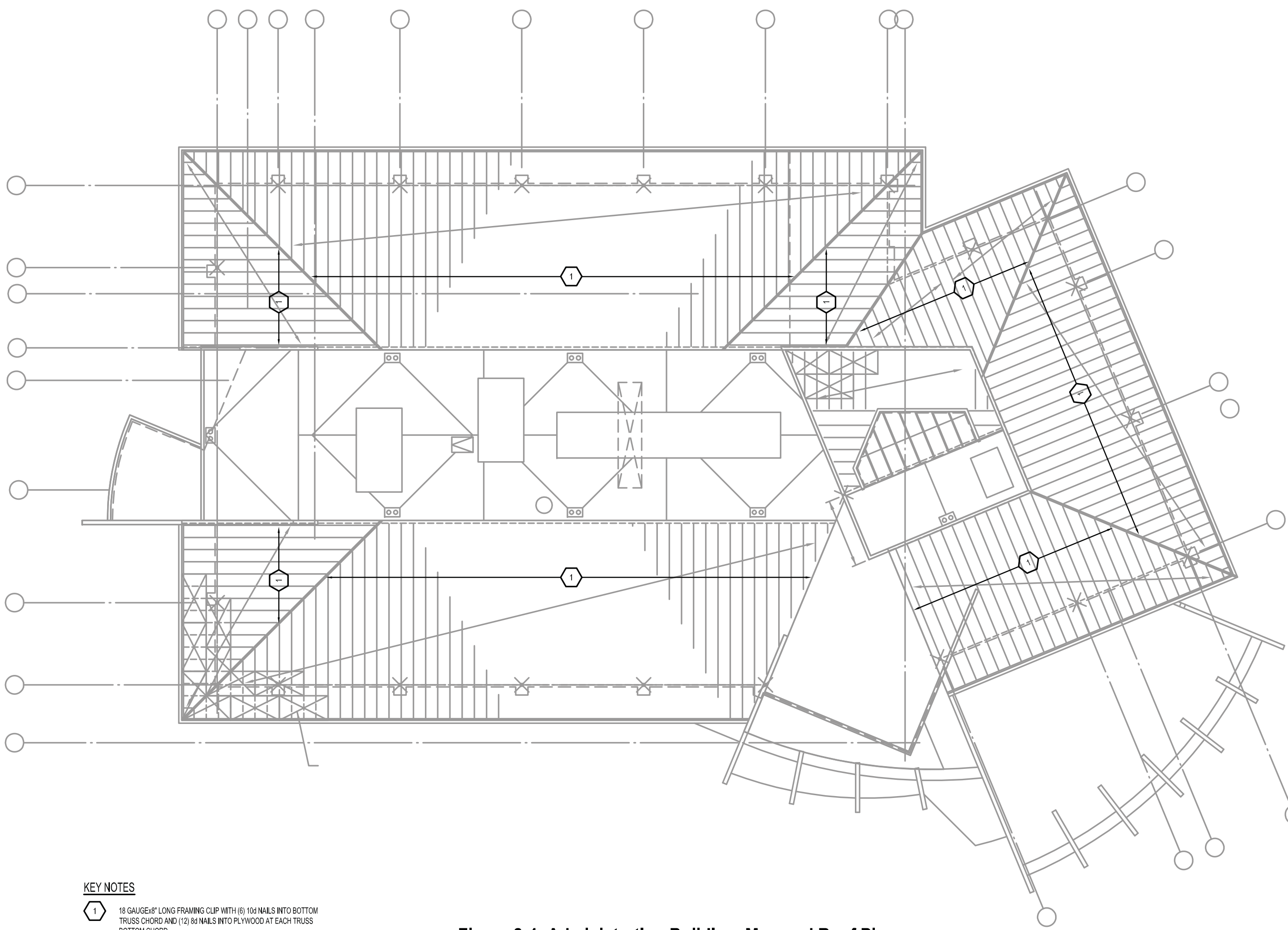


Figure 2-4: Administration Building, Mansard Roof Plan

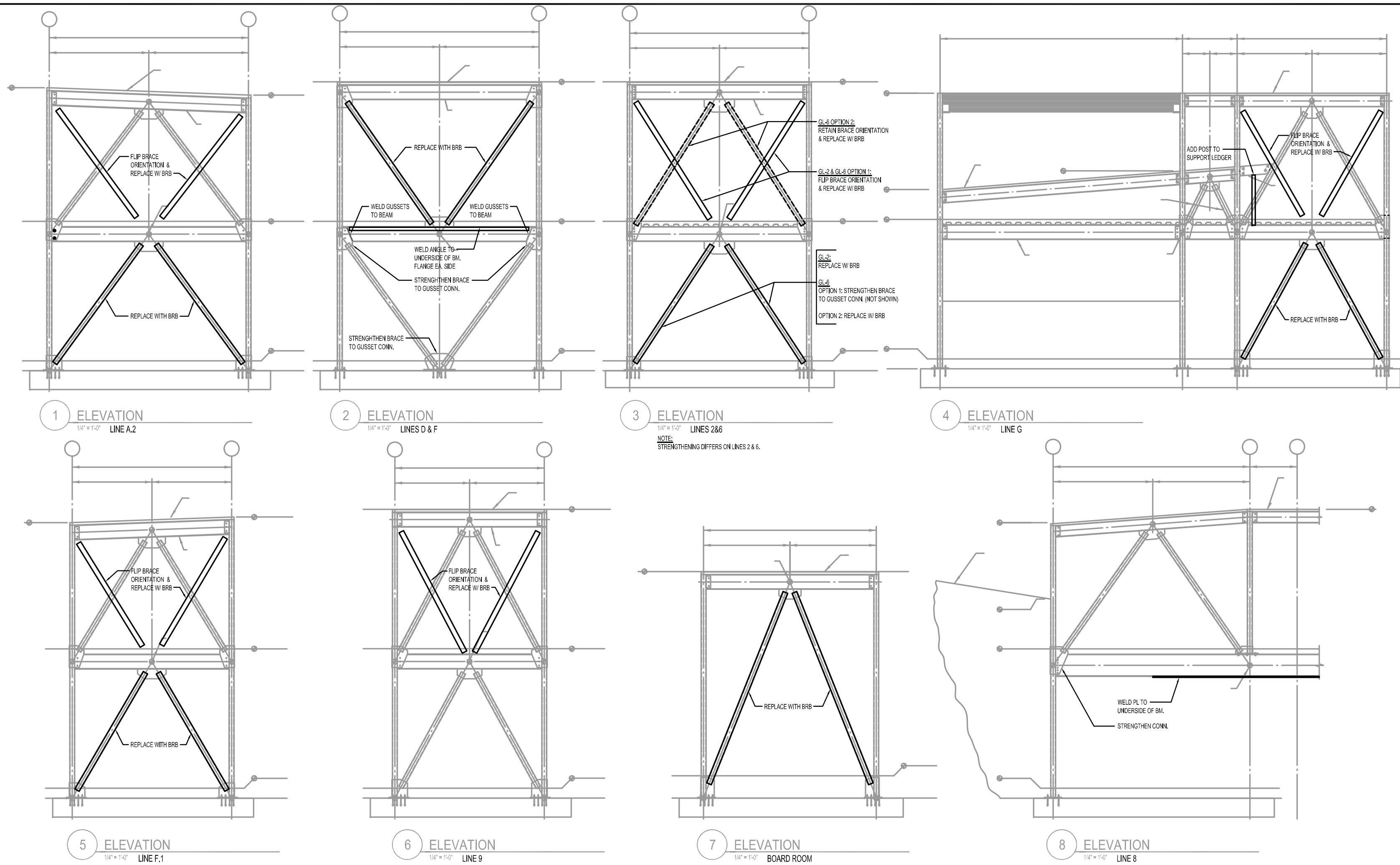
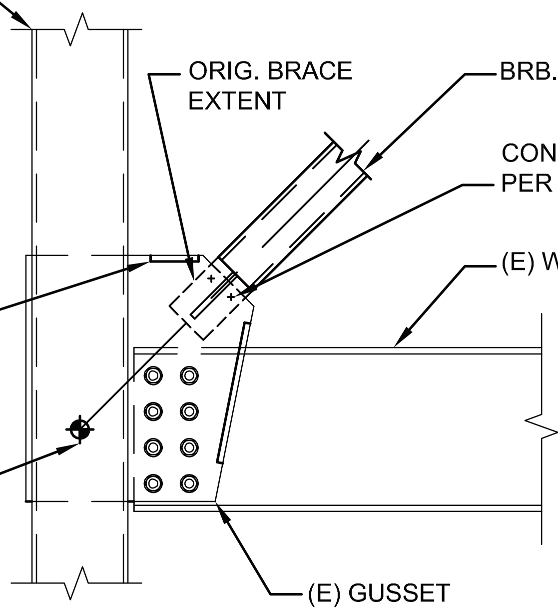


Figure 2-5: Administration Building, Braced Frame Elevations



NOTE:

1. SIM @ CHEVRON LOCATIONS AND AT BASE CONNECTION.
2. REMOVE (E) BRACE AS REQ. TO INSTALL NEW BRB. DO NOT DAMAGE (E) GUSSET WHEN REMOVING BRACES. GRIND (E) WELD METAL FLUSH WITH GUSSET AS REQ. TO INSTALL (N) BRB.
3. REMOVE & REPLACE (E) CONC. ELEVATED SLAB & S.O.G. AS REQ. FOR BRB INSTALLATION .

Figure 2-6: Typical BRB to (E) Gusset Connection

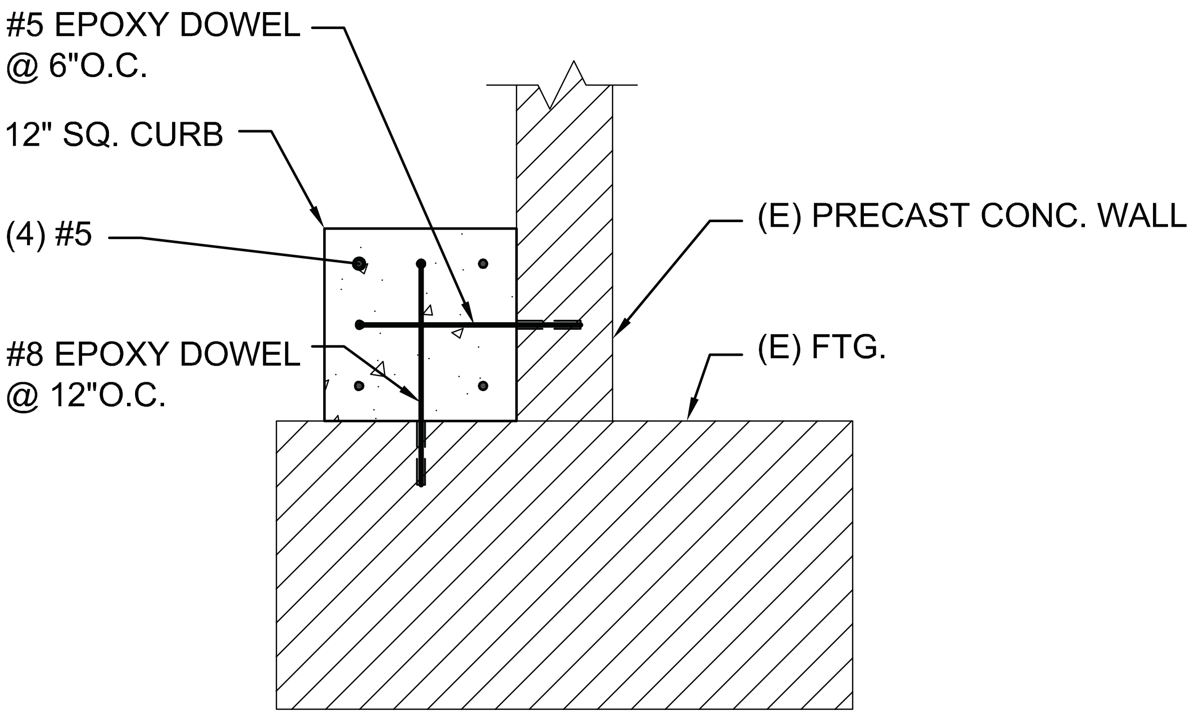
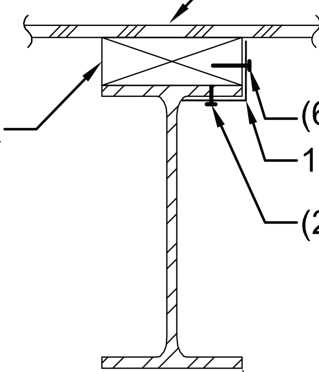


Figure 2-7: Concrete Curb @ (E) Precast Wall



NOTE:
1. ALT. SIDES OF BEAM.
2. INSTALL ANGLE BTW. (E) 1/2"Ø CARRIAGE BOLTS

Figure 2-8: (E) Nailing Strengthening

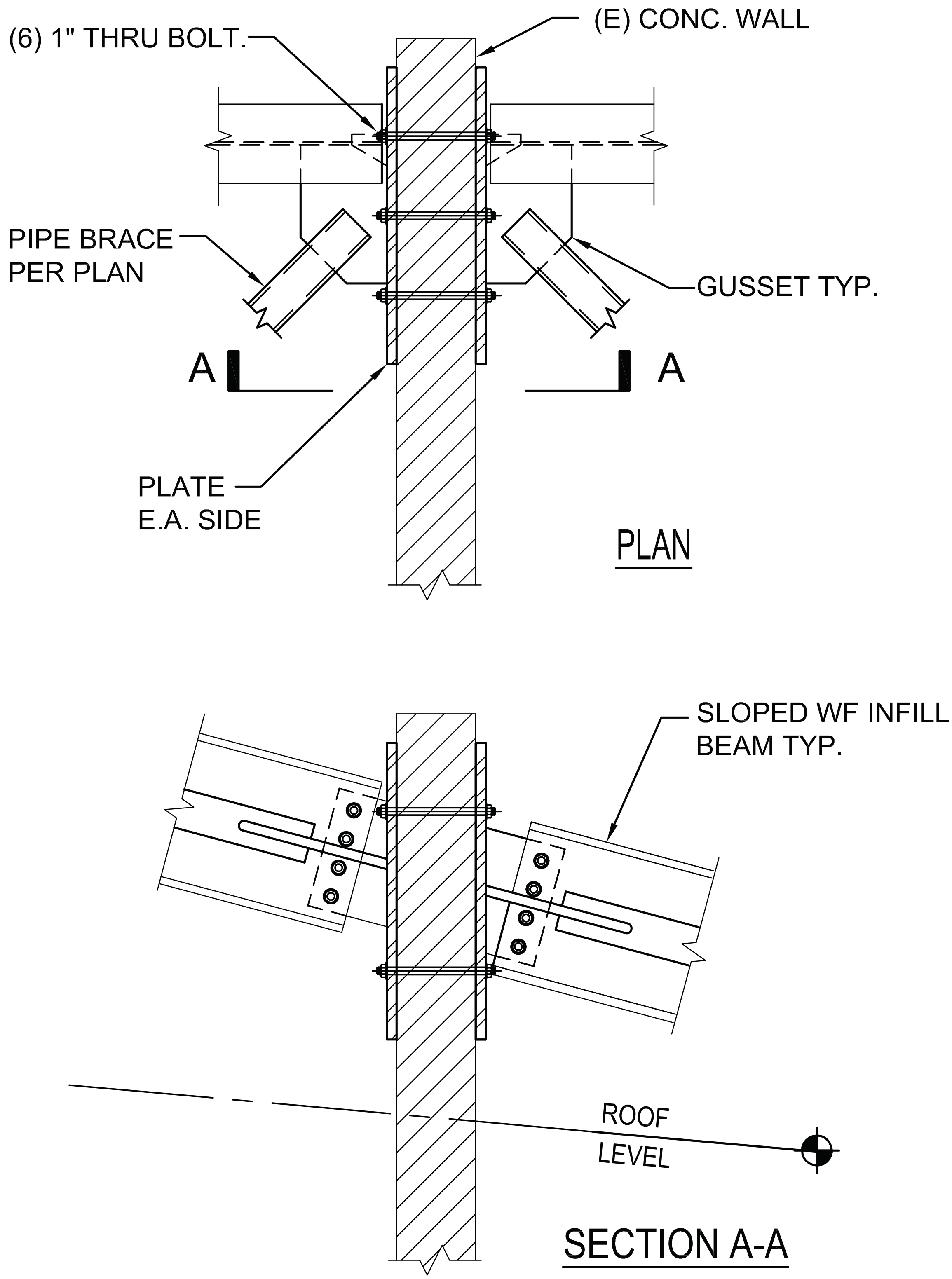


Figure 2-9: Connecting at Sloped Grid

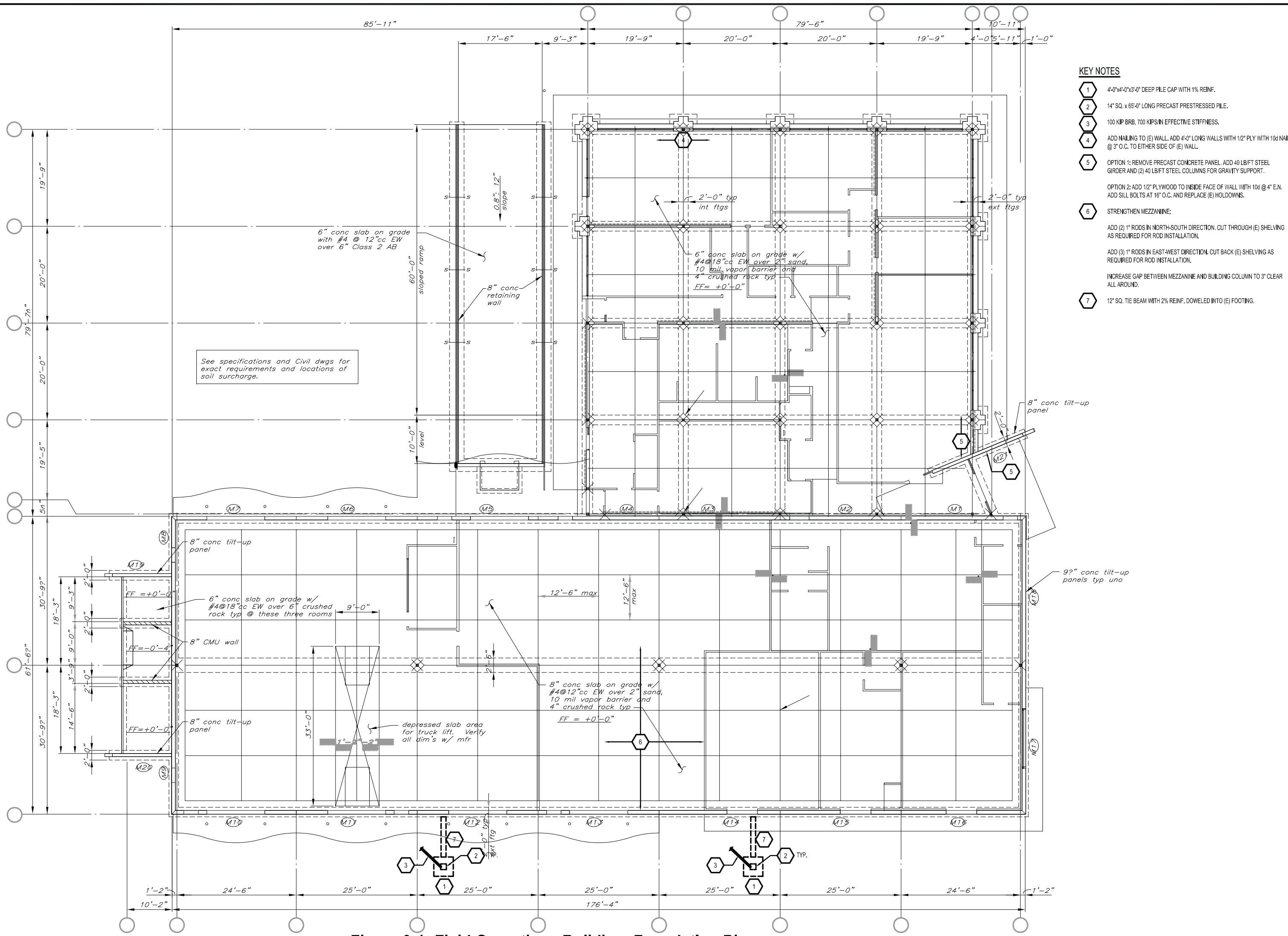


Figure 3-1: Field Operations Building, Foundation Plan

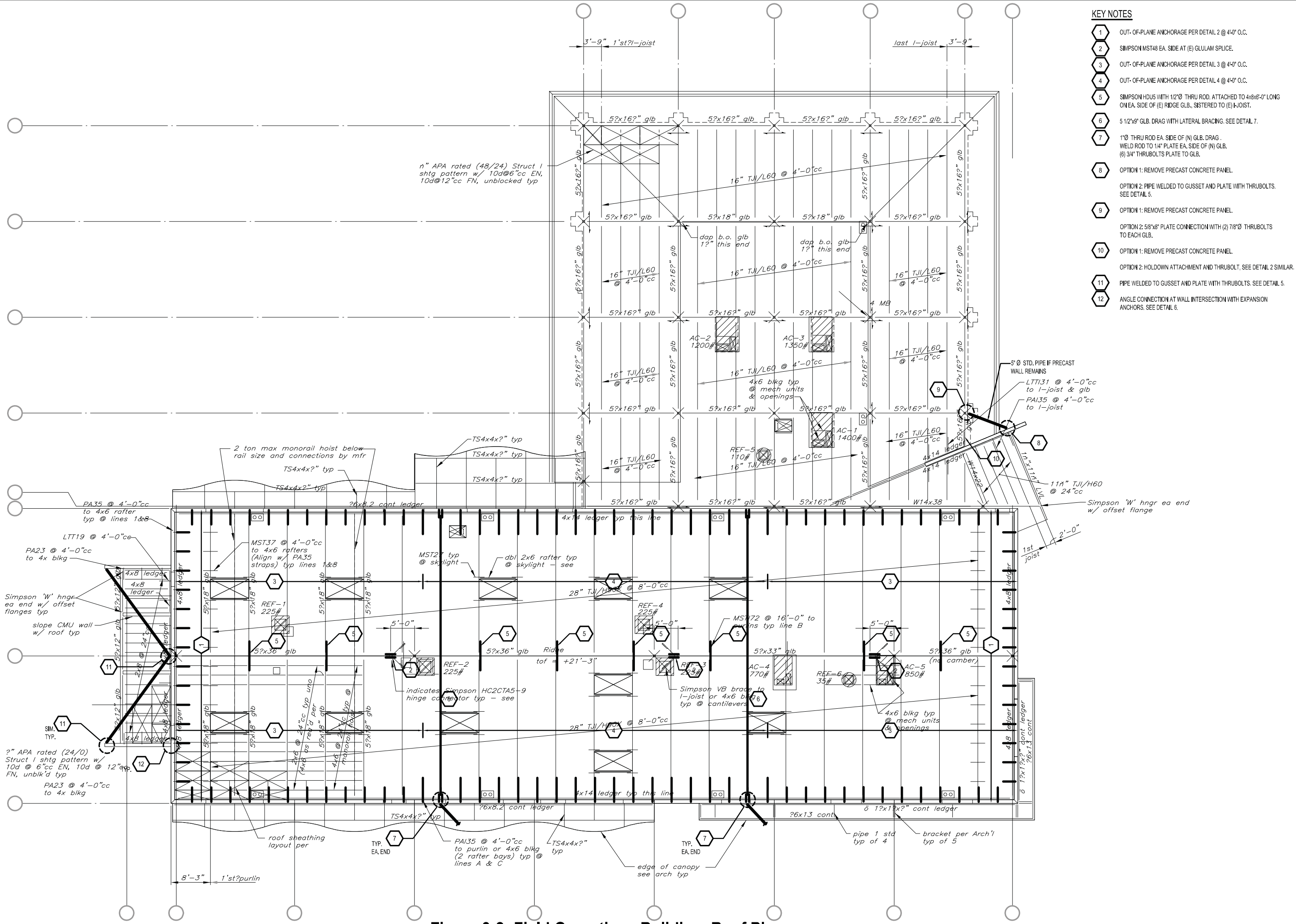


Figure 3-2: Field Operations Building, Roof Plan

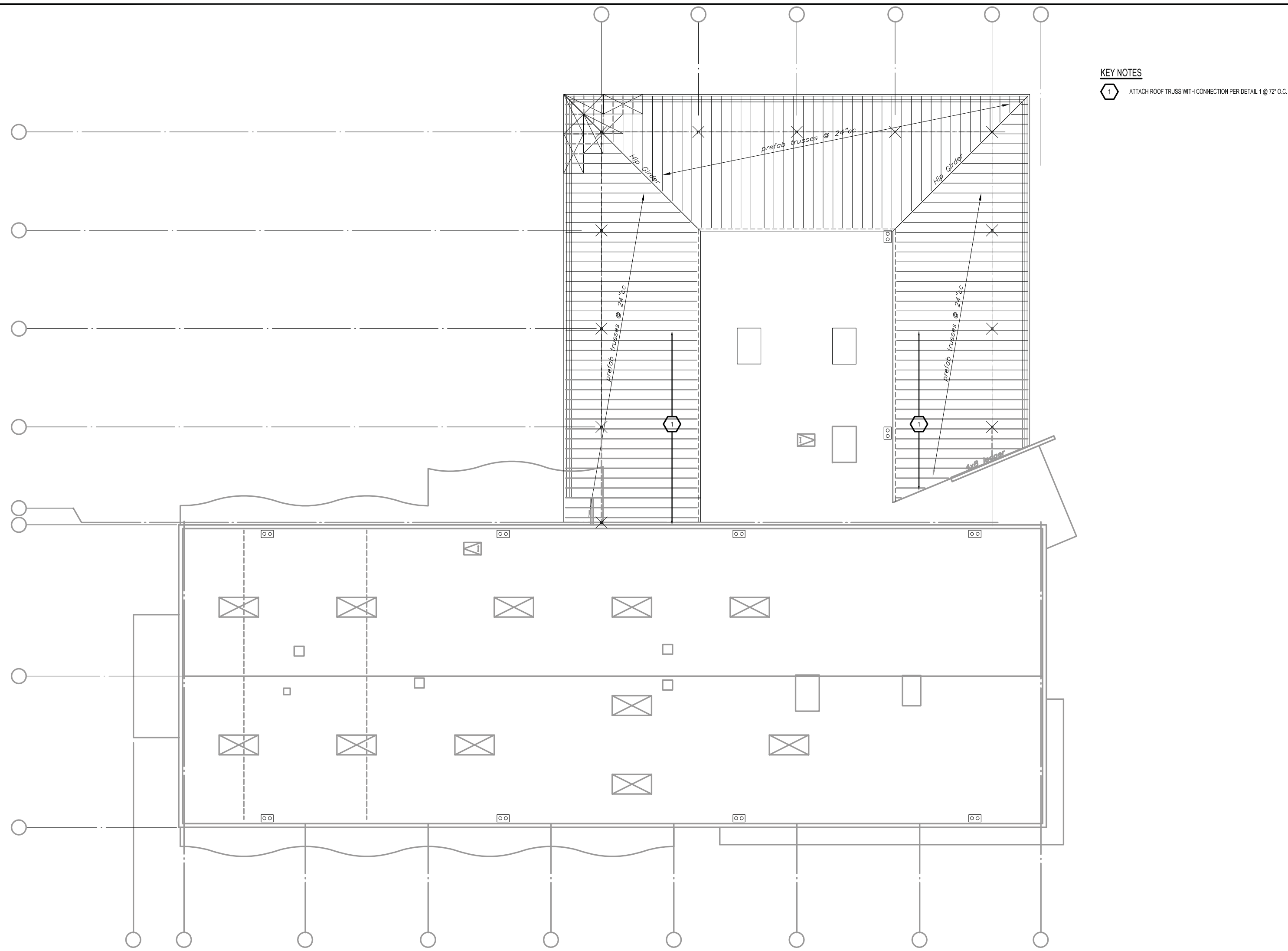


Figure 3-3: Field Operations Building, Mansard Roof Plan

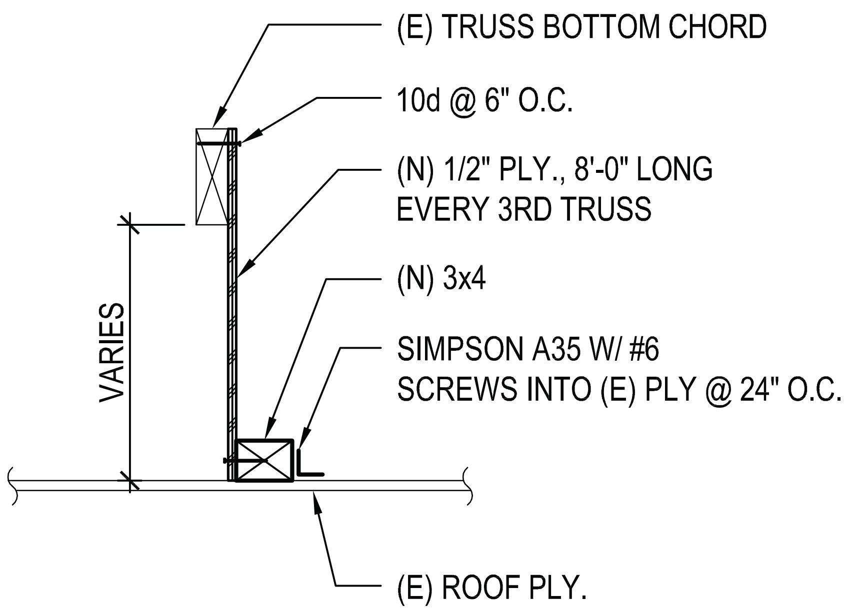


Figure 3-4: Mansard Roof Attachment

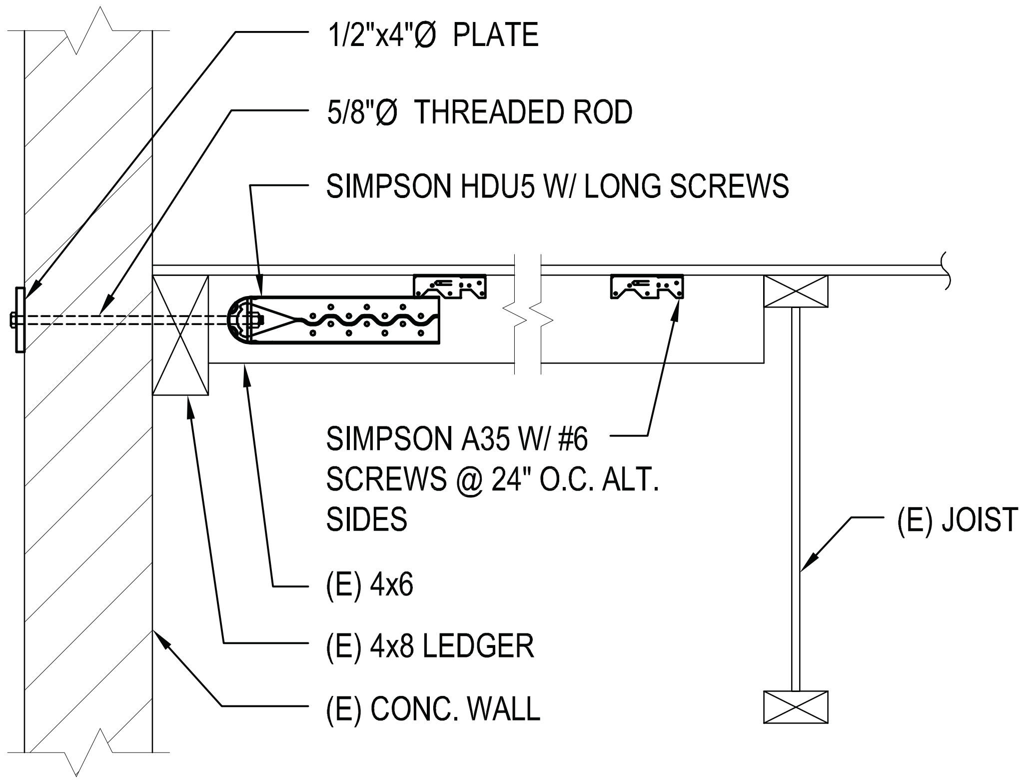


Figure 3-5: Wall Anchorage, Perpendicular to Joists

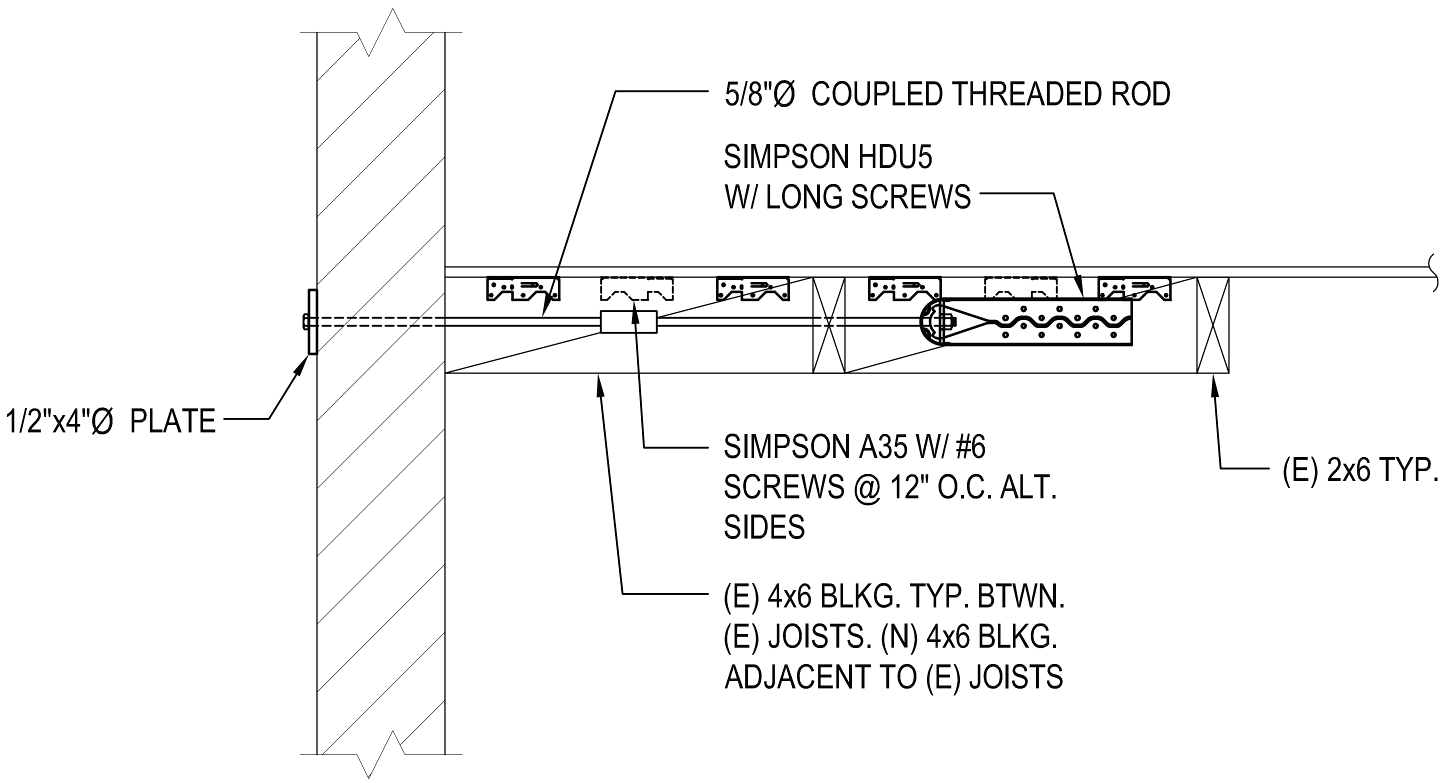


Figure 3-6: Short Wall Anchorage, Parallel to Joists

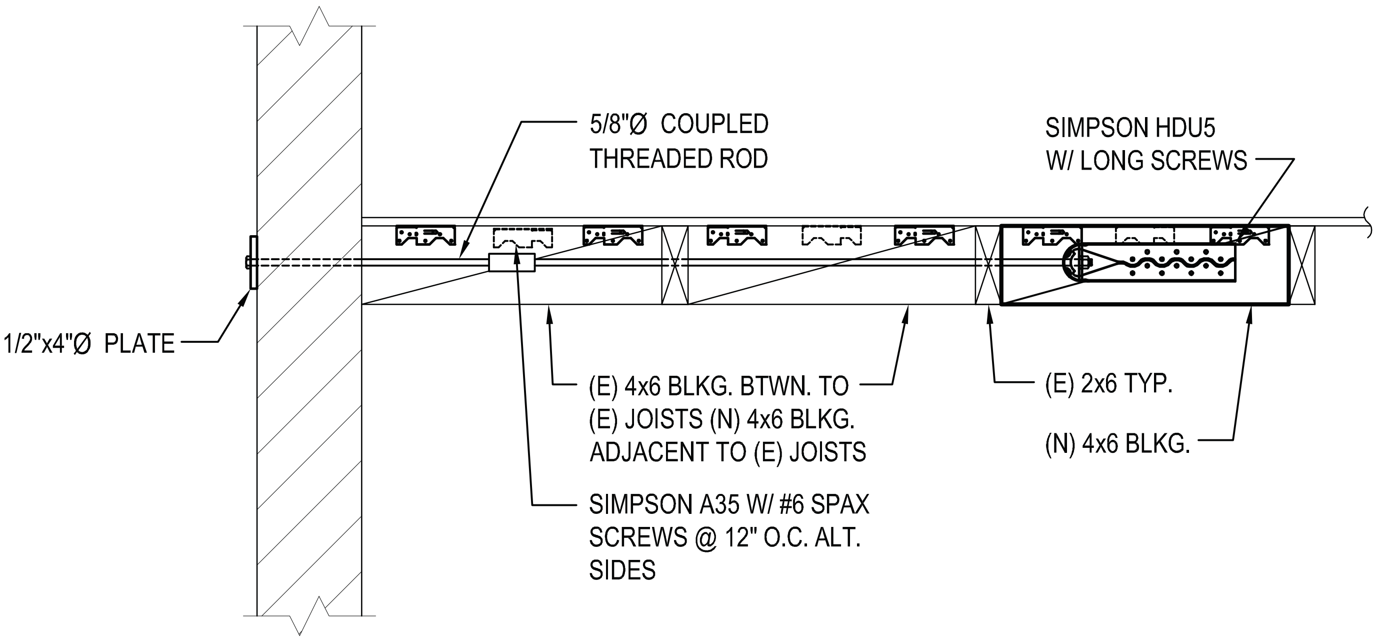


Figure 3-7: Long Wall Anchorage, Parallel to Joists

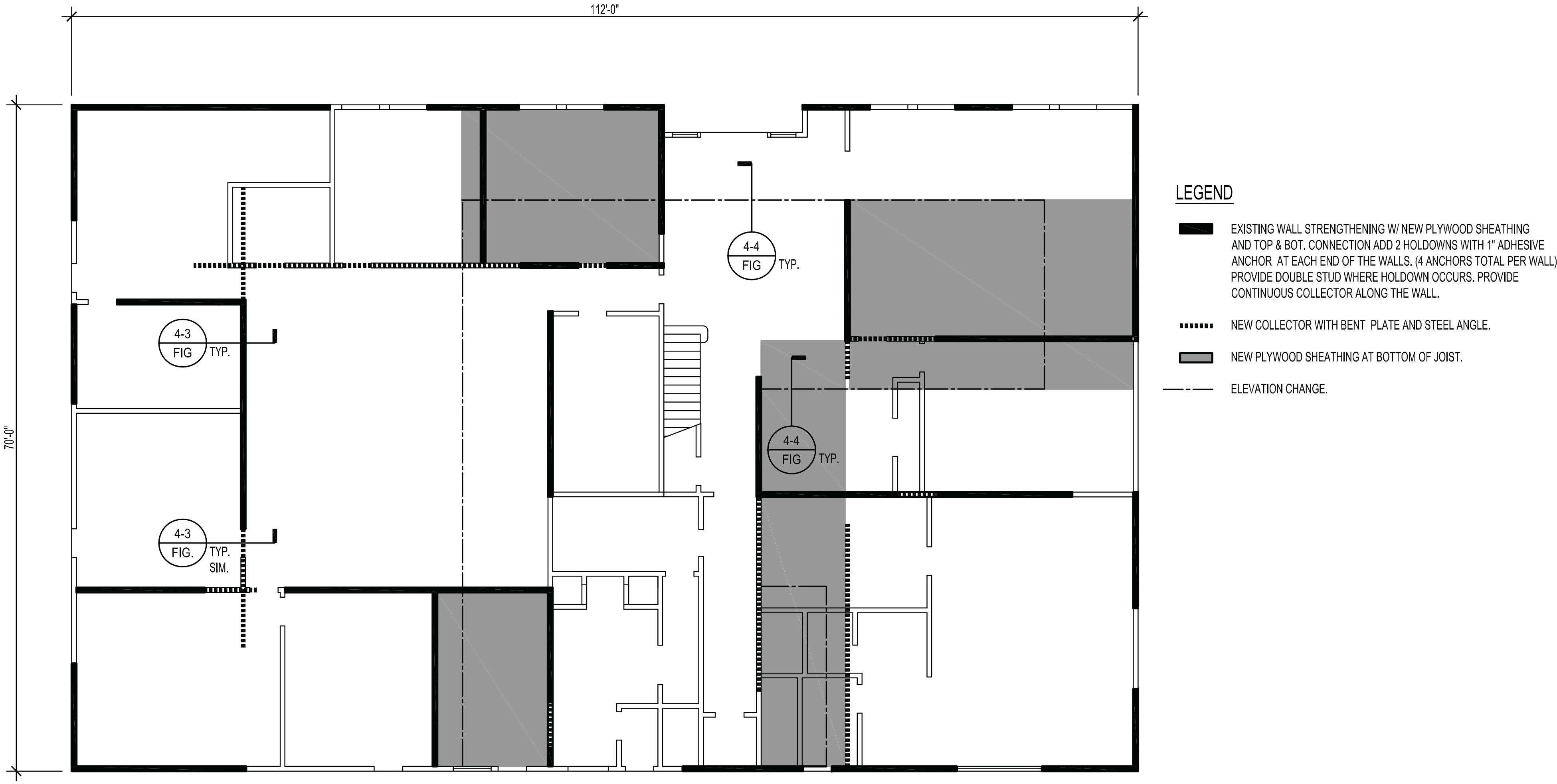


Figure 4-1: Control Building, Ground Floor Plan

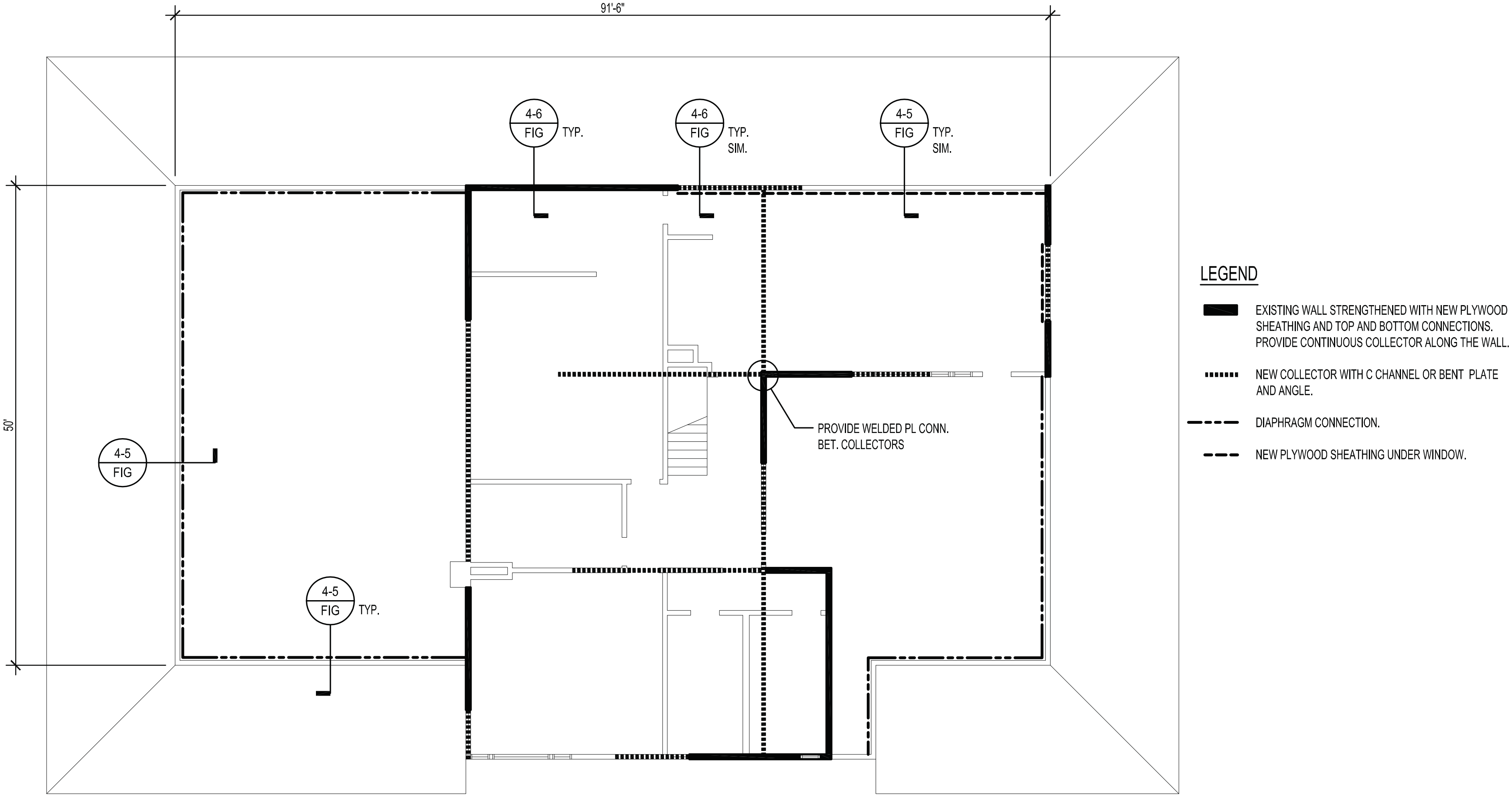


Figure 4-2: Control Building, Second Floor Plan

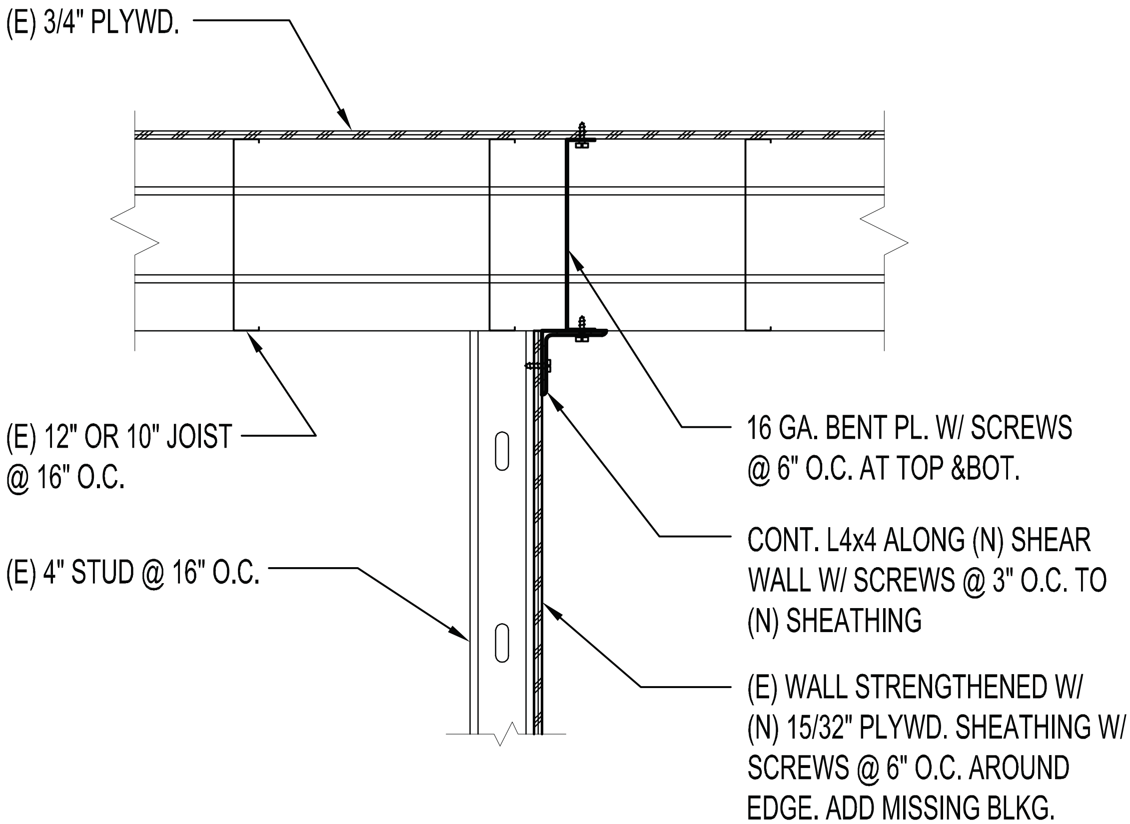


Figure 4-3: (E) Wall Strengthening Details

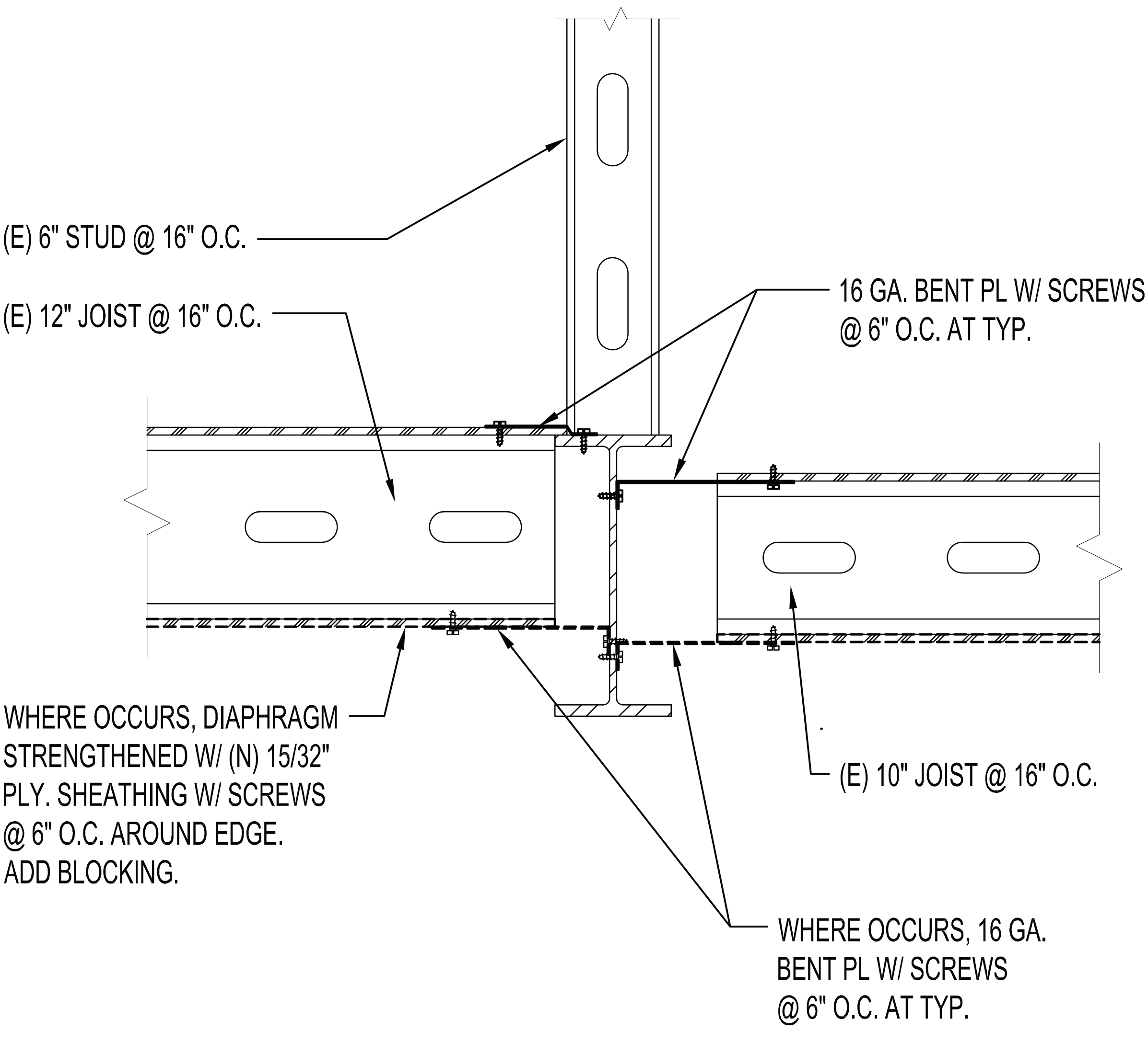


Figure 4-4: Diaphragm Strengthening Details

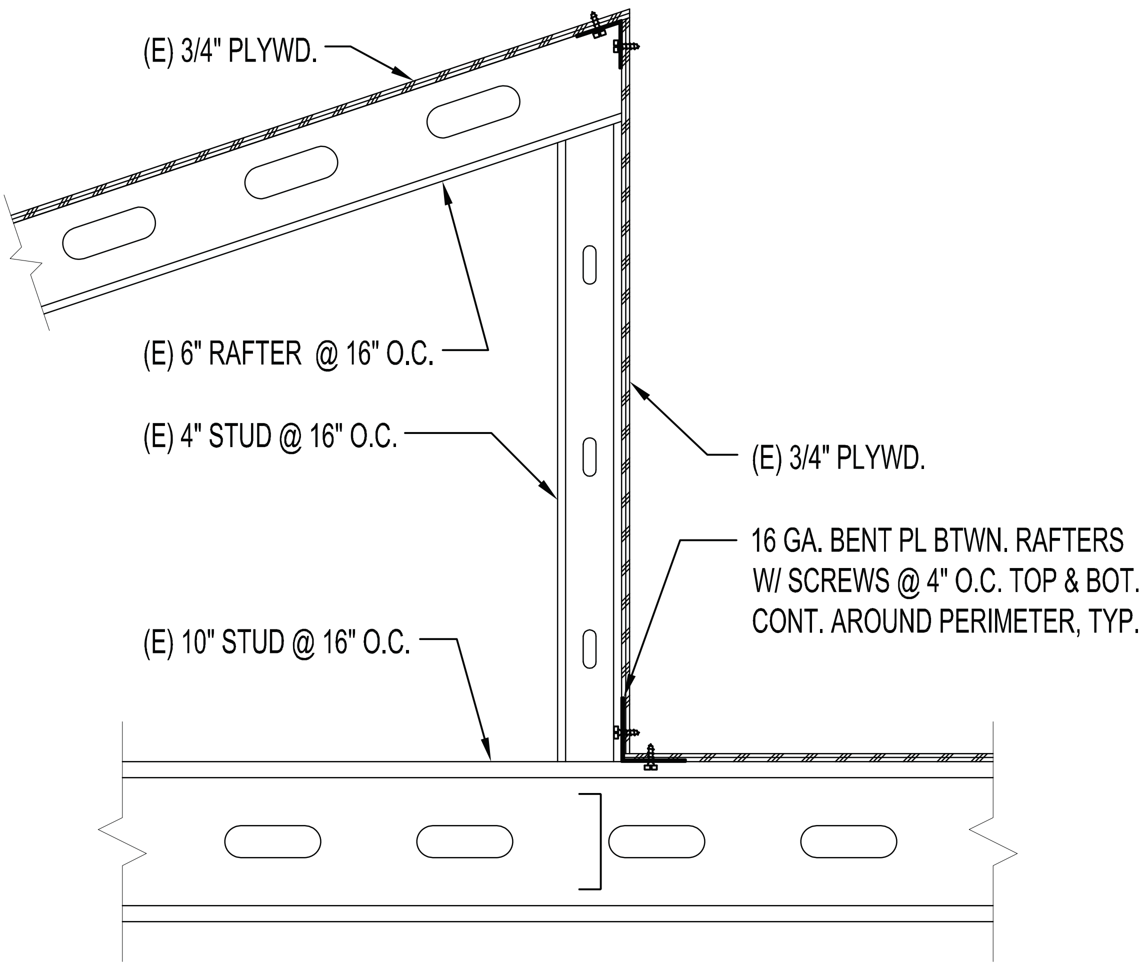


Figure 4-5: Diaphragm Connection Strengthening Details

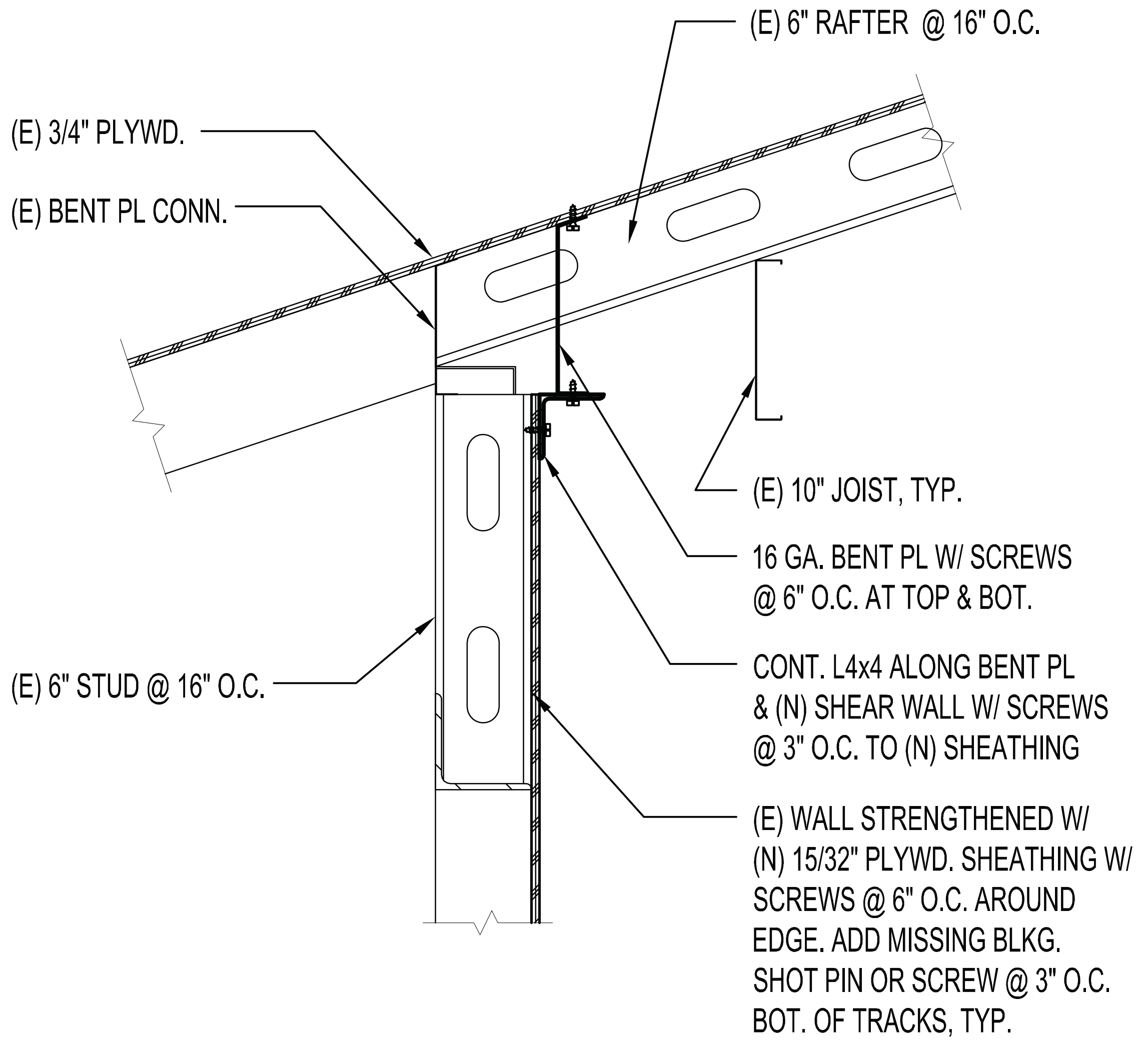


Figure 4-6: (E) Wall Strengthening Details

APPENDIX B - Conceptual Design Construction Cost Estimate

Union Sanitary District

Union City, California

Conceputal Design Estimate
Statement of Probable Cost
May 27, 2015
Cumming Project No. 15-00XXX.00

Prepared for Degenkolb Engineers

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PHONE: 415-748-3080 • FAX: 415-748-3090

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INTRODUCTION

Project Description

This estimate has been prepared, pursuant to an agreement between Degenkolb Engineers and Cumming Corporation, for the purpose of establishing probable construction costs for seismic upgrades to four buildings for the Union Sanitary District in Union City, California.

Basis of Estimate

This estimate is based on the information listed below, as provided by Degenkolb Engineers:

1 Concept Documents for the following buildings
Primary Clarifiers Building - S1.0, S2.0, dated Apr 30, 2015
Control Building S1.0, S1.1, S3.0 dated March 13, 2015, A2.1D, A2.1A, 201A, 202 & 204
Field Operations S1.0, S1.1, S1.2 dated March 11,2015, Detail 1, 2, 3, 4, 5, 6 & 7
Administration Building S1.0, S1.1, S1.2, S1.3, S2.0 dated March 11, 2015, Detail 1, 2, 3, 4, 5, 6, 7

Construction Schedule
Costs included herein have been based upon the following construction period:

	Constr. Start	Constr. Duration
Seismic Upgrades	**1/1/2016**	**12 months**

Any costs for excessive overtime to meet accelerated schedule milestone dates are not included in this estimate.

Basis for Quantities
Wherever possible, this estimate has been based upon the actual measurement of different items of work. For the remaining items, parametric measurements were used in conjunction with references from other projects of a similar nature.

Basis for Unit Costs
Unit costs as contained herein are based on current San Francisco, CA - Prevailing Wage prices. Subcontractor’s overhead and profit is included in each line item unit cost. This overhead and profit covers each subcontractor’s cost for labor burden, materials and equipment sales taxes, field overhead, home office overhead, and profit. The general contractor’s overhead and profit is shown separately on the Summary.

Sources for Pricing
This estimate was prepared by a team of qualified cost consultants experienced in estimating construction costs at all stages of design. These consultants have used pricing data from Cumming’s database for construction, updated to reflect current conditions in the Union City, California area. In some cases, quotes were solicited from outside sources to substantiate in-house pricing data.

Subcontractor's Mark-ups
Depending on the trade, subcontractor mark-ups can range from 5% to 15% of the raw cost for that particular item of work. It should be noted that Design Assist Sub Contractors may influence Sub Contractor costs.

INTRODUCTION

Design Allowances

An allowance of 15**%** for undeveloped design details has been included in the summary of this estimate. As the design of each system is further developed, details which historically increase cost become apparent and must be incorporated into the estimate. This allowance is intended to cover the cost of such details.

General Contractor's Overhead and Profit

Jobsite general conditions, home office overhead, profit, and bond are shown on the Summary of this estimate.

Escalation Allowance

Escalation has been included on the summary level and calculated to the construction mid-point based on the following parameters:

Estimated start date:	Jan-16
Estimated completion date:	Jan-17
Mid-point of construction:	Jul-16

Year	Rate
2015	7.0%
2016	6.0%
2017	5.0%
2018	3.0%

Construction Contingency

It is prudent for all program budgets to include an allowance for change orders which occur during construction. These change orders normally increase the cost of the project. It is recommended that for projects of this nature a **10% - 15%** contingency is carried in this respect.

Bidding

Historical cost data indicates that the number of competitive bids obtained has the following effect:

1 bid	add	15% to 40%
2 to 3 bids	add	8% to 12%
4 to 6 bids	deduct/add	-4% to +4%
7 to 8 bids	deduct	5% to 7%

It is understood that the works will be competitively bid.

It is our opinion that there is potential to incur a premium cost by limiting the bidding pool to a list of preferred bidders .

Items Included in the Estimate

Subcontractor’s and general contractor’s mark-ups.
Design allowances.
Escalation.
Construction Contingency.

Items Excluded from the Base Estimate

Professional fees, inspections and testing.
Escalation beyond midpoint of construction, (07/02/16)
Plan check fees and building permit fees.
Modifications to utilities

INTRODUCTION

Major site and building structures demolition unless noted in body of estimate.
Costs of hazardous material surveys, abatements, and disposals unless noted in estimate.
Costs of off-site construction unless noted in estimate.

Items Affecting the Cost Estimate

1. Items which may change the estimated construction cost include, but are not limited to:
2. Modifications to the scope of work included in this estimate.
3. Restrictive technical specifications or excessive contract conditions.
4. Any specified item of equipment, material, or product that cannot be obtained from at least 3 different sources.
5. Any other non-competitive bid situations.
6. Bids delayed beyond the projected schedule.

Statement of Probable Cost

Cumming has no control over the cost of labor and materials, the general contractor's or any subcontractor's method of determining prices, or competitive bidding and market conditions. This opinion of the probable cost of construction is made on the basis of the experience, qualifications, and best judgment of a professional consultant familiar with the construction industry. Cumming, however, cannot and does not guarantee that proposals, bids, or actual construction costs will not vary from this or subsequent cost estimates.

Cumming has no control over the quality, completeness, intricacy, constructability, or coordination of design documents. Cumming also has no control over the amount of funds available for the project. We, therefore, cannot be responsible for any design revision costs incurred in the event that this estimate is in excess of the budget.

Cumming's staff of professional cost consultants has prepared this estimate in accordance with generally accepted principles and practices. This staff is available to discuss its contents with any interested party.

Recommendations for Cost Control

Cumming recommends that the Owner and the Architect carefully review this entire document to insure that it reflects their design intent. Requests for modifications of any apparent errors or omissions to this document must be made to Cumming within ten days of receipt of this estimate, otherwise, it will be understood that the contents have been concurred with and accepted. If the project is over budget, or there are unresolved budgeting issues, alternate systems/schemes should be evaluated before proceeding into further design phases.

Master Summary

CONSTRUCTION COST SUMMARY

Element	Area (sf)	Cost / sf	Total
Primary Clarifiers Building	26,430	$128.30	$3,391,059
Control Building	11,855	$158.35	$1,877,226
Field Operations Building	19,065	$81.92	$1,561,882
Administration Building	28,328	$165.55	$4,689,713
TOTAL ESTIMATED CONSTRUCTION COST			**$11,519,879**

Schedule of Areas and Control Quantities

Schedule of Areas

1. Enclosed Areas (x 100%)

Primary Clarifiers	26,430
Control Building	11,855
Field Operations	19,065
Administration Building	28,328
Total Enclosed	**85,678**

2. Unenclosed Areas (x 50%)

Not applicable	N/A
Total Unenclosed	
Total Gross Roof Area	**85,678**

PRIMARY CLARIFIERS BUILDING

PROJECT SUMMARY - PRIMARY CLARIFIERS BUILDING

Element		Subtotal	Total	Cost / SF	Cost / SF
A) Shell (1-5)			**$2,033,591**		**$76.94**
1 Foundations					
2 Vertical Structure					
3 Floor & Roof Structures		$1,635,819		$61.89	
4 Exterior Cladding					
5 Roofing and Waterproofing		$397,772		$15.05	
B) Interiors (6-7)					
6 Interior Partitions, Doors and Glazing					
7 Floor, Wall and Ceiling Finishes					
C) Equipment and Vertical Transportation (8-9)					
8 Function Equipment and Specialties					
9 Stairs and Vertical Transportation					
D) Mechanical and Electrical (10-13)			**$20,000**		**$0.76**
10 Plumbing Systems					
11 Heating, Ventilation and Air Conditioning					
12 Electrical Lighting, Power and Communications		$20,000		$0.76	
13 Fire Protection Systems					
E) Site Construction (14-16)			**$85,796**		**$3.25**
14 Site Preparation and Demolition		$85,796		$3.25	
15 Site Paving, Structures & Landscaping					
16 Utilities on Site					
Subtotal			**$2,139,386**		$80.95
General Conditions	9.0%		**$192,545**		$7.29
Subtotal			**$2,331,931**		$88.23
Bonds & Insurance	2.0%		**$46,639**		$1.76
Subtotal			**$2,378,570**		$90.00
General Contractor Fee	5.0%		**$118,928**		$4.50
Subtotal			**$2,497,498**		$94.49
Design Contingency	15.0%		**$374,625**		$14.17
Subtotal			**$2,872,123**		$108.67
Escalation to MOC, 07/02/16	7.33%		**$210,658**		$7.97
Subtotal			**$3,082,781**		$116.64
Construction Contingency	10.00%		**$308,278**		$11.66
TOTAL ESTIMATED CONSTRUCTION COST			**$3,391,059**		**$128.30**

Total Area: **26,430 SF**

DETAIL ELEMENTS - PRIMARY CLARIFIERS BUILDING

Element	Quantity	Unit	Unit Cost	Total
1 Foundations				
Not applicable				
Total - Foundations				
2 Vertical Structure				
Not applicable				
Total - Vertical Structure				
3 Floor & Roof Structures				
Roof Structure				
Detail 1				
Formwork	2,074	sf	$30.00	$62,223
Concrete, 4,000 PSI	50	cy	$400.00	$20,050
Pump concrete	50	cy	$40.00	$2,005
Reinforcement (4) #10	6,007	lbs	$2.00	$12,014
Reinforcement #5 @ 6" O.C	2,184	lbs	$2.00	$4,368
Reinforcement #5	364	lbs	$2.00	$728
Inserting (2) #5 thru web of (E) BM mechanically coupled between BMS	728	lbs	$2.00	$1,456
Drilling holes through precast tee	176	ea	$30.00	$5,280
Coupling to rebar	176	ea	$7.50	$1,320
Adhesive Dowels (2) #4 @ 6" O.C	1,396	ea	$57.00	$79,572
Adhesive Dowel #5 @ 6" O.C	698	ea	$57.00	$39,786
Roughen (E) concrete surface	1,750	sf	$8.50	$14,875
Detail 2				
Formwork	1,356	sf	$30.00	$40,670
Reinforcement (4) #10	1,534	lbs	$2.00	$3,068
Concrete, 4,000 PSI	34	cy	$400.00	$13,613
Pump concrete	34	cy	$40.00	$1,361
Reinforcement #5	1,221	lbs	$2.00	$2,442
Reinforcement (4) #10	5,198	lbs	$2.00	$10,396
Reinforcement #10 @ 12" O.C (assumed)	1,300	lbs	$2.00	$2,599
Reinforcement #6 @1'-0 O.C, connecting to coupling	907	lbs	$2.00	$1,814
Coupling to existing rebar	304	ea	$5.00	$1,520
Adhesive Dowels w/T Head (2) #5 @ 12" O.C	302	ea	$75.00	$22,650
Chip min 6" to expose (E) bar	302	lf	$173.50	$52,397
Roughen (E) concrete surface	604	sf	$8.50	$5,134

DETAIL ELEMENTS - PRIMARY CLARIFIERS BUILDING

Element	Quantity	Unit	Unit Cost	Total
Detail 3				
Formwork	2,183	sf	$30.00	$65,498
Concrete, 4,000 PSI	53	cy	$400.00	$21,390
Pump concrete	53	cy	$40.00	$2,139
Reinforcement (2) #3	605	lbs	$2.00	$1,211
Reinforcement #5 6" OC W/7" embed (ext wall)	2,720	ea	$57.00	$155,040
Reinforcement #4 6" O.C W/ 4 1/4 embed (int wall)	320	ea	$57.00	$18,240
Reinforcement (2) #6 T&B	4,836	lbs	$2.00	$9,673
Reinforcement #4 1'-6" 0.C	477	lbs	$2.00	$954
Reinforcement #3 1'-0" 0.C	630	lbs	$2.00	$1,261
Roughen (E) concrete surface	1,272	sf	$8.50	$10,811
Detail 8				
Formwork	1,090	sf	$30.00	$32,712
Concrete, 4,000 PSI	20	cy	$400.00	$7,819
Pump concrete	20	cy	$40.00	$782
Reinforcement (4) #10	2,616	lbs	$2.00	$5,232
Reinforcement #5 @ 1'-0 O.C	151	ea	$97.00	$14,647
Reinforcement #6 @1'-0 O.C, mechanically coupled to (E)	472	lbs	$2.00	$945
Drilling holes through precast tee	151	ea	$30.00	$4,530
Coupling to rebar	151	ea	$7.50	$1,133
Adhesive Dowels (4) #4 @ 1'-0 O.C	608	ea	$57.00	$34,656
Chip min 6" to expose (E) bar	304	lf	$58.50	$17,784
Roughen (E) concrete surface	708	sf	$8.50	$6,021
FRP				
BCC FRP ($25 / sq ft/layer)	1	ls	$550,000.00	$550,000
Additional Bracing				
(4) 1" dia rod bracing in each directions (8 total) in the mezzanine structure along interior wall	1,216	lf	$100.00	$121,600
Misc				
Shoring/propping	304	lf	$100.00	$30,400
Working platform	4,000	sf	$30.00	$120,000

Total - Floor & Roof Structures ***$1,635,819***

4 Exterior Cladding

Not applicable

Total - Exterior Cladding

DETAIL ELEMENTS - PRIMARY CLARIFIERS BUILDING

Element	Quantity	Unit	Unit Cost	Total
5 Roofing and Waterproofing				
Roof Deck				
Allowance for replacement roof covering and alterations to existing drainage following installation of curb	26,430	sf	$15.05	$397,772
Total - Roofing and Waterproofing				***$397,772***
6 Interior Partitions, Doors and Glazing				
Not applicable				
Total - Interior Partitions, Doors and Glazing				
7 Floor, Wall and Ceiling Finishes				
Not applicable				
Total - Floor, Wall and Ceiling Finishes				
8 Function Equipment and Specialties				
Not applicable				
Total - Function Equipment and Specialties				
9 Stairs and Vertical Transportation				
Not applicable				
Total - Stairs and Vertical Transportation				
10 Plumbing Systems				
Not applicable				
Total - Plumbing Systems				
11 Heating, Ventilation and Air Conditioning				
Not applicable				
Total - Heating, Ventilation and Air Conditioning				

DETAIL ELEMENTS - PRIMARY CLARIFIERS BUILDING

Element	Quantity	Unit	Unit Cost	Total
12 Electrical Lighting, Power and Communications				
Allowance for relocation of services as necessary	1	sf	$20,000.00	$20,000.00
Total - Electrical Lighting, Power and Communications				***$20,000***
13 Fire Protection Systems				
Not applicable				
Total - Fire Protection Systems				
14 Site Preparation and Demolition				
Selective Demolition				
Surveying and cutting joint between double tee units	302	lf	$29.75	$8,985
Dislodge double tee unit for removal	302	lf	$58.50	$17,667
Lift double tee unit and swing to ground	4	ea	$1,036.00	$4,144
Remove and dispose of double tee	4	ea	$5,000.00	$20,000
Crane	1	ls	$35,000.00	$35,000
Total - Site Preparation and Demolition				***$85,796***
15 Site Paving, Structures & Landscaping				
Not applicable				
Total - Site Paving, Structures & Landscaping				
16 Utilities on Site				
Not applicable				
Total - Utilities on Site				

CONTROL BUILDING

PROJECT SUMMARY - CONTROL BUILDING

Element		Subtotal	Total	Cost / SF	Cost / SF
A) Shell (1-5)					
1 Foundations					
2 Vertical Structure					
3 Floor & Roof Structures					
4 Exterior Cladding					
5 Roofing and Waterproofing					
B) Interiors (6-7)			**$795,356**		**$67.09**
6 Interior Partitions, Doors and Glazing		$795,356		$67.09	
7 Floor, Wall and Ceiling Finishes					
C) Equipment and Vertical Transportation (8-9)					
8 Function Equipment and Specialties					
9 Stairs and Vertical Transportation					
D) Mechanical and Electrical (10-13)			**$213,390**		**$18.00**
10 Plumbing Systems		$29,638		$2.50	
11 Heating, Ventilation and Air Conditioning		$59,275		$5.00	
12 Electrical Lighting, Power and Communications		$88,913		$7.50	
13 Fire Protection Systems		$35,565		$3.00	
E) Site Construction (14-16)			**$175,578**		**$14.81**
14 Site Preparation and Demolition		$175,578		$14.81	
15 Site Paving, Structures & Landscaping					
16 Utilities on Site					
Subtotal			**$1,184,324**		$99.90
General Conditions	9.0%		**$106,589**		$8.99
Subtotal			**$1,290,913**		$108.89
Bonds & Insurance	2.0%		**$25,818**		$2.18
Subtotal			**$1,316,731**		$111.07
General Contractor Fee	5.0%		**$65,837**		$5.55
Subtotal			**$1,382,568**		$116.62
Design Contingency	15.0%		**$207,385**		$17.49
Subtotal			**$1,589,953**		$134.12
Escalation to MOC, 07/02/16	7.33%		**$116,616**		$9.84
Subtotal			**$1,706,569**		$143.95
Construction Contingency	10.00%		**$170,657**		$14.40

TOTAL ESTIMATED CONSTRUCTION COST	**$1,877,226**	**$158.35**

Total Area: **11,855 SF**

DETAIL ELEMENTS - CONTROL BUILDING

Element	Quantity	Unit	Unit Cost	Total
1 Foundations				
Not applicable				
Total - Foundations				
2 Vertical Structure				
Not applicable				
Total - Vertical Structure				
3 Floor & Roof Structures				
Not applicable				
Total - Floor & Roof Structures				
4 Exterior Cladding				
Not applicable				
Total - Exterior Cladding				
5 Roofing and Waterproofing				
Not applicable				
Total - Roofing and Waterproofing				
6 Interior Partitions, Doors and Glazing				
Remove existing gypsum board to walls	637	lf	$35.00	$22,299
Remove existing gypsum board to walls under windows	46	lf	$12.00	$552
Remove 2 rows of ceiling T-bar 2 x 4, set aside for reuse and reinstall following completion of work	3,993	sf	$15.25	$60,886
Remove ceiling T-bar 2 x 4, set aside for reuse and reinstall following completion of work	1,777	sf	$15.25	$27,094
Remove glue on ceiling tiles and dispose	4,106	sf	$15.25	$62,612
Plywood sheathing 15/32 to walls with screws at 6" O.C around edge and blocking	7,645	sf	$21.40	$163,607
Plywood sheathing 15/32 to ceiling with screws at 6" O.C around edge and blocking	1,777	sf	$21.40	$38,022
Plywood sheathing 15/32 under windows with screws at 6" O.C around edge and blocking	184	sf	$21.40	$3,938
1" dia adhesive anchors @ 6'-0 at bottom with double studs	683	lf	$36.13	$24,678
New Collector with 16GA Bent Plate w/ screws @ 6"-0 @ Top & Bottom and Continuous L4x4 along new shear wall w /screws @ 3" O.C	710	lf	$67.57	$47,975
New Collector 4 nr 16GA bent plate w/ screws at elevation change	996	lf	$38.64	$38,480

DETAIL ELEMENTS - CONTROL BUILDING

Element	Quantity	Unit	Unit Cost	Total
16GA bent plate between rafters w/ screws @ 4"-0 @ top & bottom and continuous around perimeter	332	lf	$44.48	$14,768
16GA bent plate w/ screws @ 6"-0 @ top & bottom where angle occurs and continuous L4x4 along new shear wall w/ screws @ 3" OC to new sheathing	245	lf	$53.08	$13,005
Continuous C6 w/ screws @ 6" O.C along existing Joist	245	lf	$50.00	$12,250
Continuous C6 w/ screws @ 6" O.C along existing Joist	59	lf	$50.00	$2,950
Provide welded plate connection between connectors	1	ea	$85.00	$85
5/8" gypsum board, fixed to plywood and painted	7,829	sf	$9.00	$70,463
Lay-in ceiling tile including framing fixed to plywood	1,777	sf	$10.28	$18,270
Gypsum board ceilings, including framing	4,106	sf	$17.80	$73,087
Replace carpet tile to 2nd floor	4,106	sf	$10.00	$41,060
Allowances for repairs to finishes	11,855	sf	$5.00	$59,275
Total - Interior Partitions, Doors and Glazing				***$795,356***
7 Floor, Wall and Ceiling Finishes				
Not applicable				
Total - Floor, Wall and Ceiling Finishes				
8 Function Equipment and Specialties				
Not applicable				
Total - Function Equipment and Specialties				
9 Stairs and Vertical Transportation				
Not applicable				
Total - Stairs and Vertical Transportation				
10 Plumbing Systems				
Allowance for relocation of services	11,855	sf	$2.50	$29,637.50
Total - Plumbing Systems				***$29,638***
11 Heating, Ventilation and Air Conditioning				
Allowance for relocation of services	11,855	sf	$5.00	$59,275.00
Total - Heating, Ventilation and Air Conditioning				***$59,275***

DETAIL ELEMENTS - CONTROL BUILDING

Element	Quantity	Unit	Unit Cost	Total
12 Electrical Lighting, Power and Communications				
Allowance for relocation of services	11,855	sf	$7.50	$88,912.50
Total - Electrical Lighting, Power and Communications				***$88,913***
13 Fire Protection Systems				
Allowance for relocation of services	11,855	sf	$3.00	$35,565.00
Total - Fire Protection Systems				***$35,565***
14 Site Preparation and Demolition				
Miscellaneous				
Provide protection to the existing building and finishes, remove protective lining on completion	11,855	sf	$3.00	$35,565
Allowance for removal and disposal of hazardous materials discovered during the works	1	alw	$50,000.00	$50,000
Temporary barriers and protection to existing	11,855	sf	$2.50	$29,638
Allowance working within an occupied structure - 15% of labor hours	525	hrs	$115.00	$60,375
Total - Site Preparation and Demolition				***$175,578***
15 Site Paving, Structures & Landscaping				
Not applicable				
Total - Site Paving, Structures & Landscaping				
16 Utilities on Site				
Not applicable				
Total - Utilities on Site				

FIELD OPERATIONS

PROJECT SUMMARY - FIELD OPERATIONS

Element		Subtotal	Total	Cost / SF	Cost / SF
A) Shell (1-5)			**$372,504**		**$19.54**
1 Foundations		$65,155		$3.42	
2 Vertical Structure		$59,300		$3.11	
3 Floor & Roof Structures		$248,049		$13.01	
4 Exterior Cladding					
5 Roofing and Waterproofing					
B) Interiors (6-7)			**$133,655**		**$7.01**
6 Interior Partitions, Doors and Glazing		$24,753		$1.30	
7 Floor, Wall and Ceiling Finishes		$108,902		$5.71	
C) Equipment and Vertical Transportation (8-9)					
8 Function Equipment and Specialties					
9 Stairs and Vertical Transportation					
D) Mechanical and Electrical (10-13)			**$254,317**		**$13.34**
10 Plumbing Systems		$30,629		$1.61	
11 Heating, Ventilation and Air Conditioning		$72,158		$3.78	
12 Electrical Lighting, Power and Communications		$117,320		$6.15	
13 Fire Protection Systems		$34,211		$1.79	
E) Site Construction (14-16)			**$224,900**		**$11.80**
14 Site Preparation and Demolition		$224,900		$11.80	
15 Site Paving, Structures & Landscaping					
16 Utilities on Site					
Subtotal			**$985,376**		$51.69
General Conditions	9.0%		**$88,684**		$4.65
Subtotal			**$1,074,060**		$56.34
Bonds & Insurance	2.0%		**$21,481**		$1.13
Subtotal			**$1,095,541**		$57.46
General Contractor Fee	5.0%		**$54,777**		$2.87
Subtotal			**$1,150,318**		$60.34
Design Contingency	15.0%		**$172,548**		$9.05
Subtotal			**$1,322,866**		$69.39
Escalation to MOC, 07/02/16	7.33%		**$97,026**		$5.09
Subtotal			**$1,419,892**		$74.48
Construction Contingency	10.00%		**$141,989**		$7.45
TOTAL ESTIMATED CONSTRUCTION COST			**$1,561,882**		**$81.92**

Total Area: **19,065 SF**

DETAIL ELEMENTS - FIELD OPERATIONS

Element	Quantity	Unit	Unit Cost	Total
1 Foundations				
Excavation & Disposal				
Excavate pile caps & tie beam	6	cy	$20.00	$124
Extra hand excavation at existing foundation	6	cy	$300.00	$1,867
Compacting surfaces of excavation	56	sf	$1.00	$56
Earthwork support to faces of excavation	132	sf	$3.00	$396
Removal of surplus soil off site	6	cy	$15.00	$93
Cast in Place Concrete				
Foundations	6	cy	$400.00	$2,564
Pump concrete	6	cy	$40.00	$256
Reinforcement				
Bar reinforcement to foundations	500	lbs	$2.00	$1,000
Inserting #5 bar into pre-drilled hole with epoxy	12	ea	$45.00	$540
Labors on Concrete				
Drilling holes in existing concrete for #5 reinforcement	12	ea	$12.00	$144
Forming key to existing concrete	36	sf	$8.50	$306
Piles				
14" Sq x 65'-0" long precast prestressed pile	130	lf	$250.00	$32,500
Dispose of spoil	6	cy	$55.00	$309
Allowance for obstructions encountered	1	ls	$10,000.00	$10,000
Mobilization	1	ls	$15,000.00	$15,000
Total - Foundations				***$65,155***
2 Vertical Structure				
Bracing				
100 KIP BRB, 700KIPS/In Effective Stiffness, approx length 27 ft	2	ea	$22,000.00	44000
Plates & angles for BRB	4	ea	$1,500.00	$6,000
Allowance for fireproofing to BRB	2	ea	$150.00	$300
Allowance for additional columns to support BRB	1	ls	$9,000.00	$9,000
Total - Vertical Structure				***$59,300***
3 Floor & Roof Structures				
Roof Structure				
# 1 Detail 2				
Ref HDU5 w /long screws	28	ea	$36.63	$1,026
Ref A35 clips	2,466	ea	$21.30	$52,526
1/2" x 4" dia plate	28	ea	$36.20	$1,014
5/8" dia threaded rod	70	lf	$23.00	$1,610
Core existing concrete wall for connections, allowance	28	ea	$12.00	$336
Connections, allowance	28	ea	$125.00	$3,500

DETAIL ELEMENTS - FIELD OPERATIONS

Element	Quantity	Unit	Unit Cost	Total
#2				
Ref MST48 each side at existing glulam splice	6	ea	$16.70	$100
# 3 Detail 3				
Ref HDU5 w /long screws	48	ea	$36.63	$1,758
Ref A35 clips	2,978	ea	$21.30	$63,431
1/2" x 4" dia plate	48	ea	$36.20	$1,738
5/8" dia threaded rod	144	lf	$23.00	$3,312
Core existing concrete wall for connections, allowance	48	ea	$12.00	$576
Connections, allowance	48	ea	$125.00	$6,000
4x6 blocking	144	lf	$23.38	$3,366
# 4 Detail 4				
Ref HDU5 w /long screws	34	ea	$36.63	$1,245
Ref A35 clips	2,110	ea	$21.30	$44,943
1/2" x 4" dia plate	34	ea	$36.20	$1,231
5/8" dia threaded rod	204	lf	$23.00	$4,692
Core existing concrete wall for connections, allowance	34	ea	$12.00	$408
Connections, allowance	34	ea	$125.00	$4,250
4x6 blocking	204	lf	$23.38	$4,769
#5				
Ref HDU5	16	ea	$36.63	$586
1/2 thru rod attached to 4x8x6-0 long on each side of existing ridge GLB, sistered to existing joist	16	ea	$329.25	$5,268
Connections, allowance	16	ea	$125.00	$2,000
# 6 Detail 7				
Glulam girders, 5 1/2 x 9	124	lf	$129.00	$15,996
3x14 blocking @ 6'-0 O.C	250	ea	$21.00	$5,250
Ref A35 clips	62	ea	$20.10	$1,246
1 GA clip @ 24 O.C	62	ea	$10.75	$667
#7				
1" dia thru rod each side of new GLB drag, weld rod to 1/4 plate each side of new GLB, (6) 3/4 thrubolt plate to GLB	8	ea	$520.40	$4,163
#8 Detail 5				
5" dia standard pipe	0.16	tn	$7,500.00	$1,231.88
Gusset & plate with (4) 1' dia thru bolts, connection through existing concrete wall	1	ea	$221.20	$221.20
Connections, allowance	1	ea	$125.00	$125.00
#9				
5/8" x 8" plate	1	ea	$40.00	$40.00
(2) 7/8 dia thrubolts to each GLB	8	ea	$28.00	$28.00
Core existing GLB for connections, allowance	1	ea	$12.00	$12
Connections, allowance	1	ea	$125.00	$125.00
#10				

DETAIL ELEMENTS - FIELD OPERATIONS

Element	Quantity	Unit	Unit Cost	Total
Hold down attachment and thrubolt, similar to detail #2				
Ref HDU5 w /long screws	1	ea	$36.63	$37
Ref A35 clips	34	ea	$21.30	$714
1/2" x 4" dia plate	1	ea	$36.20	$36
5/8" dia threaded rod	3	lf	$23.00	$58
Core existing concrete wall for connections, allowance	1	ea	$12.00	$12
Connections, allowance	1	ea	$125.00	$125
#11 Detail 5				
5" dia standard pipe	0.66	tn	$7,500.00	4927.5
Gusset & plate with (4) 1' dia thru bolts, connection through existing concrete wall	8	ea	$221.20	$1,769.60
Connection, allowance	8	ea	$125.00	$1,000.00
#12				
5 x 5 x 3/8 galvanized angle fixed to concrete walls, 2'-0" long	2	ea	$120.00	$240
3/4" dia ss expansion anchor (3) per leg	6	ea	$57.00	$342
Total - Floor & Roof Structures				***$248,049***
4 Exterior Cladding				
Not applicable				
Total - Exterior Cladding				
5 Roofing and Waterproofing				
Not applicable				
Total - Roofing and Waterproofing				
6 Interior Partitions, Doors and Glazing				
Plywood to Walls				
Remove existing gypsum board to walls	55	lf	$25.00	$1,375
Plywood sheathing 1/2" to walls with screws 10d nails 3" O.C to either side of existing wall	96	sf	$10.34	$993
Plywood sheathing 1/2" to inside face of wall with screws 10d nails @ 4" EN	564	sf	$10.34	$5,832
Premium for sill bolts at 16" O.C and replacement of existing hold downs	47	lf	28	$1,306
5/8" gypsum board, fixed to plywood and painted	660	sf	$9.00	$5,940
Mansard Roof Plan Detail 1				
Plywood sheathing 1/2" 8'-0" long every 3rd truss, with screws 10d @ 6' O.C	448	sf	$15.51	$6,948
3x4 blocking	80	lf	$21.38	$1,710
Simpson A35 w/#6 spax screws into existing ply @ 24" O.C	42	ea	$15.46	$649

DETAIL ELEMENTS - FIELD OPERATIONS

Element	Quantity	Unit	Unit Cost	Total
Total - Interior Partitions, Doors and Glazing				***$24,753***
7 Floor, Wall and Ceiling Finishes				
Strengthen Mezzanine				
Strengthen mezzanine Add (2) 1" North -South Direction cut through existing shelving as required for rod installation and Add (3) 1" rods in East-West Direction cut back existing shelving as required	700	sf	$50.00	$35,000
Lay-in ceiling tile including framing fixed to plywood	700	sf	$7.68	$5,378
Miscellaneous				
Repairs to finishes to tilt up building	10,798	sf	$5.00	$53,990
Repairs to finishes to timber building	7,267	sf	$2.00	$14,534
Total - Floor, Wall and Ceiling Finishes				***$108,902***
8 Function Equipment and Specialties				
Not applicable				
Total - Function Equipment and Specialties				
9 Stairs and Vertical Transportation				
Not applicable				
Total - Stairs and Vertical Transportation				
10 Plumbing Systems				
Allowance for relocation of services as necessary to tilt up building	10,798	sf	$2.50	$26,995.00
Allowance for relocation of services as necessary to timber building	7,267	sf	$0.50	$3,633.50
Total - Plumbing Systems				***$30,629***
11 Heating, Ventilation and Air Conditioning				
Allowance for relocation of services as necessary to tilt up building	10,798	sf	$5.00	$53,990.00
Allowance for relocation of services as necessary to timber building	7,267	sf	$2.50	$18,167.50
Total - Heating, Ventilation and Air Conditioning				***$72,158***
12 Electrical Lighting, Power and Communications				
Allowance for relocation of services as necessary to tilt up building	10,798	sf	$7.50	$80,985.00

DETAIL ELEMENTS - FIELD OPERATIONS

Element	Quantity	Unit	Unit Cost	Total
Allowance for relocation of services as necessary to timber building	7,267	sf	$5.00	$36,335.00
Total - Electrical Lighting, Power and Communications				***$117,320***
13 Fire Protection Systems				
Allowance for relocation of services as necessary to tilt up building	10,798	sf	$3.00	$32,394.00
Allowance for relocation of services as necessary to timber building	7,267	sf	$0.25	$1,816.75
Total - Fire Protection Systems				***$34,211***
14 Site Preparation and Demolition				
Miscellaneous				
Provide protection to the existing building and finishes, remove protective lining on completion of works to tilt up building	10,798	sf	$3.00	$32,394
Provide protection to the existing building and finishes, remove protective lining on completion of works to timber building	7,267	sf	$1.20	$8,720
Temporary barriers and protection to tilt up building	10,798	sf	$2.50	$26,995
Temporary barriers and protection to timber building	7,267	sf	$1.00	$7,267
Allowance for scaffolding to tilt up building	10,798	sf	$5.00	$53,990
Allowance for scaffolding to timber building	7,267	sf	$2.00	$14,534
Allowance working within an occupied structure - 15% of labor hours	704	hrs	$115.00	$81,000
Total - Site Preparation and Demolition				***$224,900***
15 Site Paving, Structures & Landscaping				
Not applicable				
Total - Site Paving, Structures & Landscaping				
16 Utilities on Site				
Not applicable				
Total - Utilities on Site				

ADMINISTRATION BUILDING

PROJECT SUMMARY - ADMINISTRATION BUILDING

Element		Subtotal	Total	Cost / SF	Cost / SF
A) Shell (1-5)			**$1,590,855**		**$56.16**
1 Foundations		$96,896		$3.42	
2 Vertical Structure		$1,402,627		$49.51	
3 Floor & Roof Structures		$91,331		$3.22	
4 Exterior Cladding					
5 Roofing and Waterproofing					
B) Interiors (6-7)			**$141,640**		**$5.00**
6 Interior Partitions, Doors and Glazing					
7 Floor, Wall and Ceiling Finishes		$141,640		$5.00	
C) Equipment and Vertical Transportation (8-9)					
8 Function Equipment and Specialties					
9 Stairs and Vertical Transportation					
D) Mechanical and Electrical (10-13)			**$509,904**		**$18.00**
10 Plumbing Systems		$70,820		$2.50	
11 Heating, Ventilation and Air Conditioning		$141,640		$5.00	
12 Electrical Lighting, Power and Communications		$212,460		$7.50	
13 Fire Protection Systems		$84,984		$3.00	
E) Site Construction (14-16)			**$716,296**		**$25.29**
14 Site Preparation and Demolition		$716,296		$25.29	
15 Site Paving, Structures & Landscaping					
16 Utilities on Site					
Subtotal			**$2,958,695**		$104.44
General Conditions	9.0%		**$266,283**		$9.40
Subtotal			**$3,224,977**		$113.84
Bonds & Insurance	2.0%		**$64,500**		$2.28
Subtotal			**$3,289,477**		$116.12
General Contractor Fee	5.0%		**$164,474**		$5.81
Subtotal			**$3,453,951**		$121.93
Design Contingency	15.0%		**$518,093**		$18.29
Subtotal			**$3,972,043**		$140.22
Escalation to MOC, 07/02/16	7.33%		**$291,332**		$10.28
Subtotal			**$4,263,375**		$150.50
Construction Contingency	10.00%		**$426,338**		$15.05
TOTAL ESTIMATED CONSTRUCTION COST			**$4,689,713**		**$165.55**

Total Area: **28,328 SF**

DETAIL ELEMENTS - ADMINISTRATION BUILDING

Element	Quantity	Unit	Unit Cost	Total
1 Foundations				
Detail 2 Concrete Curb				
Formwork	32	sf	$25.00	$788
Concrete, 4,000 PSI	1	cy	$400.00	$458
Pump concrete	1	cy	$40.00	$46
Reinforcement (4) #5	125	lbs	$2.00	$250
Adhesive Dowels #5 @ 6" O.C	60	ea	$57.00	$3,420
Adhesive Dowel #8 @ 12" O.C	30	ea	$57.00	$1,710
Roughen (E) concrete surface	60	sf	$5.14	$308
Excavation & Disposal				
Excavate pile caps & tie beam	18	cy	$20.00	$364
Extra hand excavation at existing foundation	18	cy	$300.00	$5,456
Compacting surfaces of excavation	125	sf	$1.00	$125
Earthwork support to faces of excavation	452	sf	$3.00	$1,356
Removal of surplus soil off site	18	cy	$15.00	$273
Cast in Place Concrete				
Foundations	19	cy	$275.00	$5,151
Pump concrete	19	cy	$40.00	$749
Reinforcement				
Bar reinforcement to foundations	500	lbs	$2.00	$1,000
Inserting dowels into pre-drilled hole with epoxy (assume #6 per tie beam)	18	ea	$45.00	$810
Labors on Concrete				
Drilling holes in existing concrete for dowels (assume #6)	18	ea	$12.00	$216
Forming key to existing concrete	24	sf	$8.50	$204
Piles				
14" Sq x 65'-0" long precast prestressed pile	195	lf	$250.00	$48,750
Dispose of spoil	8	cy	$55.00	$463
Allowance for obstructions encountered	1	ls	$10,000.00	$10,000
Mobilization	1	ls	$15,000.00	$15,000
Total - Foundations				***$96,896***
2 Vertical Structure				
Bracing				
Remove partitions and finishes to gain access to bracing	16,200	sf	$2.00	$32,400
180 KIP BRB, Effective Stiffness 1800 KIPS/IN, approx 15 ft long	6	ea	$13,000.00	$78,000
220 KIP BRB, Effective Stiffness 2300 KIPS/IN, approx 16 ft long	4	ea	$16,500.00	$66,000
120 KIP BRB, 700KIPS/IN Effective Stiffness, approx 27 ft long	1	ea	$25,000.00	$25,000
400 KIP BRB, 3000KIPS/IN Effective Stiffness, approx 27 ft long	3	ea	$25,000.00	$75,000
120 KIP BRB, 1100KIPS/IN Effective Stiffness, approx 23 ft long	2	ea	$20,000.00	$40,000

DETAIL ELEMENTS - ADMINISTRATION BUILDING

Element	Quantity	Unit	Unit Cost	Total
120 KIP BRB, Effective Stiffeners 1300/1100 KIPS/IN, approx 16 ft long	18	ea	$12,500.00	$225,000
Plates & angles for BRB	52	ea	$1,500.00	$78,000
Allowance for fireproofing to BRB	34	ea	$150.00	$5,100
Allowance for additional columns to support external BRB	1	ls	$37,000.00	$37,000
Flip brace orientation	14	ea	$1,840.00	$25,760
Weld (2) L3x3x3/8 angle to underside of beam flange ea side	80	lf	$80.00	$6,400
Supplement brace to gusset weld at each ends	6	ea	$109.00	$654
Weld angle to underside of BM flange ea side	80	lf	$104.00	$8,320
Strengthen column with (2) 3/4 x 6 side plate continuous between foundation and roof level	1.25	t	$6,000.00	$7,528
75 PLF strongback at each BRB buttress. Attach to wall with L4x4x1/4 angles	3	ea.	$1,500.00	$4,500
Expansion angles at 18" OC, including drilling existing concrete wall	74	ea	$87.00	$6,438
Weld 1/2 x 8" plate to underside of beam bottom flange center on transfer column, #5 not shown on drawing -TBD, assume 20 ft long	20	lf	$80.00	$1,600
Weld plate to underside of beam line 8, size TBD	38	lf	$150.00	$5,700
Strengthen connection line 8	1	ea	$300.00	$300
HSS 8x4x3-0" long	0.24	tn	$7,500.00	$1,777
Drag connection with 3/4 plates and 1 1/4 dia thrubolts 5'-0" min extension into wall	12	ea	$303.60	$3,643
12"x8" 1 side of wall with (4) 1" thrubolts extend min 3"-0 into wall, weld plate to web of existing W16 beam	15	lf	$137.50	$2,063
Drill hole thru wall for 2"-0 rod. Attach rod to (E) W18 beam on each side	2	ea	$92.00	$184
Replace existing corroded connections	1	ls	$50,000.00	$50,000
25 lb/ft infill beam	1	tn	$7,500.00	$7,594
30 lb/ft STL Ledger	0.11	tn	$7,500.00	$844
35 lb/ft infill beam	3	tn	$7,500.00	$21,853
5" dia standard pipe	3.19	tn	$7,500.00	$23,898
W18x 35	1.10	tn	$7,500.00	$8,269
Replace stud partitions and paint following installation of bracing	16,200	sf	$33.48	$542,295
Premium bolt through stud channel to concrete floor	600	lf	$19.18	$11,508
Total - Vertical Structure				***$1,402,627***

3 Floor & Roof Structures

Roof Structure

Element	Quantity	Unit	Unit Cost	Total
18 GA x 8" long framing clip with (8) 10d nails into bottom truss chord and 12(d) nails into plywood at each truss bottom chord	214	ea	$295.00	$63,130
Nailer Strengthening 16 GA x 8" long angle (6)10 d nails and (2)#14 self- tapping screws per detail 3 @ 12" OC	80	ea	$197.00	$15,760
Nailer Strengthening 16 GA x 8" long angle (6)10 d nails and 2()#14 self- tapping screws per detail 3 @ 36" OC	9	ea	$197.00	$1,707
Add columns to support ledger, assume one column 35lb/ft	0.71	lf	$7,500.00	$5,316
Remove existing ledger that is connected to HSS Brace	1	ls	$1,631.00	$1,631
6" dia standard pipe	0.20	tn	$7,500.00	$1,496
Thru bolted plate connection (4) 1" dia thru bolts	2	ea	$242.00	$484

DETAIL ELEMENTS - ADMINISTRATION BUILDING

Element	Quantity	Unit	Unit Cost	Total
HSS 8x4x3-0" long	0.08	tn	$7,500.00	$592
Drag connection with 3/4 plates and 1 1/4 dia thrubolts 5'-0" min extension into wall	4	ea	$303.60	$1,214
Total - Floor & Roof Structures				***$91,331***
4 Exterior Cladding				
Not applicable				
Total - Exterior Cladding				
5 Roofing and Waterproofing				
Not applicable				
Total - Roofing and Waterproofing				
6 Interior Partitions, Doors and Glazing				
Not applicable				
Total - Interior Partitions, Doors and Glazing				
7 Floor, Wall and Ceiling Finishes				
Miscellaneous				
Repairs to finishes following works	28,328	sf	$5.00	$141,640
Total - Floor, Wall and Ceiling Finishes				***$141,640***

DETAIL ELEMENTS - ADMINISTRATION BUILDING

Element	Quantity	Unit	Unit Cost	Total
8 Function Equipment and Specialties				
Not applicable				
Total - Function Equipment and Specialties				
9 Stairs and Vertical Transportation				
Not applicable				
Total - Stairs and Vertical Transportation				
10 Plumbing Systems				
Allowance for relocation of services as necessary	28,328	sf	$2.50	$70,820
Total - Plumbing Systems				***$70,820***
11 Heating, Ventilation and Air Conditioning				
Allowance for relocation of services as necessary	28,328	sf	$5.00	$141,640
Total - Heating, Ventilation and Air Conditioning				***$141,640***
12 Electrical Lighting, Power and Communications				
Allowance for relocation of services as necessary	28,328	sf	$7.50	$212,460
Total - Electrical Lighting, Power and Communications				***$212,460***
13 Fire Protection Systems				
Allowance for relocation of services as necessary	28,328	sf	$3.00	$84,984
Total - Fire Protection Systems				***$84,984***
14 Site Preparation and Demolition				
Miscellaneous				
Shoring/propping	1	ls	$50,000.00	$50,000
Allowance for scaffolding	28,328	sf	$4.00	$113,312
Protect work in-place and remove protective lining on completion	28,328	sf	$3.00	$84,984
Allowance working within an occupied structure - 30% of labor hours	4,070	hrs	$115.00	$468,000

DETAIL ELEMENTS - ADMINISTRATION BUILDING

Element	Quantity	Unit	Unit Cost	Total
Total - Site Preparation and Demolition				***$716,296***
15 Site Paving, Structures & Landscaping				
Not applicable				
Total - Site Paving, Structures & Landscaping				
16 Utilities on Site				
Not applicable				
Total - Utilities on Site				

UNION SANITARY DISTRICT

DATE: May 16, 2016

MEMO TO: Board of Directors - Union Sanitary District

FROM: Paul R. Eldredge, General Manager/District Engineer
Sami E. Ghossain, Manager of Technical Services
Raymond Chau, CIP Coach
Thomas Lam, Associate Engineer

SUBJECT: Agenda Item No. 11 – Meeting of May 23, 2016
Authorize the General Manager to Execute Amendment No. 2 to Task Order No. 2 with West Yost Associates for the Plant Facilities Improvements Project

Recommendation

Staff recommends the Board authorize the General Manager to Execute Amendment No. 2 to Task Order No. 2 with West Yost Associates in the amount of $63,257 for providing additional design services for the Plant Facilities Improvements Project.

Background

On November 13, 2014, staff executed an agreement and Task Order No. 1 with West Yost Associates in the amount of $55,567 to provide predesign services for the Plant Facilities Improvements Project. During the predesign phase, West Yost evaluated the various improvements to be included in the project and provided recommendations for design approach and constructability that served as the design basis during the next phase.

On April 27, 2015, the Board authorized the General Manager to execute Task Order No. 2 with West Yost in the amount of $318,074 for providing design services for the project. The task order includes design services for two separate construction projects. Staff combined related project elements from the Plant Facilities Improvements Project and created the new Sodium Hypochlorite Tanks and Piping Replacement Project. The improvements were similar in scope and the District may benefit in more competitive bids from contractors who specialize in this type of work.

On September 30, 2015, staff executed Amendment No. 1 to Task Order No. 2 in the amount of $37,445 to include installation of a new emergency potable water storage tank at the Plant, modification of the metering pumps control in the Odor Control Building and replacement of chemical piping at the Newark Pump Station. The new project scope was included in the Sodium Hypochlorite Tanks and Piping Replacement Project. Staff also renamed this project the Chemical Tanks and Piping Replacement Project to better reflect the overall project scope.

Amendment No. 2 to Task Order No. 2

As West Yost proceeded with the design, staff identified additional issues requiring services that were not included in Task Order No. 2. The additional project scope for each project is summarized below:

Plant Facilities Improvements Project

1. Develop special relining procedures for handling portions of the 60-inch primary effluent pipeline where no existing lining remains. The pipeline condition assessment report prepared by another consultant was the basis for West Yost's design services scope and recommended only the application of a new lining material and not the repair of the steel pipeline where the lining has completed failed.
2. Include additional portion of the 60-inch primary effluent pipeline located at the Primary Clarifiers 5 and 6 to the relining scope. This portion of the pipeline was not included in the initial condition assessment. However, due to the presence of sulfides and location of the pipeline, staff believes the lining in this pipeline will also need to be repaired.
3. Add a new flowmeter to the new emergency water storage tank and make modifications to the existing plant water piping system to allow the connection to the tank. The flowmeter will allow staff to monitor the usage of potable water in the office buildings and the piping modifications will provide separation of the potable water supply to the plant process buildings.

Chemical Tanks and Piping Replacement Project

1. Develop the sodium hypochlorite final effluent dosing strategies and verify the control capabilities of the new chemical metering pumps located at the Odor Control Building (OCB) and the Maintenance Shop Building (MSB). The review of the strategies resulted in the new project scope as outlined in the next two items below.
2. Provide independent connections of the sodium hypochlorite piping from the OCB and MSB located at the final effluent mixer. This will provide staff the ability to quickly switch between the two sources of sodium hypochlorite for final effluent dosing in the event the main source from MSB is unavailable.

Design a new programmable logic controller and cabinet at the MSB and develop new hypo dosing control strategies and SCADA programming. This will improve the communication between the OCB and MSB chemical metering pumps controls in order to maintain reliable final effluent dosing operation.

3. Modify the design of the floor pipe trenches and discharge piping rack to be constructed in the OCB to provide additional space required for future sodium hypochlorite pipe repairs and replacement.
4. At the OCB, modify the vent piping alignment for OCB Chemical Metering Pump No. 2 to drain any chemical that escapes through the vent pipe back to the tank, install a new moisture probe for leak detection, and modify the chemical tank concrete bases to provide level surfaces for the replacement tanks.

The following table summarizes the design fees of the two projects under Task Order No. 2 and the two amendments:

West Yost Contract	Total Fee	Plant Facilities Improvements Project Breakdown	Chemical Tanks and Piping Replacement Project Breakdown
Task Order No. 2 – Design Services	$318,074	$158,294	$159,780
Amendment No. 1 to Task Order No. 2	$37,445	$0	$37,445
Amendment No. 2 to Task Order No. 2	$63,257	$3,771	$59,486
Task Order No. 2 Total	**$418,776**	**$162,065**	**$256,711**
Construction Amount/Estimate	**$4,935,346**	**$1,570,346**	**$3,365,000**
Design Fee as % of Construction Amount/Estimate	**8.5%**	**10.3%**	**7.6%**

Under Amendment No. 2, the additional design fee of $3,771 for the Plant Facilities Improvements Project will increase the design fee percentage only slightly from 10.1% to 10.3% of the awarded construction amount of $1,570,346.

For the Chemical Tanks and Piping Replacement Project, the additional scope increased the construction estimate from $3 million to $3.365 million. With the amended design fee of $59,486, this results in an increase of the design fee percentage of the construction estimate from 6.5% to 7.6%. For a project of this scope and size, staff expects the design fee to be 8% to 10%.

Below is a summary of the task orders and amendments with West Yost under the project agreement:

Task Order / Amendment	Not to Exceed Amount
Task Order No. 1 (Predesign)	$55,567
Task Order No. 2 (Design)	$318,074
Amendment No. 1 to Task Order No. 2	$37,445
Amendment No. 2 to Task Order No. 2	$63,257
Total	**$474,343**

The Plant Facilities Improvements Project is currently under construction and is expected to be completed by January 2017.

The Chemical Tanks and Piping Replacement Project is currently in design and staff expects construction to start in fall 2016.

Staff recommends the Board authorize the General Manger to execute Amendment No. 2 to Task Order No. 2 with West Yost Associates in the amount of $63,257 for providing additional design services for the Plant Facilities Improvements Project.

PRE/SG/RC/TL:ks

Attachments: Figure 1 – Site Plan
Amendment 2 to Task Order No. 2

Figure 1 - Site Map

PLANT FACILITIES IMPROVEMENTS PROJECT

(USD Project No. 800-448)

AMENDMENT NO. 2 TO TASK ORDER NO. 2
TO

AGREEMENT DATED NOVEMBER 13, 2014
BETWEEN UNION SANITARY DISTRICT AND
WEST YOST ASSOCIATES FOR
PROFESSIONAL SERVICES

1. PURPOSE

The purpose of Amendment No. 2 to Task Order No. 2 is to authorize the Engineer to perform design services for additional project scope that were not included in the original Task Order. The additional project scope is summarized below:

Plant Facilities Improvements Project

1. Develop special relining procedures for handling portions of the 60-inch primary effluent pipeline where no existing lining remains.
2. Add additional portion of the 60-inch primary effluent pipeline located at the Primary Clarifiers 5 and 6 to the relining scope.
3. Add a new flowmeter to the new emergency water storage tank and make modifications to the existing plant water piping system to allow the connection to the tank.

Sodium Hypochlorite Tanks and Piping Replacement Project

1. Develop the sodium hypochlorite final effluent dosing strategies and verify the controls capabilities of the new chemical metering pumps located at the Odor Control Building (OCB) and the Maintenance Shop Building (MSB).
2. Provide independent connections of the sodium hypochlorite piping from the OCB and MSB located at the final effluent mixer.
3. Widen the floor pipe trenches to be constructed in the OCB to provide space required for future sodium hypochlorite pipe repairs and replacement.
4. Modify the pipe spacing to be provided on the chemical metering pumps' discharge piping rack in the OCB to provide space required for future sodium hypochlorite pipe repairs and replacement.
5. Revise the vent piping alignment for OCB Chemical Metering Pump No.2 to allow the recirculation of chemical back to the tank in the event of release of air in the pipe.
6. Design a new programmable logic controller and cabinet at the MSB. This is required to improve the communication between the OCB and MSB chemical metering pumps in order to maintain reliable final effluent dosing operation.

7. Design a new moisture probe to be located in the OCB chemical metering pump room to alert the plant staff of a chemical leak.
8. Develop hypo dosing control strategies and SCADA programming.
9. Modify the existing OCB sodium hypochlorite tank concrete bases to provide level surfaces for the replacement tanks.

The project elements described above will be incorporated into Task Order No. 2 as described in the SCOPE OF SERVICES.

2. SCOPE OF SERVICES

The scope of services described in Task Order No. 2 shall be modified as follows:

Task 1. Project Management
No changes.

Task 2. Supplemental Predesign Engineering
No changes.

Task 3. Contract Document Preparation – Plant Facilities Improvements Project
Engineer shall complete additional design service as summarized in Section 1 of this amendment, and revise, finalize, and sign the documents for use in soliciting competitive construction bids for the project.

Project deliverables shall be as outlined in Task Order No. 2.

Task 4. Contract Document Preparation – Sodium Hypochlorite Tanks and Piping Replacement Project

Subtask 4.1 – 50 Percent Design Submittal
No changes.

Subtask 4.2 – 90 Percent Design Submittal
No changes.

Subtask 4.3-Final Design Submittal
Engineer shall complete additional design services as summarized in Section 1 of this amendment, and revise, finalize, and sign the documents for use in soliciting competitive construction bids for the project.

Project deliverables shall be as outlined in Task Order No. 2.

Task 5. Bid Period Services – Plant Facilities Improvements
No changes.

Task 6. Bid Period Services – Chemical Hypochlorite Tanks and Piping Replacement
No changes.

3. PROJECT COORDINATION

All work related to this task order shall be coordinated through the District's Project Manager, Thomas Lam.

4. PAYMENT TO THE ENGINEER

Compensation shall be on a time and materials cost basis for services provided under Article 2 of this Agreement in accordance with the Billing Rate Schedule (updated annually) contained in Task Order No. 2. The billing rate schedule is generally comparable to a labor multiplier of approximately 3.22.

The estimated costs for the work included in this Amendment No. 2 are presented in Exhibit A. The not-to-exceed amount of Amendment No. 2 shall be $63,257

The following table summarizes all task orders and amendments, if any, including those previously executed under the Agreement, ending with this amendment:

Task Order / Amendment	Not to Exceed Amount	Board Authorization Required? (Yes/No)	District Staff Approval
Task Order No. 1	$55,567	No	Paul Eldredge
Task Order No. 2	$318,074	Yes	Paul Eldredge
Amendment No. 1 to Task Order No. 2	$37,445	No	Sami Ghossain
Amendment No. 2 to Task Order No. 2	$63,257	Yes	Paul Eldredge
Total	$474,343		

5. TIME OF COMPLETION

Engineer shall complete the additional design services and submit the final design submittal within thirty (30) calendar days of approval of Amendment No. 2.

6. KEY PERSONNEL

Key engineering personnel or subconsultants assigned to Amendment No. 2 to Task Order No. 2 are as follows:

Role	Personnel/Subconsultant
Principal-in-Charge	John D. Goodwin
Project Manager/Engineer	Jim Waters
Electrical Engineer	Todd Beecher (Beecher Engineering, Inc.)

Key personnel shall not change except in accordance with Article 8 of the Agreement.

IN WITNESS WHEREOF, the parties hereto have made and executed this Amendment No. 2 to Task Order No. 2 as of ____________________, 2016 and therewith incorporated it as part of the Agreement.

ENGINEER:	DISTRICT:
WEST YOST ASSOCIATES	UNION SANITARY DISTRICT
By: ___________________________________	By: ___________________________________
John D. Goodwin Vice President	Paul R. Eldredge, P.E. General Manager/ District Engineer

EXHIBIT A **USD Facilities Improvements Design - Amendment 2: Added Scope Design**

West Yost Associates **PROJECT: USD Plant Facilities Improvements Project**	P/VP $266	PE/PS/PG II $235	SCADD $133	ADM IV $122	**Labor** Hours	Fee	Sub. **CPS** **DETAILED**	Sub. **BEECH** **DETAILED**	**Costs** Sub. w/ markup 5%	Other Direct	Total Costs
Task 1 Project Management & QA/QC											
1.01					0						
Subtotal, Task 1 (hours)	**0**	**0**	**0**	**0**	**0**						
Subtotal, Task 1 ($)											
Task 2 Supplemental Pre-design Engineering											
2.01					0						
Subtotal, Task 2 (hours)	**0**	**0**	**0**	**0**	**0**						
Subtotal, Task 2 ($)											
Task 3 Contract Documents - Plant Facilities Improvements											
3.01		13	5		18	$ 3,720				$ 51	$ 3,771
Subtotal, Task 3 (hours)	**0**	**13**	**5**	**0**	**18**						
Subtotal, Task 3 ($)		**$ 3,055**	**$ 665**			**$ 3,720**				**$ 51**	**$ 3,771**
Task 4 Contract Documents - Chemical Tanks & Piping Replacement											
4.01 Added scope project elements		122	70	10	202	$ 39,200	$ 1,292	$ 18,000	$ 20,257	$ 29	$ 59,486
Subtotal, Task 4 (hours)	**0**	**122**	**70**	**10**	**202**						
Subtotal, Task 4 ($)		**$ 28,670**	**$ 9,310**	**$ 1,220**		**$ 39,200**	**$ 1,292**	**$ 18,000**	**$ 20,257**	**$ 29**	**$ 59,486**
Task 5 Bid Period Services - Plant Facilities Improvements											
5.01					0						
Subtotal, Task 5 (hours)	**0**	**0**	**0**	**0**	**0**						
Subtotal, Task 5 ($)											
Task 6 Bid Period Services - Tanks & Piping Replacment											
6.01					0						
Subtotal, Task 6 (hours)	**0**	**0**	**0**	**0**	**0**						
Subtotal, Task 6 ($)											
TOTAL (hours)	**0**	**135**	**75**	**10**	**220**						
TOTAL ($)		**$ 31,725**	**$ 9,975**	**$ 1,220**		**$ 42,920**	**$ 1,292**	**$ 18,000**	**$ 20,257**	**$ 80**	**$ 63,257**

UNION SANITARY DISTRICT
CHECK REGISTER
04/30/2016-05/13/2016

Check No.	Date	Invoice No.	Vendor	Description	Invoice Amt	Check Amt
160917	5/5/2016	5784	DW NICHOLSON CORP	HIGH SPEED AERATION BLOWER	$89,879.01	$152,791.32
	5/5/2016	5779		PLANT FACILITIES IMPROVEMENTS	$62,912.31	
161012	5/12/2016	30103957	SYNAGRO WEST LLC	MARCH 2016 BIOSOLIDS DISPOSAL	$61,310.30	$61,310.30
160926	5/5/2016	1E5125210	JM EQUIPMENT CO	3 EA PLANT UTILITY CARTS	$52,261.65	$52,261.65
161019	5/12/2016	533620160422	US BANK CORP PAYMENT SYSTEM	MONTHLY CAL-CARD STMT - APR 2016	$36,129.38	$36,129.38
161020	5/12/2016	33790	VALLEY OIL COMPANY	70 TUBES ASTD GREASE	$287.65	$23,048.25
	5/12/2016	33911		20 TUBES GREASE & 2 DRUMS OIL	$1,520.71	
	5/12/2016	33921		6 DRS FLEETGUARD ES COMPLEAT 50/50 ANTIFREEZE	$7,964.96	
	5/12/2016	33789		10 DRS FLEETGUARD ES COMPLEAT 50/50 ANTIFREEZE	$13,274.93	
160914	5/5/2016	XJR638NR4	DELL MARKETING LP C/O DELL USA	RECONCILE BALANCE FROM PO 1005565	$1,655.31	$22,071.88
	5/5/2016	XJX169NT2		FY16 Q3 DESKTOPS & MISSION CONFERENCE ROOM	$20,416.57	
160918	5/5/2016	902589507	EVOQUA WATER TECHNOLOGIES	45,140 LBS HYDROGEN PEROXIDE	$21,053.30	$21,053.30
160915	5/5/2016	1633404C	DELTA DENTAL SERVICE	APRIL 2016 DENTAL	$18,291.16	$20,711.08
	5/5/2016	1633404A		APRIL 2016 DENTAL	$2,419.92	
160974	5/12/2016	111251	FOLGER GRAPHICS	PRINT AND MAIL 2016 NEWSLETTER	$18,929.38	$18,929.38

Check No.	Date	Invoice No.	Vendor	Description	Invoice Amt	Check Amt
161003	5/12/2016	892820160502	PACIFIC GAS AND ELECTRIC	SERV TO 05/01/16 HAYWARD MARSH	$53.72	$18,073.65
	5/12/2016	224720160422		SERV TO 04/21/16 CS TRAINING TRAILER	$62.90	
	5/12/2016	380420160502		SERV TO 04/28/16 CHERRY ST PS	$190.55	
	5/12/2016	898220160502		SERV TO 05/01/16 FREMONT PS	$224.26	
	5/12/2016	761520160426		SERV TO 04/25/16 NEWARK PS	$17,272.59	
	5/12/2016	096020160502		SERV TO 05/01/16 CATHODIC PROJECT	$50.50	
	5/12/2016	666720160502		SERV TO 05/01/16 PASEO PADRE PS	$219.13	
160886	5/5/2016	65528	3T EQUIPMENT COMPANY INC	2 PIPEPATCH KIT - WINTER	$1,003.20	$17,279.80
	5/5/2016	65529		4 PIPEPATCH KIT - WINTER	$3,862.00	
	5/5/2016	65549		10 PIPEPATCH KIT - WINTER	$5,643.00	
	5/5/2016	65530		12 PIPEPATCH KIT - WINTER	$6,771.60	
160972	5/12/2016	93114028	ESRI INC	ESRI ARCGIS SOFTWARE MAINTENANCE.	$13,045.07	$13,045.07
160963	5/12/2016	16130	BELAIRE ENTERPRISES INC.	FUEL ISLAND REPAIRS	$11,458.00	$11,458.00
161006	5/12/2016	21691	RMC WATER AND ENVIRONMENT	HAYWARD MARSH REHABILITATION OPTIONS	$10,915.00	$10,915.00
160952	5/5/2016	36446	WECO INDUSTRIES LLC	120 GALS SANAFOAM VAPOROOTER II	$8,009.45	$8,009.45
160931	5/5/2016	37432220160501	LINCOLN NATIONAL LIFE INS COMP	LIFE & DISABILITY INSURANCE - MAY 2016	$7,666.78	$7,666.78
161018	5/12/2016	741272	UNIVAR USA INC	4,800 GALS SODIUM HYPOCHLORITE	$2,170.61	$6,702.65
	5/12/2016	741867		5,011 GALS SODIUM HYPOCHLORITE	$2,266.02	
	5/12/2016	741738		5,011 GALS SODIUM HYPOCHLORITE	$2,266.02	

UNION SANITARY DISTRICT
CHECK REGISTER
04/30/2016-05/13/2016

Check No.	Date	Invoice No.	Vendor	Description	Invoice Amt	Check Amt
160948	5/5/2016	740311	UNIVAR USA INC	5,011 GALS SODIUM HYPOCHLORITE	$2,266.02	$6,699.49
	5/5/2016	740507		5,008 GALS SODIUM HYPOCHLORITE	$2,264.66	
	5/5/2016	740246		4,796 GALS SODIUM HYPOCHLORITE	$2,168.81	
160988	5/12/2016	513689	INSTRUMART	4 EA INSTRUMART PH METER FOR APS AND HEADWORKS	$5,658.40	$5,658.40
161014	5/12/2016	130859	TOTAL WASTE SYSTEMS INC	APRIL 2016 GRIT DISPOSAL	$5,622.59	$5,622.59
160905	5/5/2016	28944	CALIFORNIA WATER TECHNOLOGIES	44,520 LBS FERROUS CHLORIDE	$5,129.14	$5,129.14
160967	5/12/2016	28924	CALIFORNIA WATER TECHNOLOGIES	42,440 LBS FERROUS CHLORIDE	$4,806.21	$4,806.21
160939	5/5/2016	21690	RMC WATER AND ENVIRONMENT	ALVARADO TREATMENT PLANT SITE USE STUDY	$4,609.00	$4,609.00
160932	5/5/2016	12643	LOOKINGPOINT INC	WEBEX ANNUAL USER LICENSES	$2,142.00	$4,512.00
	5/5/2016	12634		WEBEX ANNUAL USER LICENSES	$2,370.00	
160997	5/12/2016	58232922	MCMASTER SUPPLY INC	ASTD PARTS & MATERIALS	$989.26	$4,471.02
	5/12/2016	54823244		2 EA STEEL DRUMS	$263.55	
	5/12/2016	59043275		1 STAINLESS STEEL SHOWER STATION WITH EYE WASH	$2,361.48	
	5/12/2016	59079668		8 COUPLINGS	$429.91	
	5/12/2016	58232572		ASTD PARTS & MATERIALS	$426.82	
160936	5/5/2016	1040438	POLYDYNE INC	41,340 LBS CLARIFLOC WE-539	$4,092.66	$4,092.66
160971	5/12/2016	20160193	ENVIRO SAFETECH INC	NPS AND IPS WET WELL CLEANING	$3,958.50	$3,958.50

Check No.	Date	Invoice No.	Vendor	Description	Invoice Amt	Check Amt
160934	5/5/2016	24862729	MOTION INDUSTRIES INC	1 EA SHEAR PIN	$44.83	$3,918.30
	5/5/2016	24862800		EFFLUENT SCREEN CHAIN AND SPROCKETS	$3,750.93	
	5/5/2016	24862803		BELTS & FILTERS	$122.54	
160994	5/12/2016	12656	LOOKINGPOINT INC	SIP MIGRATION - FIRST 50%	$3,703.50	$3,703.50
160968	5/12/2016	7500	COAST TROPICAL	REFUND # 18925	$3,515.00	$3,515.00
160977	5/12/2016	113602771001	GEXPRO	ADDITIONAL IFIX RDP CLIENT LICENSES	$3,422.54	$3,422.54
160976	5/12/2016	8286	GENMOR PLUMBING	REFUND # 18921	$3,300.00	$3,300.00
160950	5/5/2016	20160501	VISION SERVICE PLAN - CA	MAY 2016 VISION STMT	$3,248.64	$3,248.64
160924	5/5/2016	28267017007	HERTZ EQUIPMENT RENTAL	MIX TANK RENTAL 3/16 TO 4/13/16	$2,926.00	$2,926.00
160922	5/5/2016	9065763584	GRAINGER INC	2 EA LOUVER PLATE KITS	$108.54	$2,777.39
	5/5/2016	9066182636		1 EA GRADED DENSITY CARTRIDGE	$21.37	
	5/5/2016	9065298391		2 EA PILOT LIGHTS	$236.18	
	5/5/2016	9065179476		2 EA PILOT LIGHTS	$206.46	
	5/5/2016	9066747859		ASTD PARTS & MATERIALS	$1,983.96	
	5/5/2016	9062917464		2 EA FILTERS	$11.74	
	5/5/2016	9067164658		2 EA REDUCING COUPLINGS	$24.49	
	5/5/2016	9067164666		20 EA BOXES OF BANDAGES	$66.00	
	5/5/2016	9065992126		2 EA FILTERS	$118.65	
160991	5/12/2016	15120096	KNOWLEDGELAKE	KNOWLEDGELAKE IMAGING ANNUAL SUPPORT	$2,767.60	$2,767.60

Check No.	Date	Invoice No.	Vendor	Description	Invoice Amt	Check Amt
160965	5/12/2016	201601	BLAKELY PICTURES INC.	USD TREATMENT PLANT VIDEO	$2,450.00	$2,450.00
160989	5/12/2016	64539	JACK DOHENY SUPPLIES, INC.	REPAIR OIL LEAK TRUCK T3252	$2,353.23	$2,353.23
160945	5/5/2016	3784	SIGNET TESTING LABS INC	PLANT FACILITIES IMPROVEMENTS	$551.76	$2,295.89
	5/5/2016	3775		ALVARADO BLVD SEWER MAIN REPAIR	$1,744.13	
161005	5/12/2016	916002737504	REPUBLIC SERVICES #916	RECYCLE & ROLL OFF - APRIL 2016	$2,216.88	$2,216.88
160969	5/12/2016	264658	CURTIS & TOMPKINS, LTD	18 LAB SAMPLE ANALYSIS	$1,755.00	$2,210.00
	5/12/2016	264586		1 LAB SAMPLE ANALYSIS	$120.00	
	5/12/2016	264565		3 LAB SAMPLE ANALYSIS	$335.00	
161000	5/12/2016	20160430	NAPA AUTO PARTS	MONTHLY AUTO PARTS STMT - APR 2016	$2,152.38	$2,152.38
160920	5/5/2016	235236	FRANK A OLSEN COMPANY	1 SITE WASTE CK VLV AIR CUSHIONS	$1,966.80	$1,966.80
160959	5/12/2016	8174048	ABC IMAGING, INC.	NEWARK BACKYARD SS RELOCATION - PHASE 3	$1,917.61	$1,917.61
160923	5/5/2016	3J3988	HARRINGTON INDUSTRIAL PLASTICS	ASTD PVC PARTS & MATERIALS	$248.64	$1,861.86
	5/5/2016	3J3987		40 FEET PVC PIPE	$45.55	
	5/5/2016	3J3986		ASTD PVC PARTS & MATERIALS	$1,567.67	
161021	5/12/2016	9764112893	VERIZON WIRELESS	WIRELESS SERV 03/21/16-04/20/16	$1,834.19	$1,834.19

Check No.	Date	Invoice No.	Vendor	Description	Invoice Amt	Check Amt
160951	5/5/2016	8044579360	VWR INTERNATIONAL LLC	1 BUFFER PH 4.01	$18.90	$1,822.42
	5/5/2016	8044542482		ASTD PARTS & MATERIALS - CREDIT	$-172.82	
	5/5/2016	8044528533		1 UPGRADE KIT HQ30D/LBOD PROBE	$1,071.36	
	5/5/2016	8044556927		1 BRUCINE-SULFANILIC ACID	$50.48	
	5/5/2016	8044563517		2 PKS TAPE WRITE-ON 1INX40YD WHT	$77.13	
	5/5/2016	8044584418		10 PKS FILTER GLASS FIBR 4.25CM	$777.37	
160912	5/5/2016	264317	CURTIS & TOMPKINS, LTD	24 LAB SAMPLE ANALYSIS	$1,320.00	$1,795.00
	5/5/2016	264256		18 LAB SAMPLE ANALYSIS	$475.00	
160958	5/12/2016	65556	3T EQUIPMENT COMPANY INC	6 LEADERHOSE F X F	$1,702.27	$1,702.27
160941	5/5/2016	1710374003	SAN LEANDRO ELECTRIC SUPPLY	10 CONDUIT REDUCER FEMALE 3/4 X 1/2	$35.97	$1,484.38
	5/5/2016	1710374007		10 CORD GRIP 1/2 NPT CABLE RANGE .310 - .560	$57.99	
	5/5/2016	1710374004		1 CONDULET 1/2IN PVC COATED	$44.35	
	5/5/2016	1712661003		ASTD PARTS & MATERIALS	$79.35	
	5/5/2016	1712661002		ASTD PARTS & MATERIALS	$592.09	
	5/5/2016	1710374008		ASTD PARTS & MATERIALS	$33.83	
	5/5/2016	1710374009		ASTD PARTS & MATERIALS	$88.71	
	5/5/2016	1712661001		ASTD PARTS & MATERIALS	$341.50	
	5/5/2016	1710374005		ASTD PARTS & MATERIALS	$210.59	

Check No.	Date	Invoice No.	Vendor	Description	Invoice Amt	Check Amt
161009	5/12/2016	3301312602	STAPLES CONTRACT & COMMERCIAL	ASTD JANITORIAL SUPPLIES - INVENTORY	$675.45	$1,357.47
	5/12/2016	3301312600		ASTD JANITORIAL SUPPLIES - INVENTORY	$624.80	
	5/12/2016	3301312601		ASTD JANITORIAL SUPPLIES - INVENTORY	$57.22	
160996	5/12/2016	99940	MCINERNEY & DILLON, P.C.	LEGAL SERVICES	$1,332.00	$1,332.00
160938	5/5/2016	294469	RKI INSTRUMENTS INC	EAGLE CALIBRATION SERVICE LEVEL 3	$723.75	$1,177.50
	5/5/2016	294199		10 PROBE, 10", HYDROPHOBIC WITH PARTICLE FILTER	$453.75	
160975	5/12/2016	1083786783	G&K SERVICES CO	UNIFORM LAUNDERING SERVICE	$235.24	$1,135.79
	5/12/2016	1083786784		ASTD DUST MOPS, WET MOPS & TERRY TOWELS	$33.70	
	5/12/2016	1083786782		UNIFORM LAUNDERING & RUGS	$241.48	
	5/12/2016	93805190		11 EA CUSTOMIZED POLO SHIRTS - R. SIMONICH	$625.37	
160962	5/12/2016	3TW49	BAY AREA AIR QUALITY MGMT DIST	ANNUAL PERMIT A1702 RNWL 6/16-6/18	$557.00	$1,114.00
	5/12/2016	3TW55		ANNUAL PERMIT A2737 RNWL 6/16-6/18	$557.00	
160944	5/5/2016	8122768042116	SIERRA SPRING WATER COMPANY	BOTTLESS COOLERS RENTAL	$239.00	$1,109.87
	5/5/2016	4868173042116		WATER SERVICE 03/25/16 - 04/21/16	$870.87	
160984	5/12/2016	3255662838	HILTON LONG BEACH	PREPAY LODGING FOR A. PAREDES	$1,035.14	$1,035.14
160961	5/12/2016	546570	A-PRO PEST CONTROL INC	APR PEST CONTROL	$1,005.00	$1,005.00
161010	5/12/2016	8340	STREAMLINE PLUMBING & DRAIN	REFUND # 18919 & 18920	$1,000.00	$1,000.00
160935	5/5/2016	45532540	OFFICE TEAM	TEMP LABOR-PENALOSA, J., WKEND 04/08/16	$998.00	$998.00
161002	5/12/2016	45593850	OFFICE TEAM	TEMP LABOR-PENALOSA, J., WKEND 04/15/16	$998.00	$998.00

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160921	5/5/2016	1083784822	G&K SERVICES CO	ASTD DUST MOPS, WET MOPS & TERRY TOWELS	$33.70	$934.98
	5/5/2016	1083784821		UNIFORM LAUNDERING SERVICE	$647.00	
	5/5/2016	1083784820		UNIFORM LAUNDERING & RUGS	$254.28	
160981	5/12/2016	1670635	HANSON AGGREGATES INC	5.75 TONS 1/2 MED TYPE A AC-R	$436.36	$933.77
	5/12/2016	1671200		6.57 TONS 1/2 MED TYPE A AC-R	$497.41	
160929	5/5/2016	20160502	CONGNA LI	EXP REIMB: SEPT 2016 WEFTEC CONFERENCE REGIS FEE	$924.00	$924.00
160960	5/12/2016	5137992	ALL INDUSTRIAL ELECTRIC SUPPLY	ASTD PARTS & MATERIALS	$172.05	$895.63
	5/12/2016	5138174		1 APS INTERMEDIATE LEVEL LIGHTS	$236.50	
	5/12/2016	5138175		ASTD PARTS & MATERIALS	$487.08	
160954	5/5/2016	2104135001	WHCI PLUMBING SUPPLY CO	2 EBF-650 BATT LAV FCT	$862.76	$862.76
160901	5/5/2016	90087	BARNETT MEDICAL SERVICES LLC	50 LBS PHARMACEUTICAL WASTE REMOVAL	$85.00	$855.00
	5/5/2016	90242		80 LBS PHARMACEUTICAL WASTE REMOVAL	$85.00	
	5/5/2016	90086		200 LBS PHARMACEUTICAL WASTE REMOVAL	$322.00	
	5/5/2016	89585		210 LBS PHARMACEUTICAL WASTE REMOVAL	$243.00	
	5/5/2016	90537		50 LBS PHARMACEUTICAL WASTE REMOVAL	$85.00	
	5/5/2016	89910		TRIP CHARGE	$35.00	
160928	5/5/2016	20804716	LABOR READY	TEMP LABOR-PERRY R., WK END 04/08/16	$846.45	$846.45
161017	5/12/2016	20160505	KIM TRUONG	EXP REIMB: TYLER CONFERENCE MEALS/LODGING/MILEAGE/SHUTTLE	$801.67	$801.67
160898	5/5/2016	8122	AMERICAN DISCOUNT SECURITY	04/01/16 - 04/15/16 GUARD AT DISTRICT GATE	$759.00	$759.00

Check No.	Date	Invoice No.	Vendor	Description	Invoice Amt	Check Amt
160966	5/12/2016	46373	CALIFORNIA GENERATOR SERVICE	GENERATOR SUPPORT SERVICES	$730.30	$730.30
160973	5/12/2016	163700	EXAMINETICS	HEARING & RESPIRATOR FIT TESTING QUESTIONNAIRE	$725.00	$725.00
161007	5/12/2016	150793	SANDMAN INN	BACKFLOW CERTIFICATION LODGING - CHAPARRO	$723.12	$723.12
160992	5/12/2016	20160509	DEBORAH KULL	EXP REIMB: LODGING, MEALS, AND TRAVEL - EDEN CONF	$688.98	$688.98
160900	5/5/2016	87896581204252016	AT&T	SERV: 03/18/16 - 04/17/16	$677.25	$677.25
160930	5/5/2016	603501	LIBERTY LABS	TRAIN TRACKS ANNUAL SUPPORT RENEWAL 4/1/16 - 3/31/17	$675.00	$675.00
160979	5/12/2016	9068214106	GRAINGER INC	14 PACKS OF DANGER TAGS	$308.00	$657.98
	5/12/2016	9068482513		2 EA FLOWMETERS	$349.98	
160983	5/12/2016	602039434	HILLYARD/SAN FRANCISCO	ASTD JANITORIAL SUPPLIES	$654.90	$654.90
160909	5/5/2016	37327	CLAREMONT BEHAVIORAL SERVICES	MAY 2016 EAP PREMIUM	$634.80	$634.80
160982	5/12/2016	22306	HAYWARD PIPE AND SUPPLY	ASTD PARTS & MATERIALS	$613.35	$613.35
160980	5/12/2016	963319	GRANITE CONSTRUCTION COMPANY	8.23 TONS 1/2"HMA64-10R25	$594.78	$594.78
160953	5/5/2016	20160425	WEF-WATER ENVIRONMENT FEDERATI	WEFTEC 2016 ANNUAL CONFERENCE - COSTELLO	$575.00	$575.00
160888	5/5/2016	20160428.1	ENGINEERING FAIR - 2016 ALAMEDA COU	ACSEF AWARD, SENIOR - FIRST PLACE	$500.00	$500.00
160889	5/5/2016	20160428.4	ENGINEERING FAIR - 2016 ALAMEDA COU	ACSEF AWARD, JUNIOR- FIRST PLACE	$500.00	$500.00
160970	5/12/2016	8357	DRAIN DOCTOR	REFUND # 18924	$500.00	$500.00
160990	5/12/2016	8358	MANNY JAILIE	REFUND # 18923	$500.00	$500.00
160999	5/12/2016	8347	KAREN MORAIDA	REFUND # 18918	$500.00	$500.00
161001	5/12/2016	8297	SARASWATHY NARAYAN	REFUND # 18917	$500.00	$500.00

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160998	5/12/2016	979951	MOBILE MODULAR MANAGEMENT CORP	FMC TRAILER RENTAL - MAY 2016	$493.90	$493.90
160910	5/5/2016	83641	CONCRETE WALL SAWING CO INC	CORE DRILLING	$475.00	$475.00
160904	5/5/2016	20160421	LAURIE BRENNER	TRAVEL REIMB: CASA CONF LODGING/PARKING/SHUTTLE/MEALS	$404.37	$474.59
	5/5/2016	20160503		MILEAGE REIMB: CWEA CONFERENCE	$70.22	
160903	5/5/2016	11231210	BLAISDELL'S	ASTD OFFICE SUPPLIES	$13.17	$465.09
	5/5/2016	11239530		ASTD OFFICE SUPPLIES	$183.77	
	5/5/2016	11241190		ASTD OFFICE SUPPLIES	$148.72	
	5/5/2016	11239040		ASTD OFFICE SUPPLIES	$22.67	
	5/5/2016	11241690		ASTD OFFICE SUPPLIES	$20.66	
	5/5/2016	11237030		ASTD OFFICE SUPPLIES	$37.19	
	5/5/2016	11240150		ASTD OFFICE SUPPLIES	$15.95	
	5/5/2016	11241740		ASTD OFFICE SUPPLIES	$22.96	
160955	5/5/2016	185011	WILDWOOD LODGE, PEWAUKEE, WI	JENBACHER TRAINING LODGING - TATOLA	$422.20	$422.20
161023	5/12/2016	2139032	WHAT'S HAPPENING INC	AD NAME: EARTH DAY 2016	$415.00	$415.00
160947	5/5/2016	180465516	TRENCH PLATE RENTAL COMPANY	28 DAYS TRENCH PLATE & EYEBOLT RENTAL	$350.00	$350.00
161015	5/12/2016	180516316	TRENCH PLATE RENTAL COMPANY	6 SHEETS PLYWOOD 4 FT X 8 FT	$328.50	$328.50
161016	5/12/2016	17471811	TRI DIM FILTER CORPORATION	200 TRI-DEK 15/40 2 PLY PADS	$324.67	$324.67
160985	5/12/2016	944720160427	HOME DEPOT CREDIT SERVICES	MONTHLY HARDWARE STMT - APR 2016	$312.58	$312.58
160890	5/5/2016	20160428.5	ENGINEERING FAIR - 2016 ALAMEDA COU	ACSEF AWARD, JUNIOR- SECOND PLACE	$300.00	$300.00

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Check No.	Date	Invoice No.	Vendor	Description	Invoice Amt	Check Amt
161011	5/12/2016	20160505	SWRCB - CERTIFICATIONS	GRADE III CERT RENEW - BERLING	$300.00	$300.00
160919	5/5/2016	1114007	FASTENAL	ASTD PARTS & MATERIALS	$275.34	$275.34
161013	5/12/2016	28825	THOMAS AND ASSOCIATES	6 GASKET EPDM	$272.09	$272.09
160942	5/5/2016	1710374006	SAN LEANDRO ELECTRIC SUPPLY	4 CONDULET 3/4IN PVC COATED	$256.46	$256.46
160887	5/5/2016	65000	AIR & TOOL ENGINEERING COMPANY	REPAIR APT 190 AIRGO	$136.51	$240.45
	5/5/2016	64939		REPAIR APT AIRGO 190 BREAKER - LABOR	$103.94	
160907	5/5/2016	20160503	RAYMOND CHAU	EXP RIEMB: CIP TEAM QTLY SAFETY STRATEGY	$240.00	$240.00
161008	5/12/2016	20160501	SPOK INC	MAY 2016 PAGER SERVICE	$239.82	$239.82
160899	5/5/2016	20160503	PAMELA ARENDS-KING	TRAVEL REIMB: MILEAGE FOR CMTA CONFERENCE	$228.29	$228.29
160993	5/12/2016	20840939	LABOR READY	TEMP LABOR-PERRY R., WK END 04/15/16	$220.31	$220.31
160986	5/12/2016	5605922	HOSE & FITTINGS ETC	ASTD PARTS & MATERIALS	$173.15	$218.46
	5/12/2016	5605142		ASTD PARTS & MATERIALS	$45.31	
160908	5/5/2016	54544370	CINTAS CORPORATION	2 THERMAL-LINED SWEATSHIRT - J POWELL	$215.93	$215.93
160902	5/5/2016	168540	BAY CENTRAL PRINTING	1000 ORGANIC WASTE MANIFEST FORMS	$214.50	$214.50
160913	5/5/2016	201604.10	DALE HARDWARE INC	04/16 - ASTD PARTS & MATERIALS	$206.57	$206.57
160978	5/12/2016	2775611001	GLACIER ICE COMPANY INC	36 EA 7-LB BAG OF ICE	$50.04	$175.14
	5/12/2016	2775611002		90 EA 7-LB BAG OF ICE	$125.10	
160933	5/5/2016	56653117	MCMASTER SUPPLY INC	ASTD PARTS & MATERIALS	$134.28	$162.71
	5/5/2016	57476150		1 PACK MATERIAL MGMT LABELS	$28.43	

Check No.	Date	Invoice No.	Vendor	Description	Invoice Amt	Check Amt
160911	5/5/2016	20160502	SOL COOPER	EXP REIMB: MILEAGE - BACK FLOW PREVENTION TRAINING	$153.90	$153.90
161004	5/12/2016	8200000009511	RED WING SHOE STORE	SAFETY SHOES - RODRIGUES	$152.74	$152.74
160964	5/12/2016	11241741	BLAISDELL'S	1 WIRELESS MOUSE	$32.99	$151.63
	5/12/2016	11244460		ASTD OFFICE SUPPLIES	$50.68	
	5/12/2016	11253020		1 PARTITION COAT HOOK	$12.64	
	5/12/2016	11253270		ASTD OFFICE SUPPLIES	$32.65	
	5/12/2016	11252370		1 PK LABEL TAPE	$22.67	
160891	5/5/2016	20160428.2	ENGINEERING FAIR - 2016 ALAMEDA COU	ACSEF AWARD, SENIOR - SECOND PLACE	$150.00	$150.00
160892	5/5/2016	20160428.3	ENGINEERING FAIR - 2016 ALAMEDA COU	ACSEF AWARD, SENIOR - SECOND PLACE	$150.00	$150.00
160943	5/5/2016	903253541	SHARP BUSINESS SYSTEMS	MTHLY MAINTENANCE BASED ON USE	$115.88	$115.88
160925	5/5/2016	528184	HULBERT LUMBER SUPPLY	3 BUNDLES OF STAKES	$47.22	$106.08
	5/5/2016	528233		ASTD LUMBER SUPPLIES	$58.86	
160894	5/5/2016	20160428.7	ENGINEERING FAIR - 2016 ALAMEDA COU	ACSEF AWARD, SENIOR - TEACHER AWARD	$100.00	$100.00
160896	5/5/2016	20160428.9	ENGINEERING FAIR - 2016 ALAMEDA COU	ACSEF AWARD, JUNIOR - TEACHER AWARD	$100.00	$100.00
160956	5/5/2016	20160428.6	ENGINEERING FAIR - 2016 ALAMEDA COU	ACSEF AWARD, SENIOR - TEACHER AWARD	$100.00	$100.00
	5/9/2016	20160428.6		ACSEF AWARD, SENIOR - TEACHER AWARD	$100.00	
160957	5/9/2016	20160428.8	ENGINEERING FAIR - 2016 ALAMEDA COU	ACSEF AWARD, JUNIOR - TEACHER AWARD	$100.00	$100.00
	5/5/2016	20160428.8		ACSEF AWARD, JUNIOR - TEACHER AWARD	$100.00	
160937	5/5/2016	80288	REMOTE SATELLITE SYSTEMS INT'L	IRIDIUM SVC FEE MAY 2016	$97.90	$97.90

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160927	5/5/2016	20160428	KATHLEEN KING	EXP REIMB: GIFT CARDS (ADMIN PROFESSIONALS) & QAI SNACKS	$71.36	$71.36
160946	5/5/2016	20160502	THOMAS SOLARI	EXP REIMB: MILEAGE FOR CALL OUT	$65.33	$65.33
160940	5/5/2016	7612657201	RS HUGHES CO INC	1 PR KNEE BOOTS RUBBER WITH SAFETY TOE	$55.58	$55.58
161022	5/12/2016	8044602171	VWR INTERNATIONAL LLC	1 BUFFER PH7 & 2 BUFFER PH 10CC	$55.06	$55.06
160897	5/5/2016	4088644120160422	ALAMEDA COUNTY WATER DISTRICT	SERV TO: 04/22/16 - BOYCE ROAD	$53.04	$53.04
160916	5/5/2016	615320160418	DISH NETWORK	MAY 2016 - SERVICE FEE	$50.89	$50.89
160906	5/5/2016	159374	STATE OF CALIFORNIA	1 NEW HIRE FINGERPRINTS	$49.00	$49.00
160949	5/5/2016	9853156.0	UPS - UNITED PARCEL SERVICE	SHIPPING CHARGES W/E 04/09/16	$45.14	$45.14
160995	5/12/2016	77795237	MATHESON TRI-GAS INC	MONTHLY CYLINDER FEE - DEC 2015	$35.13	$35.13
160987	5/12/2016	528584	HULBERT LUMBER SUPPLY	ASTD LUMBER SUPPLIES	$15.74	$15.74

Invoices:

Credit Memos :	1	-172.82
$0 - $1,000 :	161	45,999.61
$1,000 - $10,000 :	51	156,710.88
$10,000 - $100,000 :	14	447,148.65
Over $100,000 :	0	
Total:	227	649,686.32

Checks:

$0 - $1,000 :	80	31,561.72
$1,000 - $10,000 :	44	138,846.54
$10,000 - $100,000 :	13	326,286.74
Over $100,000 :	1	152,791.32
Total:	138	649,486.32

DATE: May 16, 2016

MEMO TO: Board of Directors - Union Sanitary District

FROM: Paul R. Eldredge, General Manager / District Engineer
Sami E. Ghossain, Manager of Technical Services
Rollie Arbolante, Customer Service Team Coach

SUBJECT: Agenda Item No. 12.b - Meeting of May 23, 2016
Information Item: **Standard Specifications and Information Bulletin Update**

Recommendation

Information Only.

Background

Union Sanitary District's Standard Specifications govern the design and construction requirements of sanitary sewer main and lateral installations by private contractors (non-CIP projects). The Specification was last updated in 2006. A number of revisions to building and plumbing codes, as well as other standards, have occurred since the last revision. Manufacturers of a number of the products specified in the Standard Specifications have changed material references or, through company acquisitions, are available through a different manufacturer. Furthermore, recently adopted procedures and District's experience with certain construction materials and methods are not reflected in the 2006 Standard Specifications. To keep current, the Union Sanitary District Standard Specifications and Information Bulletin requires updating.

A Request for Proposals was issued for the 2016 Standard Specifications and Information Bulletin Update (Project) in March 2016. Three proposals were received from qualified consulting engineers. West Yost Associates was selected based on their extensive experience in District projects and similar work.

A scope of work and cost proposal was submitted by West Yost Associates for the Project. The tasks were reviewed and determined appropriate. The negotiated cost proposal is $65,072.

The scope of work for this project includes day-to-day project administration, the kickoff meeting and review workshop, and technical reviews. West Yost will review the Standard Specifications and Details, review and compare material references and call outs with current standards, and revise the documents as appropriate. West Yost will also review and update the AutoCAD details and create up to 10 additional details. Finally, the Standard Specifications and details will be converted to PDF format and indexed by table of contents and index allowing the user to select the item in the table of contents or index and bring up the applicable page quickly.

Work on the project is expected to start in late May 2016, and be completed by the end of August 2016.

PRE/SEG/RA:ks

Brackish Groundwater Can Augment Fresh Water

April 21, 2016

Development of brackish groundwater in the United States, if carried out responsibly, can augment supplies and relieve growing stress on freshwater resources, according to an issue brief from Rice University's Baker Institute for Public Policy.

"Brackish Groundwater: Current Status and Potential Benefits for Water Management" describes the current state of brackish groundwater use and development in the U.S. Because water is regulated primarily at the state level, the paper considers four examples of states with specific regulations for brackish groundwater resources -- Texas, Florida, Arizona, and New Mexico -- and discusses management objectives and policy recommendations that will encourage the responsible use of this resource.

The brief was co-authored by Regina Buono, the Baker Botts Fellow in Energy and Environmental Regulatory Affairs at the Baker Institute; Katherine Zodrow, postdoctoral research associate in the institute's Center for Energy Studies; Pedro Alvarez, the George R. Brown Professor of Materials Science and NanoEngineering; and Qilin Li, associate professor of civil and environmental engineering and of materials science and nanoengineering.

"Researchers have documented a growing disparity between water supply and demand, which is caused by a rapidly increasing population, economic growth, drought, and rising calls for environmental flows," the authors wrote. "The shortage, if left unaddressed, is likely to lead, ultimately, to crisis or conflict between water users, with the attendant effects on the economy and human well-being. Increased understanding and utilization of unconventional water resources will increase water security and assist economic growth into the future. Facilitating the responsible development of brackish groundwater will help relieve pressure on freshwater resources and mitigate potential water crises in the years to come."

Brackish groundwater has a high concentration of total dissolved solids (TDS), including the common salt sodium chloride. It is often defined as water containing between 1,000 and 10,000 parts per million (ppm) TDS. (Seawater contains about 35,000 ppm TDS, and the secondary standard for drinking water in the U.S. is 500 ppm TDS.) The cost of extracting groundwater is proportional to its depth, and many regions of the U.S. have brackish groundwater within 1,000 feet of the land surface.

In Texas, several oil and gas well operators are turning to brackish groundwater as an alternative source of water. The use of brackish water for hydraulic fracturing operations has increased, especially in the Eagle Ford, Permian, and Anadarko basins, more arid parts of Texas that lack easy access to freshwater.

Due to differences in brackish groundwater sources, recharge rates and connectivity with fresh aquifers, policy development requires a detailed understanding of hydrogeology, and regulation of brackish aquifers may vary depending on the aquifer type, the authors said. Different states have chosen different definitions for brackish or impaired aquifers, resulting in a variety of approaches to regulating the resource. Because brackish groundwater contains a high level of salts, it requires advanced treatment prior to most common uses.

Allowing the current system of groundwater governance to control this resource is a missed opportunity to facilitate the expansion of water supply, to provide an incentive for smarter, targeted water use and to enable freshwater conservation, the report stated.

The authors caution that water resources should be regulated and managed in a way that encourages brackish groundwater development without adversely affecting freshwater resources, creates regulatory certainty, protects potential brackish groundwater resources for the future and respects property rights. “Legislators and agency regulators must be careful to find the proper balance between deregulation that may lead to environmental harm and restrictions that may make the use of brackish groundwater economically unviable,” they wrote.

“Also important are laws that protect both freshwater sources and brackish groundwater sources, which are likely to serve as important water resources now and in the future,” the authors wrote. “Finally, acquiring better knowledge and understanding of hydrogeological resources will allow policymakers to make better decisions about how to manage brackish groundwater resources and protect aquifers, both brackish and fresh.”

The issue brief draws upon a longer article by the authors, “A New Frontier in Texas: Managing and Regulating Brackish Groundwater,” which will be published in the June issue of the journal Water Policy.

The Future of Industrial Water Reuse

By Kevin Westerling   April 25, 2016

A market expert shines a light on the bright spots and trouble spots for industrial water reuse, revealing who should consider the practice and why.

There's no doubt that water reuse is a trend on the rise, or that reclaimed water is integral for the future of water security. But technology rollouts, even of the trendy variety, can unfurl slowly. Municipalities and businesses won't make the leap to water reuse unless the case is compelling, and municipalities must also deal with lack of opportunity (new plant builds being few and far between), while pinning their hopes on a multitude of approvals. By contrast, the industrial sector has much more opportunity to dive right into water reuse — but are they compelled to do so?

I talked to Nate Maguire, Americas Business Unit Director at Xylem, to understand the drivers, opportunities, and trends in industrial water reuse, and what needs to be done to overcome the real and perceived obstacles holding back potential practitioners.

What are the main drivers for water reuse from an industrial perspective?

There seems to be much better awareness. Industrial users of water are really educating themselves on the true cost of water and the implications of that on them and their businesses.

One element that people are starting to look at is the embedded energy costs in water. It's tremendously energy-intensive to not only get water out of the ground, but also to treat it, get it to your facility, and then to properly dispose of it by treating to effluent standards and discharging back into the environment.

There's also an opportunity cost in water. As water starts to become increasingly scarce, relative to your demand, what are all the things you could do with that water? Companies that are in populous areas with limited supply are starting to be impacted by that.

There's also the risk of water supply from a factor-of-input standpoint — looking at water as an ingredient to your process, and what happens when that ingredient is no longer available in the quantities you need.

And perhaps the last piece is social responsibility. All over the news you're starting to see examples of where companies are coming under scrutiny for taking what is perceived as a disproportionate share of a community's water supply. Those are just a few of the embedded risks within water.

As companies start understanding what the risks are, they start to recognize the true value of water and they start to look at investments that would help to mitigate some of those risks much differently than they had in the past.

Historically, from a hard ROI [return on investment] calculation, water reuse doesn't always look attractive. But when you start peeling back that onion and looking at the true cost of water, in many cases water reuse becomes a much, much more attractive option.

How do you convince hesitant, would-be adopters of the value of reuse?

I think the conversation would probably have to start with a simple mapping of their operation and really looking at where the water is used, how much is used, and the quality levels required for the uses of water within and around their facility.

Then, I think you have to look externally as well, and understand what your risks of water supply are. It is crucial to understand and empathize with the community that you're operating in and the concerns they may have with regards to water and to your business, and look at it in a multifaceted way.

In the past, a lot of these decisions were made purely on just hard financial calculations, which didn't contemplate all the other elements of business and business value.

I think you start there. How much do you need to lower your risk of water supply? How much do you need to address these other areas that you've identified in your research or your analysis?

The other important piece is that you want to fight the perception that there's a 'one size fits all' water reuse solution. The reality is that, across any facility, there are a variety of water requirements, many of them at very different water quality levels and volume requirement levels.

There are many different types of treatment and treatment trains, at obviously very different levels of investment. The concept of 'fit for purpose' is a very important element in this discussion. What are your needs? What are you trying to accomplish? Once those questions are answered, the conversation about an investment profile is appropriate.

What industries can benefit the most from tapping into water reuse?

There are a couple that have a lot of potential, to the extent that water requirements are extremely high — food and beverage and the energy sector. Oil and gas, in particular, requires a tremendous amount of water and treatment, including pretreatment and wastewater treatment, etc., in their processes.

If you look at potential, it's actually pretty diverse across a lot of industries. Food and beverage and oil and gas are probably a couple of the heavy hitters, but within the industrial space there are many, many different subsectors and niche markets. Producers need water, whether it's for heating and cooling, as an ingredient in this thing they're making, as a component in processing, or for potable purposes for employees. There's water embedded in practically everything that we buy and consume.

Reuse obviously isn't suitable for everybody; there has to be a certain scale and the economics have to work, but there's a tremendously diverse space out there that is already starting to adopt water reuse — and the potential is much, much higher.

What obstacles could slow the rate of adoption, and how are they surmounted?

First and foremost, regulations and policies need to catch up. A lot of our policies — in the U.S., in particular, but elsewhere as well — were written many years ago and need to be updated in a consistent way that reflects the current water crisis that we're facing, while also contemplating the technologies of today. We've made significant advances in the way we can treat and test water and wastewater.

There are absolutely some new and very efficient regulations that have come out more recently, but if you step back and look at the regulations that are relevant to or that guide water reuse in particular, it's really a patchwork — there are some holes, some inconsistencies across different states, and so on. I think one action item that needs to happen, and is starting to happen slowly, is an update to our policy and our regulation framework to help speed up the adoption and to really clarify what's expected and what needs to be done.

For example, with the NPDES [National Pollutant Discharge Elimination System], which is a division of the [U.S.] EPA, you would want to see additional water and wastewater reuse provisions applied specifically to wastewater reuse for industrial facilities. Having something that crosses all 50 states, creates a regulatory framework, and clarifies definitions would be a very good thing for industrial wastewater reuse.

What are the factors or industry trends that might hasten the growth of water reuse?

One [is] data management, enabled by the proliferation of sensors and new sensor technology. The ability to track and manage data on the effectiveness of your treatment systems, how well they're performing and at what quality levels, and really helping organizations build tracking and measurement into their treatment systems is probably going to be one of the sub-areas or growth pieces of water reuse.

There's also a lot more discussion around decentralized systems, and this concept is playing out in certain small communities around the U.S. and around the world. One thing that we're starting to see happen out here in California — and it's happening elsewhere, too — is industrial users of water seeking out more treated wastewater instead of just direct, basically potable, water supply from the municipalities. There have been examples where a wastewater facility happens to be located pretty closely to an industrial plant and their water requirements are such that they would like to lower their investment to procure water; meanwhile, the wastewater treatment facility is happy to begin conversations with industrial users to help put their effluent to good use. The conversation on water reuse is growing and the industry is getting smarter in how they evaluate water reuse investments. Does it necessarily mean decentralized? I don't know that it does in all cases, but certainly there's some trend there.

WRDA Bill Passes Senate Committee With Key Water Reuse Provisions

April 28, 2016

The Water Resources Development Act of 2016 (WRDA), new legislation that provides critical investment in water infrastructure and includes support for water reuse, was introduced in the Senate on April 26 and passed by the Environment and Public Works (EPW) Committee on April 28. The legislation cites a Water Environment Federation (WEF) and WateReuse Association sponsored economic study that demonstrates the value of robust funding for State Revolving Fund (SRF) programs.

U.S. Senators Jim Inhofe (R-OK), chairman of the EPW Committee, and Barbara Boxer (D-CA), ranking member of the Senate EPW Committee, introduced the legislation, which is the main vehicle for authorizing water projects to be studied, planned and developed by the U.S. Army Corps of Engineers. It is also the legislative vehicle for implementing policy changes with respect to the Corps' water resource projects and programs.

Some of the provisions of the bill that support increased water reuse include:

- A WaterSense program to identify and promote water efficient products, buildings, etc., including reuse and recycling technologies;
- An Innovative Water Technology Grant Program to accelerate innovate technologies, including reuse and recycling, to address water challenges;
- A task force to draft national drought resilience guidelines, including provisions for reuse; and
- Additional assistance for use of innovative technology in Clean and Drinking Water SRFs.

Although WRDA is intended to be biennial legislation, there have been gaps between measures. "We are happy to see such an important bill making progress," said WateReuse Association Executive Director Melissa Meeker. "While the bill isn't perfect, we appreciate the committee's continued support for reuse and water infrastructure during such a crucial period. We look forward to working with Congress and the Administration on the next steps."

The legislation still needs to be voted on by the full Senate and the House of Representatives must pass its own version.

About WateReuse

The WateReuse Association is a nonprofit coalition of utilities, government agencies and industry that advocates for laws, policies and funding to promote water reuse.

EPA Gets Pushback On Effort To Reveal Lead Pipes

By Sara Jerome May 2, 2016

The U.S. EPA is off to a rocky start in its new effort to bring transparency to where lead service lines are in use in the water system.

The agency has called on states and water utilities to put more information about lead pipes online, but some officials are fighting the possibility. The USA Today Network reviewed documents from 49 states and found a range of concerns from officials.

“Drinking water regulators in about a dozen states expressed varying degrees of resistance or concerns about the EPA’s directive encouraging water systems to voluntarily give consumers easy access to what utilities know about homes receiving drinking water through lead service lines, a key indicator of whether a home's tap water could be contaminated and whether utilities are complying with testing regulations,” the report said.

South Dakota’s water regulatory agency was among the dissenting voices.

“We do not have the initial materials inventory from systems readily available and do not intend to spend valuable staff resources sifting through microfilm to find this information,” officials wrote.

Nevertheless, the EPA has supporters in this effort. Yanna Lambrinidou, a drinking water safety watchdog and faculty member at Virginia Tech, weighed in positively, calling the EPA’s request “critically important.”

“She called resistance expressed by some states ‘highly troubling’ and an impediment to the public knowing whether utilities are testing water from the right customers’ taps, meaning those with the lead service lines that are most likely to have lead-contaminated water,” the report said.

The EPA sent a letter to governors and water regulators in February pledging to take a more active role in state water programs.

“The EPA's Office of Water is increasing oversight of state programs to identify and address any deficiencies in current implementation of the Lead and Copper Rule (LCR),” said the letter from Joel Beauvais, deputy assistant administrator.

“EPA staff are meeting with every state drinking water program across the country to ensure that states are taking appropriate actions to address lead action level exceedances, including optimizing corrosion control, providing effective public health communication and outreach to residents on steps to reduce exposures to lead, and removing lead service lines where required by the LCR,” the letter continued.

Lead contamination of drinking water is a major concern since the crisis in Flint, MI, this year. The issue was thrust into the national news when the governor declared a state of emergency over a lead contamination catastrophe that is taking a toll on public health.

One study “found elevated lead blood levels — surpassing 5 micrograms per deciliter — in 4 percent of Flint youngsters,” ThinkProgress reported.

The Mercury News

California residents cut water use 24.3 percent in March

By Paul Rogers, progers@bayareanewsgroup.com

May 4, 2016

Californians cut water use 24.3 percent in March, the largest savings in any month since last September, state officials announced Tuesday.

The water savings came largely because El Niño storms soaked much of the state throughout that month, particularly Northern California, filling reservoirs and prompting homeowners to shut off their lawn sprinklers.

"This is the most welcome news we've had in a long time," said Felicia Marcus, chairwoman of the State Water Resources Control Board, which releases the monthly conservation data for more than 400 cities, water districts and private water companies. Although the drought emergency is largely over now in Northern California, hotter weather is already here, and scientists are forecasting a 71 percent chance of La Niña conditions by November, which could mean dry weather next winter.

Since last June, when the administration of Gov. Jerry Brown first imposed mandatory water conservation targets on urban areas to address the state's historic drought, California's urban residents have reduced water consumption by 23.9 percent overall during the 10-month period, compared with the same months in 2013, the baseline year. Last June, Brown had set a goal of 25 percent.

In March, the Bay Area reduced water use 25 percent compared to March 2013, and the South Coast region -- mostly Los Angeles and San Diego -- cut by 20.7 percent, while the Sacramento region cut by 36.7 percent.

Because of the winter rains, which gave Northern California its wettest winter in five years, the state water board is scheduled to vote May 18 on changes to the conservation rules. The board is widely expected to relax or drop entirely the rules for Northern California, although it may keep in place some targets for Southern California.

The difference is largely due to rainfall. Many cities in the north this winter rain season have so far received about 100 percent of their historic average rainfall. San Jose on Tuesday was at 102 percent, San Francisco 101, Oakland 84, Stockton 124, Sacramento 91 and Redding 118. But the storms largely missed Southern California. Los Angeles on Tuesday, for example, had only received 54 percent, while San Diego was at 74 percent and Palm Springs was at 56 percent.

The snowpack in the northern Sierra was also greater than in the southern Sierra.

As a result, major reservoirs in the north, like Shasta and Oroville are near full, while reservoirs farther south, like Diamond Valley in Riverside County, and Millerton, near Fresno, were 43 and 57 percent full, respectively, on Tuesday.

The Bay Area could get a little rain later this week, with a slight chance of showers Thursday and Friday before warm, dry conditions return next week. "The real trick will be getting people to hold the line in the warmer, drier months," Marcus said. "If you don't love your lawn, you ought to lose it, and if you do love your lawn you ought to put it on a diet."

Paul Rogers covers resources and environmental issues. Contact him at 408-920-5045. Follow him at Twitter.com/PaulRogersSJMN.

Sonoma Index-Tribune

Sonoma’s sewer system under stress

CHRISTIAN KALLEN
INDEX-TRIBUNE STAFF WRITER | May 5, 2016, 6:31PM

The condition of Sonoma’s sewer lines, and their ability to absorb the new residents and visitors that are expected to arrive with the completion of several housing and hotel projects, has a number of locals worried.

Anne Gomez, who has often appeared before the City Council and Planning Commission to complain about the impact of new visitors and residents on the sewer system, even filed a brief in 2014 threatening a class action lawsuit against the Sonoma Valley County Sanitation District, and while the suit didn’t materialize, Gomez’s core argument remains: “The collection system is broken; adding further stress on it can only create more leakage thus further resulting in more pollution which will immediately cause more threat to the health and safety of the citizens of this state and the environment.”

“As a city we are, at present, in a very vulnerable and precarious situation,” wrote Bob Mosher in a recent letter-to-the-editor to the I-T. “Apparently the (Sonoma Valley Sanitation District) has been paying large penalties for our deficient sewer capacity and continuing to pay these avoidable fees rather than fix the problems.”

These and other recent citizen complaints largely stem from a June 2015 “Cease and Desist” order filed by the California Regional Water Quality Control Board, San Francisco, that found significant failures in the District’s collection system, itemized 46 sanitary sewer overflow (SSO) incidents in five years, and levied a whopping $732,300 in penalties.

While neither denying the SSO incidents nor the fines they’ve received, District representatives point out it’s a more complex picture than might first appear. Grant Davis, the general manager of the Sonoma County Water Agency which has operated the Sonoma Valley Sanitation District since 1994, emphasized that nearly all of the overflows were rainfall-related (the others were due to root infiltration and grease blockage), and there were none in the most recent and very wet winter.

“The District’s SSO problem occurs as a result of excess wet weather infiltration and inflow through leaky sewer pipes. The solution is to fix the leaky pipes and/or increase the size of pipes so that larger pipes can carry more peak wet weather flow.”

In fact the District has been engaged in upgrading the collection system for years, moving northward from the Sonoma Valley Reclamation facility on Eighth Street East, up through the City of Sonoma and the Springs toward Glen Ellen, said Kevin Booker, principal engineer for the Sonoma County Water Agency.

“Since 1994, the district has completed approximately 9.7 miles of major capacity and structural improvement projects at a total cost of $14.9 million,” states a series of talking points the District office issued just this week, possibly in response to press and public questions.

The upgrades include new sewer mains of more durable material, of greater diameter – 24-inch PVC pipes instead of 21-inch reinforced concrete – to increase the carrying capacity and reliability of the sewer collection system.

These include the replacement of the sewer main from Fifth Street West to the treatment facility; the District is now planning the replacement of the sewer main from Sixth Street West to Ramon Street. This phase is at the 60 percent design phase, will go along Highway 12, and construction will cost $4.86 million. Future replacement phases will continue toward Agua Caliente, say District officials.

One spur trunk at the Finnish American Hospitality Association (FAHA) on Verano Avenue has just been replaced in the past couple months.

But while the main trunks of the 132-mile collection system are being steadily upgraded, citizen complaint still focuses on new projects that are within the areas already upgraded, specifically the downtown area where the Hotel Sonoma project recently went before the Planning Commission. (Hotel Sonoma is being developed by Kenwood Investments, whose founder Darius Anderson is the principal investor in Sonoma Media Investments, which owns the Index-Tribune.)

David Goodison, the city planning director, recently made a presentation to the Planning Commission and the City Council in the wake of concerns about the West Napa Street “Hotel Sonoma” project. He adamantly stated that Gomez’s and others’ concerns conflate two issues and systems.

“Generally speaking, the issues raised with the collection system regarding new developments don’t have anything to do with peak storm events,” said Goodison. And peak storm events are what prompt the SSOs, the fines, and compliance failures of the district. Rainfall drainage and sewer collection are different systems, but due to the aged infrastructure of the District’s collection system – and the largely untracked and unregulated accumulated miles of sewer laterals that run from nearly every property in the Valley to the sewer main – the system is not impermeable.

During a heavy rainfall, the ground becomes saturated and water seeps into these sewer laterals, as well as the older main sewer lines. When a sewer overflows, its fluid has nowhere to go but out – up through the manhole and out into the street, or nearby neighborhoods. It’s a clear health and safety concern.

In such storm events, rainfall added to the collection system exceeds the capacity, and even large spills can and have occurred – over 65,000 gallons in a single incident at Rancho

Mobile Homes in December 2014, during a storm that produced eight documented SSOs, totaling 145,860 gallons of overflowed sewage. This single storm was a large part of the violations that brought the $723,000 in fines.

But the sewer laterals, while included in the "collection system," are not part of the Sanitation District's direct responsibility. Instead, it's the property owner's responsibility. "A major contributor to overflows are private sewer laterals in need of repair," said District Manager Davis. "Property owners are required to keep sewer laterals (the pipes that run from the home or business to the sewer main) in good condition."

District officials estimate as many of two-thirds of these lateral pipes are 30 years or older, made of material such as clay, concrete or iron that break down over time, and may be experiencing root infiltration, cracking or displacement. This might be why toilets back up during stormy weather: the homeowner's lateral pipe from the house to the sewer main is failing, taking in rain water, and unable to handle normal household usage.

The Sonoma Valley County Sanitation District has already begun working up a Sewer Lateral Ordinance, a project to tackle the most permeable and potentially explosive aspect of the sewer crisis – the accumulated miles of privately-owned and maintained lateral sewer lines on almost every property in the Valley.

The District should issue the Sewer Lateral Ordinance in the next couple months – it's required to do so by July 1, according to the Regional Board's order – outlining when an inspection of a lateral must occur, a so-called "trigger event." These might include a blockage repair performed by a licensed plumber, a major remodel, or even the sale of a property.

An underground survey of a lateral (often done by a video cable) can cost several hundred dollars; a repair or replacement can easily run into the thousands.

Ann DuBay of the Sonoma County Water Agency is working to create the Sewer Lateral Ordinance, and understands that homeowners might balk at this added responsibility. "It's kind of like a roof – you don't like to think about it until there's a leak, but as a homeowner you know you've got to fix it. It's the same thing with a sewer lateral."

Supervisor Susan Gorin, one of the three members of the Sanitation District Board of Directors, agrees. "Everybody needs to recognize they are part of the system, and take the action necessary to be part of the solution," said Gorin.

Infrastructure failure is a fact of life; keeping up with deteriorating pipes is a concern that both municipalities and their residents share.

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Wipes wreak havoc on wastewater plants

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By **Nate Lynch** Day staff writer

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New London, Connecticut

A few years ago, workers at the New London wastewater treatment plant on Trumbull Street *(New London, CT)* discovered a monster lurking in the system.

During an inspection, Director of Public Utilities Joseph Lanzafame said, workers had found a few small holes in a screen that prevents solid objects — anything from paper towels to two-by-fours — from entering the plant.

That triggered a look inside their sludge storage tank, where workers found a "rag ball" the size of a Volkswagen Bug made of congealed grease, cotton and disposable wipes.

"It was mind-boggling to accumulate something that big," Lanzafame said.

New London isn't the only system facing this problem. From Norwich to Stonington to Ledyard, in big wastewater systems and small, wipes being marketed as disposable are causing clogs everywhere: from the pipes transporting sewage to the plants themselves.

For utilities, the only recourse is educating the public until regulations change.

A recent trend

Flushable wipes, a toilet paper substitute made by brands like Cottonelle and Charmin, are a new and popular innovation on the market.

Their status as "flushable" is up for some debate, however.

"From the experiments that we've done, (there's) one wipe on the market that we think might be flushable," said Cynthia Finley, director of regulatory affairs at the National Association of Clean Water Agencies.

Discussions with the companies are ongoing and NACWA said they were not ready to release the name of that product.

Her organization has been keeping an eye on the disposable wipes phenomenon since complaints were first filed in 2008, and recently launched the "Toilets are Not Trashcans" campaign with the aim of educating the public about products that cause damage to the sewer system, including the wipes.

"What we really don't want is a ban on wipes (being) called flushable," said Finley, because research has shown people instead will buy baby wipes — a plastic product that will cause just as much, if not more, damage.

Instead they would like to see the public using "the right product, that is truly safe," she said.
"Flushable" wipes are one of many wipe products that end up in the sewers, Finley said.
Such products are often difficult to distinguish from one another once they get to the end of the system.

A study done in Maine showed that wipes made up about half of the material that hadn't broke down in the sewer system, and flushable wipes made up around a third of that, in addition to feminine hygiene products and baby wipes.

The wipes combine with fats, oils and grease to create solid masses like London's infamous 15-ton "Fatberg," as it was christened by sewer company Thames Water. The mass caused damage to the system that took six weeks to repair.

Norwich, Ledyard face a similar problem

The headworks of a wastewater treatment plant is the first stage of the treatment process.

In Norwich, which sees four million gallons on an average day, incoming sewage is piped in a deep, narrow channel through a small building.

This houses the first few pieces of equipment: a screen that separates and lifts solids (such as wipes) upward out of the stream and into a conveyor, which rotates and transports the solids into a dumpster.

The remaining sewage flows into primary clarification, where remaining solids settle and are removed.

On Monday, in the tiny building housing Norwich's headworks, a few wipes had escaped the screen.

A worker with a rake flicked a wipe off the pipe that transports solids to a digester, where bacteria break down biodegradable material.

Some wipes were lying on the ground; they likely fell as workers were shovelling them out, said Mike LaLima, the wastewater integrity manager of Norwich Public Utilities.

In Ledyard, workers at the smaller Highlands Wastewater Treatment Facility on Town Farm Road occasionally will get a late-night call from the emergency dispatch center notifying them of a "breach of screen" alarm.

The alarm means the screen has shut itself off, likely due to a clog of wipes. Workers have to drive to the plant and manually rake the wipes out of the system.

"In this industry you can't sit back and kick your feet up: something like the wipes (will) come along," Supervisor Steve Maslin said.

The National Association of Clean Water Agencies estimates nationwide that damage inflicted by wipes on wastewater systems is anywhere from $500 million to $1 billion — including replacing broken equipment, extra labor and power costs — much of which gets passed on to ratepayers.

"Monster" help needed

Sewer treatment plants in the area are using machines called "Muffin Monsters" as their first line of defense against wipes.

Muffin Monsters, made by JWC Environmental, a company that produces a wide variety of screening and shredding products for municipal treatment systems, are grinding pumps installed in places where sewage needs to be pumped upward to be sent to a wastewater treatment plant.

In the Norwich Utilities building on South Golden Street, LaLima's calendar is adorned with JWC's wide variety of Monster products, such as the Auger Monster, a machine that grinds, screens and compacts waste solids, and the Honey Monster, a multi-stage machine that pre-treats material from pumped septic systems.

As Kevin Bates, director of marketing at JWC explained, the original Muffin Monster was designed for 1970s sewage, which Bates said is fundamentally different from today's sewage.

"We were designing to take the really tough solids, the rocks and sticks. We were never designing for indestructible pieces of plastic," he said.

The company has since modified Muffin Monsters to grind material into smaller pieces, with less possibility for wipes to recombine and entangle in pipes.

In New London, each of the six pump stations in the city is equipped with a Muffin Monster, but that's no guarantee there won't be clogs.

For a while, clogged pumps were a weekly occurence, and it can be a race against the clock before sewage starts to back up into homes and businesses if the system isn't working.

"The high flow times — spring rains — that tends to be the most frustrating times," Lanzafame said.

Fortunately, sewage has not backed up into homes and businesses and clogs have become a less frequent headache, happening one or two times a month, said Lanzafame.

That's thanks to the implementation of a program that reduces fats, oils and grease in the system by requiring restaurants to install grease traps and meet compliance checks.

In Stonington, a similar program has helped Water Pollution Control Authority Director Douglas Nettleton cut down on the number of clogs, but wipes are still one of the biggest problems he faces.

He likens the attitude toward flushable wipes to the attitude toward plastic and metal consumer products before the broad adoption of recycling.

"You don't throw that aluminum can in the garbage," he said. "That's what I'm trying to get across."

The Volkswagen Bug-ball of wipes took two or three workers several days to remove — piece by piece, five-gallon bucketful by five-gallon bucketful — until the clog was cleared.

There has been no innovation on removing clogs — so far.

The effort to educate the public has been small-scale so far. Norwich Utilities has mentioned its issues with disposable wipes in past versions of a newsletter, and Veolia Water has sent out a flier to New London customers along with their bill to educate them about the problems caused by wipes.

Still, Lanzafame and other supervisors are optimistic that efforts to educate younger children in schools and during tours of the plant will make a difference in communicating the essential truth of the sewer system.

"When you flush it, it doesn't just go away," Lanzafame said. "We certainly see it again."

THE SACRAMENTO BEE

MAY 9, 2016 11:48 AM

California water regulators propose major shift in drought conservation rules

By Phillip Reese and Ryan Sabalow

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In a major shift in California's urban water policy, state regulators Monday issued proposed conservation rules that would lift the mandatory 25 percent statewide water cuts in place since last June.

Instead, urban water agencies across the state would be required to conserve on a sliding scale tailored to their unique water supply conditions. A draft of the new targets released Monday by the State Water Resources Control Board would allow districts to "self-certify" how much water they expect to have in their supply assuming three additional years of drought, and the level of conservation necessary to ensure they do not run out of water.

Districts would be required to reduce water use by an amount equal to their projected shortfall. For example, in a district where three more dry years would leave a district 10 percent short of anticipated supply, the mandatory conservation target would be 10 percent.

The release of the draft rules came on the same day Gov. Jerry Brown issued a new executive order declaring that drought conditions persist and that the state must take permanent action to mitigate the likelihood of more frequent droughts.

In the short term, the order tells the State Water Resources Control Board to adjust water conservation targets through January 2017.

The order also dictates that the water board and Department of Water Resources create new, permanent water use targets across California. Rather than the sweeping regulations in place over the last year, the order says those goals should be tailored to "the unique conditions of each water agency."

Water agencies in the Sacramento area have been asking for more customized regulations that take into account regional water supply, groundwater reserves and climate.

“It’s time for the state to “recalibrate our habits,” and change them “into an abiding ethic,” said Mark Cowin, director of the state Department of Water Resources.

Under the governor’s order, urban water districts will be required to report water use monthly to the state, extending a mandate that has been in place for more than a year. It permanently bans practices deemed wasteful, including hosing off sidewalks or driveways, washing cars with hoses that don’t have a shut-off nozzle, irrigating lawns in a way that causes runoff and irrigating lawns within 48 hours of precipitation.

“Californians stepped up during this drought and saved more water than ever before,” Brown said in a written release. “But now we know that drought is becoming a regular occurrence and water conservation must be a part of our everyday life.”

An El Niño weather pattern delivered more rain this water year than during any other year of the drought, but not as much as state officials had hoped. About three quarters of the state remains in severe, exceptional or extreme drought, according to the National Drought Mitigation Center.

Even so, the state’s two largest reservoirs –Shasta and Oroville – stand well above historic levels for this point in the year, as does Folsom Reservoir, leading some water districts to gripe the conservation targets currently in place are too high.

California water districts have faced mandatory conservation targets since June. From June to February, districts across the state were required to cut water use by 4 to 36 percent, depending on how much water per capita their residents used in 2014. All but a handful of districts in the Sacramento region were mandated to cut water use by 28 percent or more.

The targets were controversial. Soon after they were proposed, Sacramento area water districts began complaining that the one-sized-fits-all rules were onerous, didn’t account for variances in regional climate, and didn’t give enough credits to improvements some districts had made to shore up local water supplies.

In February, the State Water Resources Control Board relaxed the conservation mandates for many inland communities, where hot, dry summers make it harder to keep lawns and trees alive. Many of the water agencies in greater Sacramento saw their targets fall by 3 percentage points.

“We don’t want to cry wolf and we also don’t want to put our heads in the sand,” said water board chair Felicia Marcus.

Napa Valley Register

Facing an ultimatum, St. Helena to upgrade sewer plant

JESSE DUARTE jduarte@sthelenastar.com

May 9, 2016

ST. HELENA — City Hall and police station are in terrible shape, but regulatory pressure has made upgrading the wastewater treatment plant an even higher priority for St. Helena.

The plant has until 2021 to meet the much higher treatment standards required by its new regulatory permit, but city officials still don't know what the upgrades will look like – or, just as important given the city's financial constraints — how much they will cost.

"There's a lot of deferred maintenance that we have to catch up with, and there are also these new regulatory changes," Public Works Director Steve Palmer said during a recent tour of the city's public facilities.

The plant's new permit was accompanied by a cease-and-desist order that lays out a timeline for the city to meet the heightened treatment standards. The city has until December to complete a feasibility study assessing its options.

The state has already required Calistoga and Yountville to upgrade their treatment plants to meet the new standards, but the St. Helena plant has a unique design that will call for a different, and possibly more expensive, upgrade that will likely result in higher operating costs.

The plant at the end of Chaix Lane was designed in the early 1960s by William Oswald, a professor at UC Berkeley and a pioneer in the field of ecological engineering. It was revolutionary for its time, replacing energy-intensive mechanical and chemical processes with algae that interact with sunlight and cleanse wastewater of impurities as it passes through a snaking series of ponds.

The experiment was radical but successful, resulting in a plant that's large (22 acres) but far more energy-efficient and environmentally friendly than traditional systems. The plant was upgraded in the 1980s, and its capacity was increased further with the addition of four new brush aerators in 2007.

The problem now is not capacity, but the quality of the treated water. The San Francisco Bay Regional Water Quality Control Board wants the plant to cut down on the biochemical oxygen demand and total suspended solids contained in the discharged effluent.

That's where the plant's unique design presents a challenge. Modern wastewater is far more potent than the sewage the plant was designed to treat, and although the plant is still functioning admirably, the new standards are beyond the plant's capabilities.

Traditional plants can just be altered to do more of what they're already doing, but St. Helena's plant will have to do new tasks like filtration and pumping.

"It increases our costs exponentially," Palmer said. "Other plants are already running pumps and aerators and digesters, so they're already spending a lot on energy costs."

Although the permit doesn't require the city to reuse the treated wastewater, which is currently discharged to the spray fields adjacent to the plant, the regional water board wants the city to at least investigate that option, Palmer said.

"It's part of a concerted statewide effort by the board to push people toward beneficial reuse and recycled water," he said. "The rules are changing and we have to keep up with them."

Aside from the regulatory requirements, the building where the plant operators work needs to be improved, Palmer said. There's no separation between the regular work space and the lab area where chemicals are used.

The work space is also directly above the intake channel where raw sewage enters the plant. The building is well-ventilated, but one worker said he can occasionally feel his eyes burning from fumes that waft upward from the untreated sewage.

Corporation yard

Palmer also gave a tour of the corporation yard at the end of Charter Oak Avenue, where city vehicles are stored and maintained, and where the city stores boxes of documents it doesn't have room for at City Hall.

The property is vulnerable to flooding, with about a dozen workers operating out of what were supposed to be temporary trailers installed in 2006 after the last flood.

"When the creeks go up, we're the first to flood," said Jim Haller, parks supervisor and city arborist. "Before we do any emergency work for the city, we have to evacuate the corp yard and get everything to high ground."

As with the wastewater treatment plant, the city hasn't identified a funding source to protect the corp yard from flooding, find a more suitable site, replace the portables, or shore up sheds that are visibly sagging.

"We don't have a corp yard master plan," Palmer said. "But you can look around and see how much needs to be done."

City workers do an admirable job keeping the town running on a day-to-day basis, Palmer said, but the city government has neglected deferred maintenance, long-term asset management planning, and the continual task of keeping up with regulatory and code requirements.

"We've lacked the vision and leadership to be able to look forward," he said.

141-year-old sewer main breaks, leaving sinkhole in SF street

By Nanette Asimov
Published 9:56 pm, Tuesday, May 10, 2016

Mission Street between New Montgomery and Second Street in San Francisco will be closed off at least through Wednesday, and buses are being re-routed, after a gaping sinkhole opened up Tuesday when a sewer main broke apart after 141 years of service. No one was hurt when the sinkhole appeared around 5 p.m., though a minivan was briefly caught in the trap.

“It’s a large sinkhole,” said Charles Sheehan, spokesman for the San Francisco Public Utilities Commission.

The hole is 12-by-5 feet wide, and 9 feet deep. The sewer main itself was so large — 5 feet high and 3 feet wide — that it was considered “walkable” when it was constructed in 1875.

“You’d have to crouch, though,” Sheehan said.

Crews will mobilize through the night and will begin repairing the sinkhole Wednesday morning, when they’ll get a better sense of how long it will take to fix, he said.

Meanwhile, the 14 and 14R MUNI buses are being rerouted to Market Street, said Paul Rose, a Muni spokesman.

“We’ll have people on the street guiding people where they need to go,” Rose said.

Nanette Asimov is a San Francisco Chronicle staff writer. Email: nasimov@sfchronicle.com

EAST BAY TIMES

State to relax water mandates

RICH PEDRONCELLI/ASSOCIATED PRESS ARCHIVES

BRIAN VAN DER BRUG/LOS ANGELES TIMES

Lexington Reservoir, Los Gatos 2015/2016

PATRICK TEHAN/STAFF PHOTOS

Anderson Reservoir, Morgan Hill 2015/2016

PATRICK TEHAN/STAFF PHOTOS

Then and now: *With reservoirs bulging from El Niño rains, Brown administration moves to let local agencies set targets*

By Paul Rogers

5/10/2016

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California’s historic drought rules are going to be a whole lot looser this summer. In a major shift, the administration of Gov. Jerry Brown announced Monday plans to drop all statewide mandatory water conservation targets it had imposed on urban areas last June.

The new rules, which are expected to be approved May 18 by the State Water Resources Control Board, would instead allow more than 400 cities, water districts and private companies to each set their own water conservation targets, as long as they report them to state officials.

Water agencies, particularly in Southern California and around Sacramento, had complained bitterly about the statewide rules, saying that they were costing hundreds of millions of dollars in lost water sales and did not accurately reflect each community’s water supply conditions — and many have already begun to soften the rules for this summer.

The reversal would end one of Brown’s biggest conservation tools that forced communities to cut water consumption statewide by nearly one-fourth since June 2015 to cope with one of the worst droughts in state history.

Brown administration officials said the proposed relaxation in the rules reflects an improving water picture. This winter was the wettest in Northern California since the five-year drought began, with big reservoirs such as Oroville and Shasta now more than 90 percent full, although Southern California received far less rain.

“We are trying to recognize that conditions have changed this year and while we are in a statewide drought, conditions have eased for some parts of the state,” said Mark Cowin, director of the state Department of Water Resources.

But some environmentalists said the state should have kept the mandatory statewide targets in place, arguing it is unclear how long the drought will last, and noting that any water not used on lawns could be used instead for human consumption, firefighting and other needs.

“I’m very disappointed, but not surprised. They were getting a lot of criticism for the regulations and they sensed a waning tolerance for the targets,” said Sara Aminzadeh, executive director of the California Coastkeeper Alliance in San Francisco.

Water-wasting rules

Environmentalists, however, were among those who applauded another decision from the Brown administration Monday: to make permanent a series of water wasting rules it put in place in July 2014. Those rules ban watering lawns within 48 hours of rain, hosing off sidewalks and driveways and using ornamental fountains unless the water is recirculated.

Those rules also require shut-off nozzles on hoses used to wash a vehicle, and they ban cities and local governments from irrigating ornamental turf on public street medians.

The more immediate impact on most Californians, however, will come from the plan to drop the statewide conservation targets. Under those rules, last June each community was given a water conservation target — from 8 to 36 percent — based on its per capita water use, with fines for failure to meet the targets.

Places that already had high conservation rates, such as Santa Cruz, Hayward and San Francisco, were given 8 percent targets. Areas with high water use, like Beverly Hills and Bakersfield, were given 36 percent targets. Most Bay Area cities were at 16 to 20 percent, and hit the mark.

Under the new rules, each community now instead will set its own conservation target and report it to the state water board by June 15. The target would be based on a forecast — which water board staff members called “a stress test” — in which supply conditions would mirror the past three years, and demand would be the average of 2013 and 2014.

Using those assumptions, each city, water district and private water company would set its own conservation goal.

Sources said the Brown administration essentially cut a deal with the big water suppliers: Go along with the permanent water wasting rules and monthly reporting requirements, and the state would drop the onesize- fits-all mandatory water conservation targets.

At a news conference where reporters’ questions were cut off early, Brown administration officials insisted that they weren’t capitulating to large water agencies’ demands by setting rules that would allow spigots to be opened wide this summer on lawns from Palm Springs to San Diego to Silicon Valley.

“This is not a walk in the park,” said Felicia Marcus, chairwoman of the State Water Resources Control Board. “This is much more tailored to the circumstances that we find ourselves in now.” Max Gomberg, a water board official, said that the targets and methodologies of local agencies, along with their water use, will be posted monthly on the state water board’s website.

“If any agency is fabricating or falsely providing information, the board has remedies for that, in terms of enforcement actions and fines,” Gomberg said.

25 percent goal

FromJune2015toMarch 2016, Brown asked Californians to cut water use 25 percent overall in urban areas, compared with 2013. They reduced water use by 23.9 percent. Already some Northern California agencies have begun to ease their rules. The East Bay Municipal Utility District, which serves 1.4 million people in Alameda and Contra Costa counties, is scheduled to vote Tuesday to drop its drought surcharges. Santa Cruz ended drought penalties and rules limiting when lawns could be watered after its reservoir, Loch Lomond, filled. And the Santa Clara Valley Water District is set to vote

June 14 on easing its call for 1.9 million residents to cut water use 30 percent, a vote that will affect San Jose Water Co. and other Silicon Valley providers.

In the coming weeks, many Bay Area residents will learn how their local city or water company has altered rules, said Nicolle Sandkulla, executive director of the Bay Area Water Supply and Conservation Agency, a group of 26 agencies that receive water from San Francisco's Hetch Hetchy system.

"Storageishigh.Demand is low," she said. "The feeling is we will be able to relax the restrictions. Customers won't have to go to extraordinary measures like they have in recent years, but we still want to continue with the wise use of water."

Paul Rogers covers resources and environmental issues. Contact him at 408-9205045. Follow him at Twitter.com/PaulRogersSJMN.

EAST BAY TIMES

Instituted as conservation incentive

Tide turns on drought fee

EBMUD to drop 25% surcharge, saying it has plenty of water

By Denis Cuff

May 11, 2016

dcuff@bayareanewsgroup.com

OAKLAND — Declaring its drought emergency over, the East Bay Municipal Utility District on Tuesday scrapped a 25 percent surcharge that added about $8 a month to the average household water bill over the past 11 months.

A day after Gov. Jerry Brown announced plans to eliminate mandatory state water conservation targets for all water districts, EBMUD directors agreed unanimously to end the surcharge June 30 and go back to voluntary conservation.

The district had imposed the surcharge on July 1 as an incentive for its 1.4 million customers in Alameda and Contra Costa counties to save water. The surcharge is raising some $50 million to buy additional water from near Sacramento and to pay for conservation efforts.

But EBMUD says it has plenty of water this year after a wet winter.

"The current drought emergency is over," said Frank Mellon, water board president. "But we need to live and plan on the basis that there are always going to be droughts, and we're between chapters of a drought."

Ending the surcharge will shave about $8.08 off the monthly water bill of an average household using 246 gallons per day, district officials said.

But average bills already are scheduled to go up $3.66 a month on July 1 in a general rate increase to cover inflation and stepped-up pipe replacement.

In the net effect of the two actions, the average bill will drop by $4.42 a month from $60.25 to $55.83 per month.

Board member John Coleman said he had some qualms about collecting the surcharge for nearly two more months from customers who have done a great job of saving water.

But Sophia Skoda, the district finance director, said that while the surcharge will bring in $50 million by June 30, the district incurred some $75 million in drought-related costs to buy surplus water, expand conservation programs, educate the public and offset lost revenues due to customers saving more water than anyone expected.

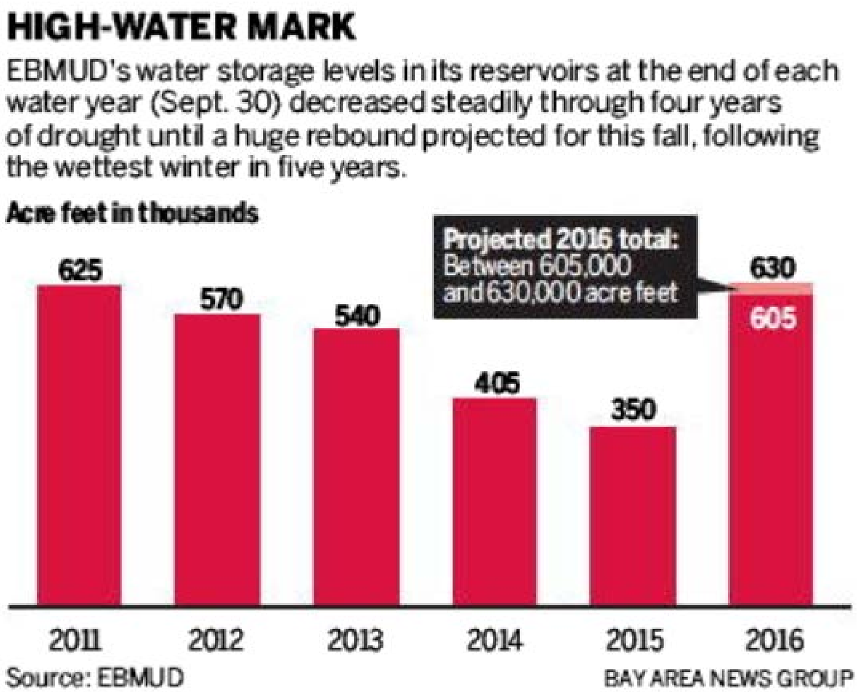
“Our customers really blew us out of the water,” Skoda said of the conservation.

Suspending the surcharge immediately instead of June 30 would leave the district another $5 million in the hole, she said.

Colemanandotherboard members said they would leave the surcharge in place until June 30 because they didn’t want to jeopardize the district’s credit rating.

“My explanation to our customers is that we incurred costs because of the drought, and we’re going to recover them,” said board member Doug Linney.

The state last year ordered EBMUD to cut use by 16 percent, and the district set an even stricter internal target of 20 percent. District customers exceeded both targets by conserving about 24 percent.



Contact Denis Cuff at 925-943-8267. Follow him at Twitter.com/deniscuff or Facebook.com/denis.cuff.

News | May 12, 2016

Senate Passes Energy And Water Appropriations Bill

Washington — The Senate today passed the fiscal year 2017 Energy and Water Development appropriations bill by a vote of 90-8. The bill funds key government agencies including the Department of Energy, the Corps of Engineers, the Bureau of Reclamation and the National Nuclear Security Administration.

"This is the first Energy and Water appropriations bill the Senate has passed under 'regular order' since 2009 and I hope it restores the committee's ability to do its work and pass appropriations bills this year," said Senator Dianne Feinstein (D-Calif.), ranking member of the Energy and Water Development Appropriations Subcommittee. **"Chairman Lamar Alexander was a great partner in this effort and a big reason we were able to pass this bill on its own for the first time in six years. I'm hopeful that the strong bipartisan vote is a sign that the Senate's appropriations process can get back on track."**

California drought

The bill includes $100 million for the Bureau of Reclamation's Western Drought Response program to help combat the historic drought in California and other Western states through direct, immediate actions to extend limited water supplies.

"Drought in the West poses a serious threat to the economic and social wellbeing of the United States," Feinstein said. **"This $100 million is critical to operating water systems more flexibly and efficiently, restoring critical wetlands and habitat and ensuring that the best science and monitoring is being brought to bear on this crisis."**

Additional projects

- U.S. Army Corps of Engineers: The bill provides $344 million for U.S. Army Corps of Engineers to fund the nation's water infrastructure projects in California. Every one dollar spent on Army Corps of Engineer projects nets $16 in economic benefits.
- California Ports: The bill provides $50 million for ports, including Los Angeles/Long Beach and other California ports that get shortchanged by the current disbursement formula of the Harbor Maintenance Trust Fund.
- Bureau of Reclamation: The bill provides $1.275 billion for the Bureau of Reclamation within the Department of Interior to fund water supply projects and programs in the western United States. In addition to the $100 million for urgent drought relief, the bill includes $117.46 million for water projects in California.
- Energy Efficiency and Renewable Energy Programs: The bill provides $2 billion for energy efficiency and renewable energy programs. This funding supports sustainable transportation programs that develop new fuels, lightweight materials, and vehicle engines; energy efficiency programs that develop standards and technologies to reduce energy bills; and renewable

energy programs that work to lower the cost of solar, wind, geothermal, and water power technologies.

- Basic Scientific Research: The bill provides $5.4 billion for the Office of Science, $50 million more than fiscal year 2016. Nearly all Office of Science programs see significant increases and the bill fully funds the requested operational levels of scientific facilities at the national laboratories.
- Environmental Cleanup: Cleanup of Cold War and other nuclear sites is funded at $6.4 billion. This program addresses a legacy of radioactive and hazardous contamination at sites across the country and the bill addresses many of the highest environmental risks posed by these sites.
- Nuclear Weapons and Nonproliferation: The bill funds the National Nuclear Security Administration at $12.9 billion. Efforts to extend the life of the current nuclear weapons stockpile are fully funded. The Naval Reactor program fully funds the Ohio Class replacement reactor program. Funding for the Nonproliferation program meets the budget request level.

SOURCE: Office of Senator Dianne Feinstein

DATE: May 16, 2016

TO: Board of Directors - Union Sanitary District

FROM: Paul R. Eldredge, General Manager/District Engineer
Pamela Arends-King, Business Services Manager/CFO
Maria Buckley, Principal Financial Analyst

SUBJECT: Agenda Item No. 9 - Meeting of May 23, 2016
Scheduling Public Hearing to Consider Collection of Sewer Service Charges on the Tax Roll for Fiscal Year 2017

Recommendation:

Set the time for holding the public hearing to consider collection of sewer service charges on the tax roll for fiscal year 2017, at 7:00 p.m. or as soon thereafter as the matter may be heard, on June 27, 2016 ~~June 23, 2016~~, in the Boardroom at 5072 Benson Road, Union City, California.

Background:

On January 25, 2016, the Board approved sewer service charge rates for fiscal years 2017 through 2021. The collection of the sewer service charges on the tax roll requires an annual hearing and consideration of the Board. The District may authorize the sewer service charges for fiscal year 2017 to be collected on the tax rolls, consistent with past practices, by 1) creating a report setting forth the amount of the sewer service charges to be assessed on each parcel in the District; 2) filing the report with the Secretary of the Board; 3) scheduling a public hearing for the Board to hear all objections and protests (if any); 4) and authorizing the collection of the sewer service charges on the tax rolls, if there is no majority protest.

If the Board would like to consider placing the sewer service charges for fiscal year 2017 on the tax rolls, it should set the date for the public hearing to consider authorizing the collection. After the hearing is set by the Board, staff will prepare the report to be considered at the public hearing and will publish the attached Notice of the time and place of the hearing in the Argus newspaper on June 3, 2016, and June 10, 2016, and in the Tri-City Voice on June 7, 2016 and June 14, 2016.

Desk Item
Item 9
5/23/2016

UNION SANITARY DISTRICT

NOTICE OF FILING REPORT AND PUBLIC HEARING IN CONNECTION WITH THE COLLECTION OF FISCAL YEAR 2017 SEWER SERVICE CHARGES ON THE PROPERTY TAX ROLL

NOTICE IS HEREBY GIVEN that pursuant to Sections 5471 and 5473, et seq. of the Health and Safety Code of the State of California and Union Sanitary District Ordinance No. 31, the Board of Directors of Union Sanitary District will consider whether to collect its charges for sewer services for fiscal year 2017 on the tax roll, in the same manner as general taxes, consistent with past practices.

The District has filed a written report with the Secretary of the Board of Directors describing each parcel of real property subject to the charges and the amount of the charges against that parcel for fiscal year 2017. The District's report is on file and available for public inspection at the District Offices.

For reference, the charges for a single family home owner (the majority of USD's customers) are based on the adopted rate of $380.05 for Fiscal Year 2017. All other rates for individual customers can be found by contacting the District at (510) 477-7500 or on the Districts website www.unionsanitary.ca.gov/sewerservice.htm

NOTICE IS FURTHER GIVEN that on Monday, the ~~23rd~~ 27th day of June 2016, at the hour of 7:00 p.m.or as soon thereafter as the matter may be heard, at the Union Sanitary District Boardroom, 5072 Benson Road, Union City, California, in said District, the Board will hold a hearing to consider the report and whether to collect the sewer service charges for fiscal year 2017 on the property tax roll. At the hearing, the Board of Directors will hear and consider all objections or protests, if any, to the District's report. Any questions regarding the charges may be directed to Business Services Manager/CFO Arends-King.

Publish dates: June 3, 2016 – Argus
June 10, 2016 – Argus
June 7, 2016 – Tri-City Voice
June 14, 2016 – Tri-City Voice

By order of the Board of Directors of Union Sanitary District.

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