EAST BAY
MUNICIPAL UTILITY DISTRICT

November 19, 2024

VIA E-MAIL

Subject: Independent Peer Review for Seismic Retrofit Design

To Whom It May Concern:

The East Bay Municipal Utility District (District) is seeking structural engineering services to perform an Independent Peer Review (IPR) of the seismic retrofit design for the Main Wastewater Treatment Plant (MWWTP) Influent Pump Station Resiliency Project.

The MWWTP is located in Oakland, California. The Influent Pump Station is responsible for lifting all wastewater flows into the MWWTP and is the most critical structure at the treatment facility. The Influent Pump Station was constructed in 1951, underwent an expansion with significant modifications in the early 1990s, and has only had minor structural changes since then. A seismic retrofit preliminary design for the Influent Pump Station was completed in November 2024.

A separate request for proposals (RFP) has been issued concurrently for the Influent Pump Station Resiliency Design Project. Firms interested in the IPR are advised to review the Influent Pump Station Resiliency Design Project RFP to understand the scope of the entire project. Interested IPR firms are also welcome to attend the pre-proposal and site walk for the Influent Pump Station Resiliency Design Project on December 5, 2024.

Design milestones will be at the 30%, 60%, 90% and Final Design submittals. While the Influent Pump Station Resiliency Design Project contract scope includes more than seismic retrofit work, the IPR will only provide review for the seismic retrofit design portion of the project. Award for the design contract is expected to be concurrent with the award of the IPR contract.

The Independent Peer Reviewer shall coordinate all IPR efforts, findings, resolutions, and deliverables directly with the District’s Project Manager. The IPR shall be consulted if the Design Consultant wants to make changes to the existing seismic Basis of Design. The IPR shall participate in meetings with the District and Design Consultant as needed to discuss and resolve IPR findings after each submittal. Submittals are expected with the Basis of Design Report, 30%, 60%, 90% and Final Design.

The IPR shall also attend the following key project meetings:

- Kick-off Meeting
- Updated Basis of Design Meeting
- 30 Percent Design Workshop
- 60 Percent User Group Meeting

- 90 Percent User Group Meeting
- Final Design Briefing

Please refer to attachments for overview of layout and structural information for the Influent Pump Station. Attachment A includes excerpts from the 2024 seismic retrofit preliminary design.

The IPR aim is to validate that the design achieves the desired seismic performance objectives. This shall include, but is not limited to:

- Ensuring that the appropriated codes and guideline are applied
- Validating of design assumptions and approach to design
- Considering alternative design options
- Confirming selection of materials and constructability

IPR efforts shall be performed in accordance with the industry peer review standard of care. Engineers undertaking the IPR shall perform reviews and document the findings under the following anticipated tasks:

Task 1: Project coordination including management efforts and invoices with the District. Invoices shall contain, at a minimum, District purchase order number, invoice number, remit to address, and a short description of services completed that month.

Task 2: Data Collection and Review

- Perform site visit
- Review Influent Pump Station record drawings and reports provided by the District. Reports will include geotechnical reports, District design criteria, concrete testing reports and previously completed structural design reports.
- Review Influent Pump Station SAP2000 model. IPR will not be expected to modify the model. The model is provided for background information purposes.

Task 3: Independent Peer Review

- Perform reviews and evaluations on design submittals
- Attend key project meetings and workshops
- Meet with District and preliminary designers to address questions and discuss findings.
- Log findings and issues in an IPR log
- Summarize IPR approach and findings in technical memorandum with IPR log attached

Deliverables: The Peer Reviewer shall document findings and issues in a peer review log following each design submittal (30%, 60% and 90%). The peer review log shall be submitted within two weeks of each task deliverable by the Design Consultant and shall include a rolling peer review log of findings, issues, and resolutions. Upon incorporation of 90 percent comments and resolution of IPR findings, the Peer Reviewer shall produce a IPR Technical Memorandum noting review process and documenting findings and outcome. The final rolling peer review log shall be attached to the memorandum.

Project Schedule*

The proposed project schedule is as follows:

Project Kick-off	April 2025
Updated Basis of Design Report	April 2025
30% Design submittal	June 2025
60% Design submittal	July 2025
90% Design submittal	October 2025
Final Design submittal	January 2026

* The dates listed is the current schedule with FEMA. This time frame will be extended based on proposals recommendations from the Influent Pump Station Resiliency Design. IPR proposals may provide an estimated project schedule with a note that the schedule will be determined by the Influent Pump Station Resiliency Design contract. A greater emphasis for the IPR proposal should be placed on labor hours by task or submittal.

If your firm is interested in performing this IPR, please provide a brief letter proposal that includes a scope of services, project approach, budget, schedule, and key personnel by December 20, 2024.

Proposal Length:

Item	Page Limit
Proposed Services and Project Approach	3
Project Management and Staff	1
Budget	1
Schedule and Labor Hours by Task	1
Exceptions, Clarifications, Amendments	As needed
Resumes	Max 2 per person

If you have any questions, please feel free to contact me by phone at 510-286-2681 or email at jennifer.ku@ebmud.com.

Sincerely,

Jennifer Ku

Jennifer Ku, P.E.
Project Manager
Wastewater Engineering Division

JSK:jsk

Attachment

ATTACHMENT A

Task 4.3 IPS Seismic Retrofit Preliminary Design Report

Document No.: 230323151650_7f64a922
Revision: Draft

East Bay Municipal Utility District

Influent Pump Station Resiliency Project
November 14, 2024

Prepared by
Jacobs

Contents

Attachments

Executive Summary

East Bay Municipal Utility District (EBMUD) is improving the resiliency of the Influent Pump Station (IPS) for the Main Wastewater Treatment Plant (MWWTP), located in Oakland, California. Jacobs was contracted to serve as the Design Engineer for the project. The IPS is a 425-million gallon per day pump station that is critical to maintaining flow into the MWWTP. The resiliency project includes seismic, electrical, and ancillary equipment upgrades and potential improvements to pumping.

A preliminary design for seismic retrofitting the IPS has been completed and is detailed in the following report and drawings. The IPS was evaluated for ASCE41 Basic and Enhanced seismic performance objectives. The Basic seismic performance objectives are the minimum required by code, while the Enhanced seismic performance objectives are consistent with those for essential facilities such as hospitals. The seismic hazard levels considered in this retrofit include the Basic Safety Earthquake-1 (BSE-1E), which has a 20% probability of exceedance in 50 years, and the Basic Safety Earthquake-2 (BSE-2E), which has 5% probability of exceedance in 50 years. The IPS was analyzed for these earthquakes using a SAP2000 model for each of the following main IPS components:

- Pumping Station
- Grit Facility
- Fine Screening Handling Building
- Sky Bridge

Material evaluation was also completed to determine the existing concrete compressive strength. This evaluation consisted of reviewing the original construction concrete cylinder test results and taking concrete cores of the IPS walls and slabs. The concrete compressive strength used in the analysis was based on the new concrete core test results, which results in a small increase in strength compared to that specified in the original design.

The IPS does require seismic retrofit to achieve both sets of seismic performance objectives. The retrofits include the following:

- Strengthening the roof diaphragms around the existing roof openings, including added concrete and steel beams and additional welding of metal roof deck
- Applying concrete overlay on select shear walls
- Applying fiber-reinforced polymer strengthening systems on select shear walls
- Applying fiber-reinforced polymer confinement of main Pump Station columns
- Strengthening the braced frame connections, columns, and collector beams in the fine screening room
- Adding anchors to secure the brick veneer to the supporting concrete wall in the fine screening room
- Adding anchors and strengthening the connections between precast concrete wall panels and braced frames in the fine screening room
- Strengthening the pumping channel walls at the cross gate openings with fabric reinforced cementitious matrix
- Placing piles and pile caps around three sides of the Grit Facility
- Increasing capacity for movement at expansion joints, including in the five pipes exiting the pumps into the Grit Facility
- Modifying equipment anchorages and adding bracing

The preliminary design drawings show the above retrofits planned for the Enhanced level criteria as recommended. At the Basic level, almost all of the above retrofits are required. While the IPS could be retrofitted to the lower-level Basic seismic performance objective, only modest increases are required to achieve the Enhanced performance objective. The following minor retrofits could be eliminated under the Basic performance objective:

- Strengthening ground floor beams at the Pump Station with fiber-reinforced polymer
- Adding roof beams at the upper roof of the Pump Station
- FRP retrofit of the pier at Grid Line C3 at the Pump Station

While the added piles at the Grit Facility could be slightly reduced in number for the Basic seismic performance objective, those piles add to the control of displacements at the expansion joints between structures thus eliminating some is not recommended. Overall, the cost reductions related to achieving only the Basic seismic performance objective would be expected to be less than $70,000.

Construction considerations include maintaining operation of the IPS during the seismic retrofit and avoiding shutdowns or reduced pumping capacity during the wet season. The retrofit of the channel walls around the cross gates will need to occur over two dry seasons because only two channels can be shutdown at a time during the dry season, and no channel shutdowns are allowed during the wet weather season. Temporary routing of power and controls will need to be in place before retrofitting the braced frames, as this work will require access to the frame braces, columns, beams, and connections. Concrete will need to be pumped into areas inside of the IPS for the concrete wall overlays. The piles and pile caps will require relocating buried conduit on the southern side of the IPS.

In conclusion, the proposed seismic retrofit will help EBMUD achieve the resiliency goals for the IPS. The retrofit can be implemented while maintaining operation of the IPS but does require relocating existing power and control conduits and conductors. The work will also require two dry seasons to complete because work is required in the pumping channels.

1. Introduction

East Bay Municipal Utility District (EBMUD) is improving the resiliency of the Influent Pump Station (IPS) for the Main Wastewater Treatment Plant (MWWTP), located in Oakland California. Jacobs was contracted to serve as the Design Engineer for the project. The IPS is a 425 million gallon per day pump station that is critical to maintaining flow into the MWWTP. The resiliency project includes seismic, electrical, and ancillary equipment upgrades and potential improvements to pumping.

This technical memorandum (TM) describes the preliminary design for seismic retrofits for the IPS building as conducted as part of Task 4. This TM will be combined with other TMs into a final report that will describe the full scope of the project. The final report will also include construction scheduling, costs, and staging which will carry forward to final design.

1.1 Summary of the IPS Structure

The IPS facility consists of four separate structures separated by expansion joints. Figure 1-1 shows the relation of those structures with respect to each other.

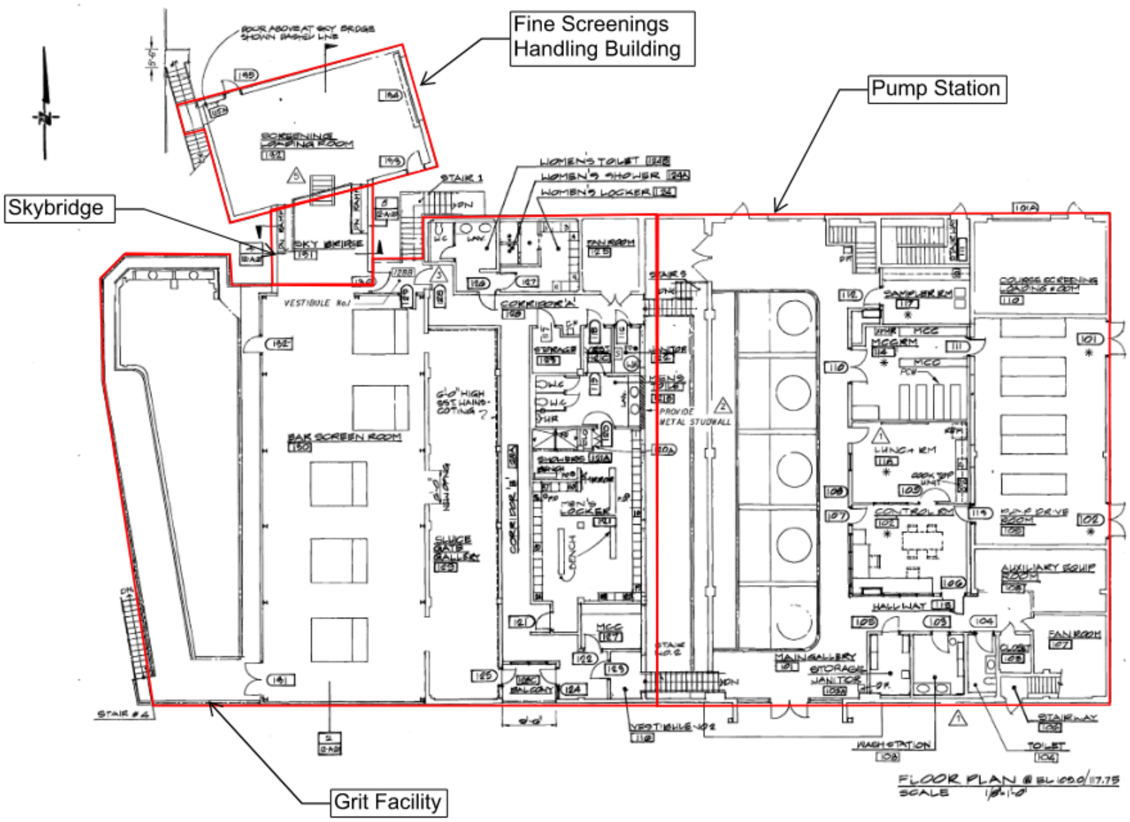


Figure 1-1. IPS Facility Elements Shown at Ground Floor Level and Above

The original construction of the four structures comprising the IPS occurred under the following construction projects:

- SD18 Project from 1949, which consisted of the Pump Station and Grit Facility. It should be noted that while the SD18 drawings call the structure the Grit Facility, the present day grit handling facilities were moved to an area adjacent to the IPS in the 1970s. For the purpose of this report, any reference to the Grit Facility will refer to the original structure constructed as part of SD18.
- SD170 Project from 1989, which consisted of the Fine Screenings Handling Building and Skybridge.

The SD170 Project work also modified portions of both the Pump Station and Grit Facility. The SD43 Project from 1950 introduced a 10-foot extension off the northwestern corner of the Grit Facility to connect to the Sedimentation Basins Influent Channel. Small changes to equipment and openings occurred in later work at the facility.

The Pump Station contains the Intake Chamber, Pump Drive Room, Lunchroom, Control Room, IPS pumps and motors, Locker Rooms, Sample Room, and pumping channels. The Intake Chamber was expanded as part of the SD170 Project work. The structure consists of reinforced concrete roof and floor slabs supported by reinforced concrete walls. This portion of the facility has a deeper basement that is approximately 35.5 feet below grade at the deepest location. The foundation is a reinforced concrete mat slab that is not pile supported.

The Grit Facility contains the fine screens and conveyors crossing the Skybridge from the screens to the Fine Screenings Handling Building. This portion of the facility is a combination of braced steel frames and concrete walls and is supported by a pile foundation. The Grit Facility is structurally isolated from the Pump Station, but the joint between the two facilities is smaller than what is needed for a seismic joint as indicated in Section 7.

1.2 Operational Requirements

The IPS must remain operable during construction of the seismic retrofit. It is assumed that the IPS can operate with a reduced pumping capacity during the dry season between May 1 and September 30. During this dry season, two pump trains, including the associated channels, can be shutdown, but three pumping trains need to remain operable at all times. During the wet season (October 1 to April 30), no shutdowns are permitted.

1.3 Permitting Requirements

The retrofit of the IPS is a voluntary improvement and is not being mandated by the building official. It is assumed that EBMUD will self-permit this work, as well as conduct an engineering peer review of the seismic retrofit. The retrofit will be funded with a FEMA Hazard Mitigation Grant and thus will require the design documents to incorporate the funding requirements and be reviewed by FEMA prior to going to bid.

2. Seismic Loads and Performance Objectives

Section 2 describes seismic loads and performance objectives related to project design.

2.1 Summary

The performance objectives for the IPS structural evaluation were selected in accordance with Table 2-1 of *ASCE 41-17, Seismic Evaluation and Retrofit of Existing Buildings* (ASCE 2017) for existing buildings, and in accordance with the EBMUD Wastewater Seismic Projects Seismic Evaluation and Retrofit Design Criteria Guidelines. The *International Building Code* assigns Risk Category III to wastewater treatment facilities and other public utility facilities, so Risk Category III will be used for this assessment for Basic Performance Objectives. Risk Category IV will be used as an Enhanced objective. This category is typical for essential facilities such as hospitals. Table 2-1 summarizes the performance objectives for the project.

Table 2-1. Performance Objectives for Existing Buildings

Risk Category	BSE-1E	BSE-2E	IPS Objective
I and II	Life Safety Structural Performance	Collapse Prevention Structural Performance	Not applicable
	Life Safety Nonstructural Performance (3-C)	Hazards Reduced Nonstructural Performance[a] (5-D)	
III	Damage Control Structural Performance	Limited Safety Structural Performance	Basic
	Position Retention Nonstructural Performance (2-B)	Hazards Reduced Nonstructural Performance[a] (4-D)	
IV	Immediate Occupancy Structural Performance	Life Safety Structural Performance	Enhanced
	Position Retention Nonstructural Performance (1-B)	Hazards Reduced Nonstructural Performance[a] (3-D)	

Source: ASCE 41-17, Table 2-1. Basic Performance Objective for Existing Buildings (ASCE 2017).

[a] Compliance with ASCE 7-16 provisions for new construction is deemed to comply.

The seismic hazard levels considered in this retrofit include the Basic Safety Earthquake-1 (BSE-1E), which has a 20% probability of exceedance in 50 years, and the Basic Safety Earthquake-2 (BSE-2E), which has a 5% probability of exceedance in 50 years.

2.2 Basic Objectives

The Basic structural performance levels selected were Damage Control Performance under the BSE-1E seismic hazard and Limited Safety Performance under the higher demands of the BSE-2E seismic hazard. Damage Control Performance is midway between Life Safety Performance—the damage level expected of a typical new Risk Category I or II building design under the Maximum Considered Earthquake in the building code—and Immediate Occupancy. Limited Safety Performance is midway between Life Safety Performance and Collapse Prevention Performance. Collapse Prevention Performance is the point at which no margin against collapse is retained for seismic loads, but the structure still supports gravity loads.

2.3 Enhanced Objectives

The Enhanced structural performance levels selected were Immediate Occupancy under the BSE-1E seismic hazard and Life Safety Performance under the higher demands of the BSE-2E seismic hazard. Life Safety Performance allows for damaged components but retains a higher margin of safety against the onset of partial or total collapse than allowed for Limited Safety.

2.4 Nonstructural Objectives

Nonstructural component screening in accordance with the Federal Emergency Management Agency's (FEMA's) standard *FEMA E-74, Reducing the Risks of Nonstructural Earthquake Damage* (2021) will be performed during this phase of the project. The FEMA E-74 evaluation will be used to identify deficiencies in nonstructural component anchorage and bracing. These deficiencies will be addressed in future phases of the project using the performance objectives shown in Table 2-1.

Position Retention is required for the lower demands of the BSE-1E seismic hazard, and Hazards Reduced Nonstructural Performance is required under the BSE-2E demands. Position Retention considers that the nonstructural components may be damaged to the point where they cannot immediately function, but they are anchored sufficiently to avoid falling over or breaking utility connections.

Hazards Reduced Nonstructural Performance allows for damage levels that can create falling hazards, but the nonstructural components are required to be anchored where that poses a risk to life safety. That anchorage will be considered in future phases of the project. Egress preservation is not required to meet the Hazards Reduced criteria; but stairways, openable doors, emergency lighting, and fire alarms should remain available under the Position Retention criteria for the more frequent earthquakes at the lower BSE-1E hazard level ground accelerations.

3. Seismic Hazards

The BSE-1E and BSE-2E seismic hazards (where "E" designates existing structures) used for this project were defined as part of the Geotechnical Study Report for the MWWTP and Dechlor/Outfall Area Site (AGS-19-030) (AGS 2020b). Figure 3-1 shows the response spectra for those two hazards, in addition to the BSE-1N and BSE-2N hazards ("N" designating new) used for evaluating new building performance for comparison. The response spectra for new structures typically produce higher demands than retrofit design for existing structures under ASCE 41-17 requirements. For the site-specific spectra shown in Figure 3-1, the demands are the same for periods of less than 0.5 seconds where the IPS structures fall.

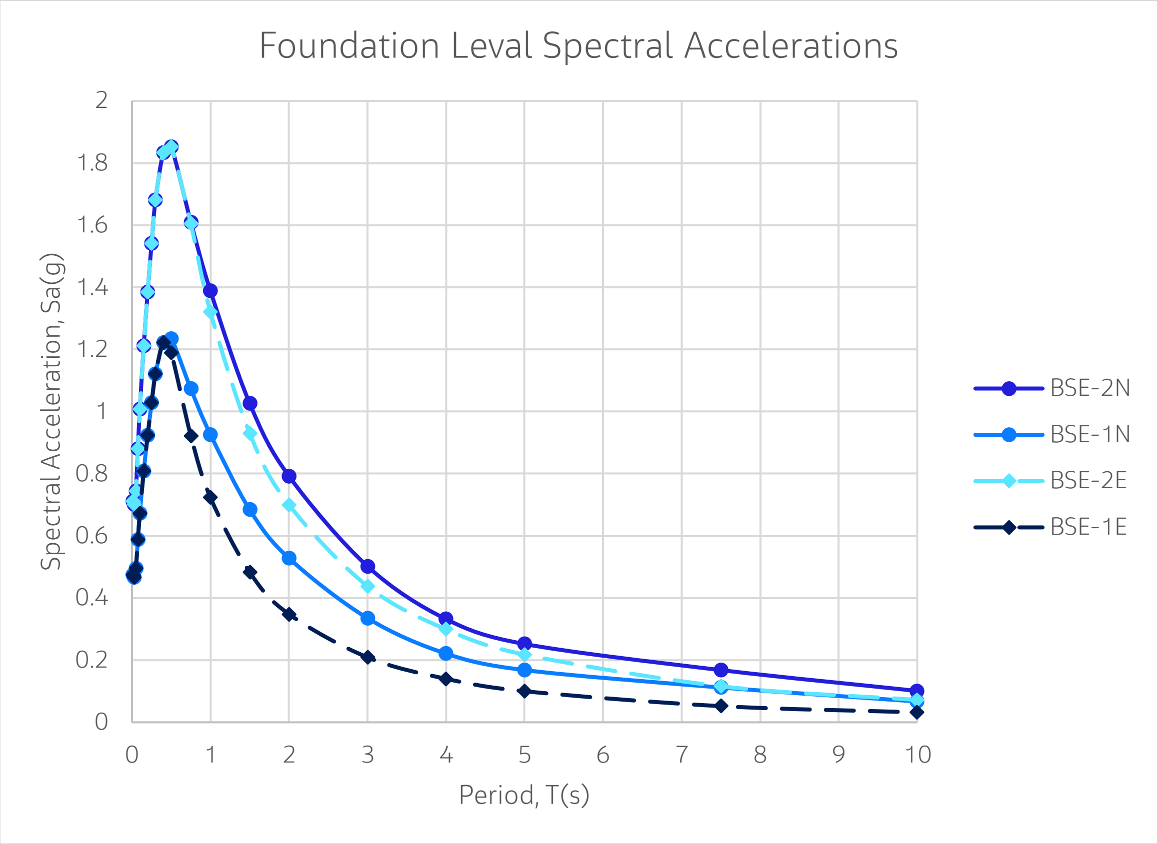


Figure 3-1. Response Spectra

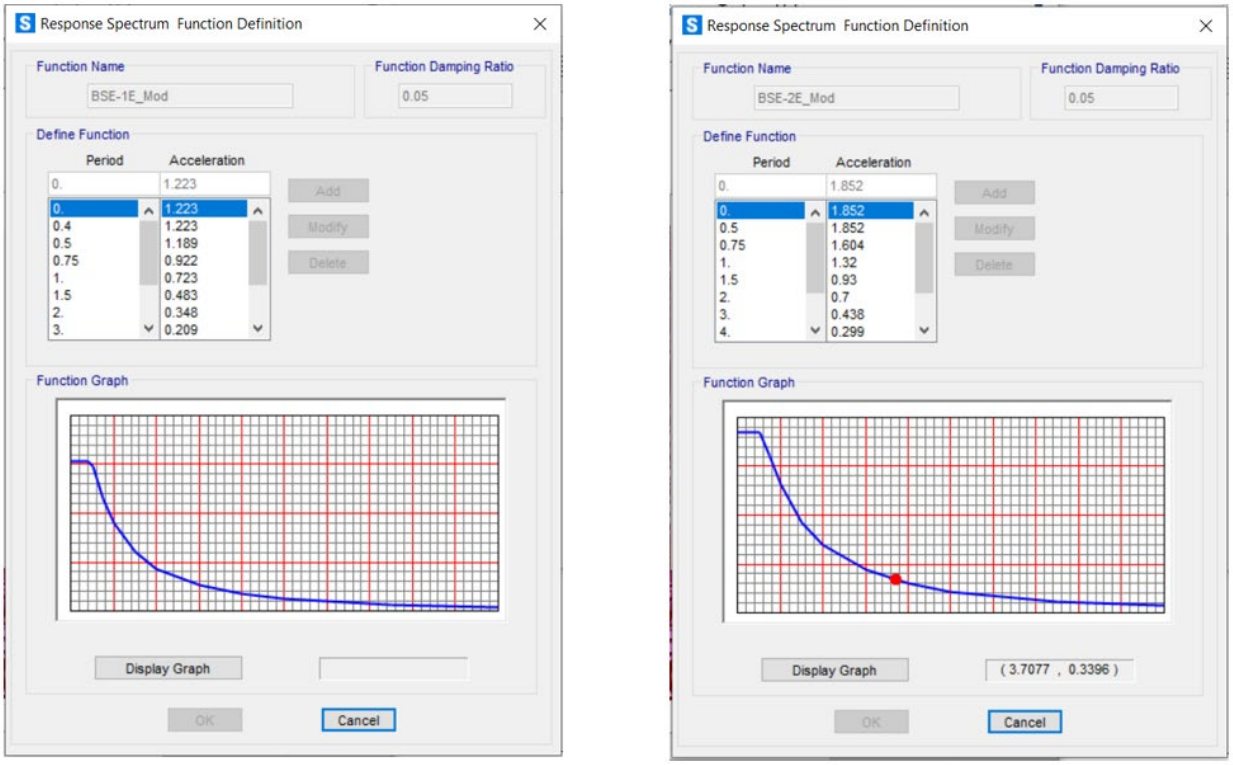
For reference, return periods associated with these hazard levels are provided in Table 3-1, as well as the probability of exceeding that level of ground motion in a 50-year design life. Return periods are considered to be the average number of years that pass before an exceedance of that level of seismic hazard.

Table 3-1. Recurrence of ASCE 41 Hazard Level Seismic Events

ASCE 41 Hazard Level	Return Period (years)	Exceedance Probability In 50 Years (%)
BSE-2N	2,475	2
BSE-2E	975	5
BSE-1N	475	10
BSE-1E	225	20

Source: ASCE 2017.

As the structure periods lengthen with damage, the response spectra input to the analysis program were modified to hold the peak acceleration constant rather than having an ascending branch of the curve from period of 0 seconds (peak ground acceleration). Figure 3-2 shows the modified response spectra used in the SAP2000 structural model.



Response Spectrum Function Definition

Function Name: BSE-1E_Mod

Function Damping Ratio: 0.05

Define Function

Period	Acceleration
0.	1.223
0.	1.223
0.4	1.223
0.5	1.189
0.75	0.922
1.	0.723
1.5	0.483
2.	0.348
3.	0.209

Response Spectrum Function Definition

Function Name: BSE-2E_Mod

Function Damping Ratio: 0.05

Define Function

Period	Acceleration
0.	1.852
0.	1.852
0.5	1.852
0.75	1.604
1.	1.32
1.5	0.93
2.	0.7
3.	0.438
4.	0.299

(3.7077 , 0.3396)

Figure 3-2. Response Spectra Used in the SAP2000 Model

7. Retrofit Requirements

The retrofit measures need to take into account the operational requirements noted in Section 1.2 above. The installation needs to minimize the invasiveness in the facility. The minimization of invasiveness involves consideration of retrofit or strengthening instead of replacement, minimization of the weight of members, and locating the retrofits in accessible areas.

7.1 Grit Facility Braced Frames

Braced frame retrofits will need to be carefully designed to minimize the impact on facility operations. This includes minimizing individual member weights and minimizing the amount of field welding.

7.1.1 Braced Frame Deficiencies

The braced frame columns on the north and south sides of the room are overstressed for both the Basic and Enhanced performance objectives. The braced frame beams on the north and south sides of the room are also overstressed for both the Basic and Enhanced performance objectives. See Section 6.3.1 of this report for which specific members are overstressed.

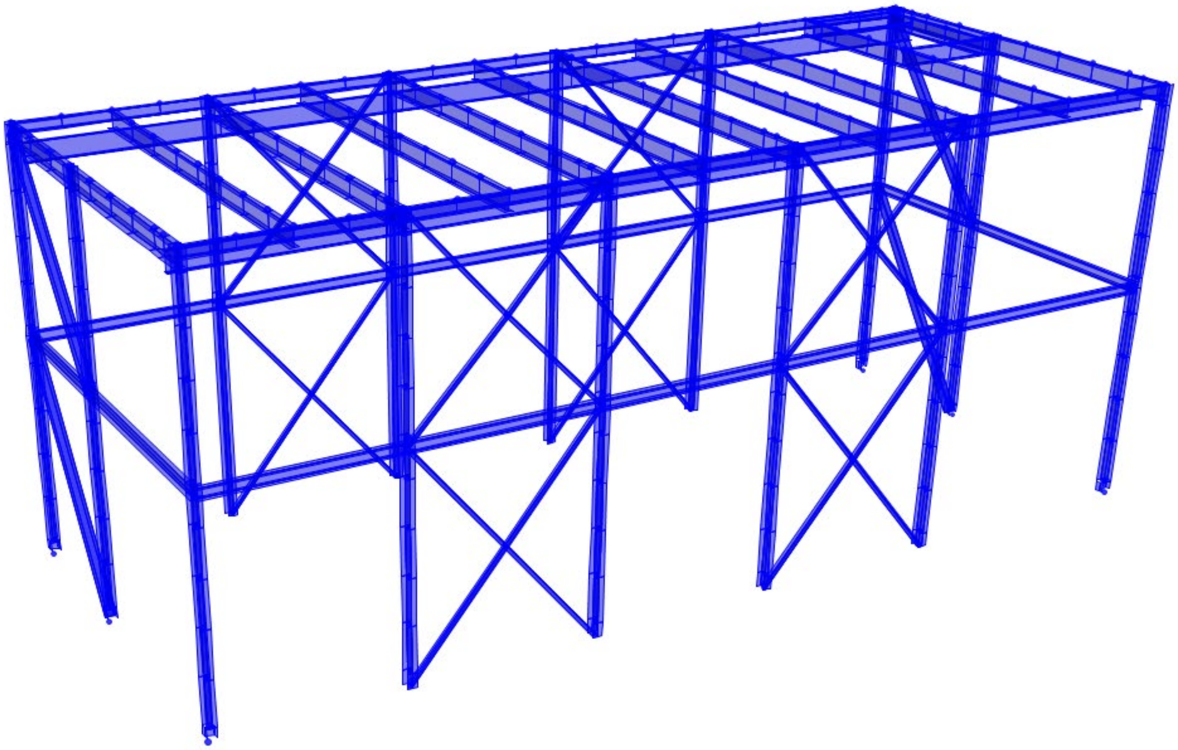


Figure 7-1. Braced Frame Configuration

The braced frame gusset plates and base plates for all of the bracing connections below EL 132.75 are overstressed for both the Basic and Enhanced performance objectives. The X-brace connections above EL 132.75 are not overstressed. The single diagonal, hollow structural section brace connections above EL 132.75 are overstressed for both Basic and Enhanced performance objectives. The overstressed connections have multiple failure modes with DCR >1.0.

7.1.2 Braced Frame Retrofits

The overstressed braced frame members can either be reinforced or replaced.

7.1.2.1 Braced Frame Reinforcing

Reinforcing the braced frame columns and beams consists of adding angle sections to the flanges of the existing members to both increase the overall area of the member and increase the section modulus. The below figure shows the general reinforcement scheme.

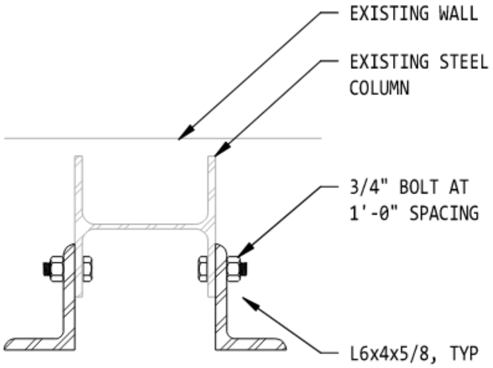


Figure 7-2. Braced Frame Column Reinforcement

7.1.3 Connection Retrofits

The braced frame connections, including gusset plates and base plates are overstressed. The connection retrofit requirements will be different between the bracing reinforcement and bracing replacement options.

For failure modes controlled by bolts, the connection will be welded to take the full load (no load sharing between bolts and welds).

See Attachment 1 for preliminary retrofit details.

7.2 Grit Facility Roof Diaphragm

There is a large opening in the roof diaphragm that will require reinforcement to provide continuity for diaphragm forces to flow around it. The reinforcement will be angle drag struts with diaphragm connections that are sized to resist the loads that the opening interrupts, see the below figure.

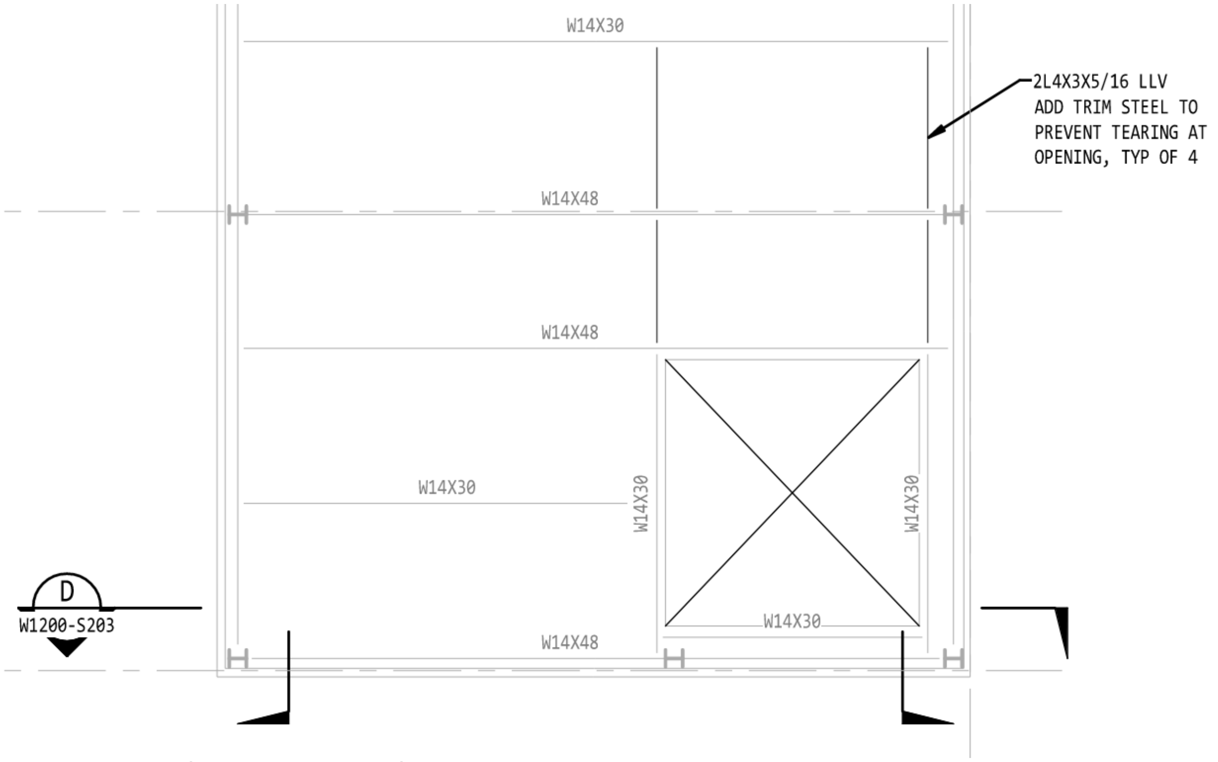


Figure 7-3. Roof Diaphragm Reinforcing

The roof decking is also overstressed. Reducing the sidelap connection spacing to 6" provides sufficient capacity for both the Basic and Enhanced performance criteria. Additional top seam welds at 12" spacing for a 6" net spacing will be provided.

7.3 Concrete Wall Retrofits

The concrete wall retrofits fall into two primary areas: overlay walls and FRP reinforcing.

7.3.1 Basic Versus Enhanced Performance Objectives for the Concrete Walls

The concrete wall sections with a DCR greater than one are typically force controlled due to the minimal existing amount of reinforcing. Adding reinforcing changes those walls from being force controlled to deformation controlled. There is, therefore, negligible difference in the amount of required reinforcing between the Basic and Enhanced performance objectives because adding sufficient reinforcing to change to a deformation controlled action allows the walls to meet either performance objective.

7.3.2 Concrete Overlay Walls

Concrete overlay walls strengthen the existing concrete sections for compression, shear, and tension by providing both additional concrete area and additional steel reinforcement. The overlay wall is connected to the existing wall with adhesive dowels to transfer loads from the existing structure into the new structure and to allow the new and existing walls to act compositely. The adhesive dowels will need to be sized and spaced to transfer the loads and the reinforcing in the overlay wall will need to be sized to allow the composite section to resist the applied forces. See the below figure for an example of a typical overlay wall.

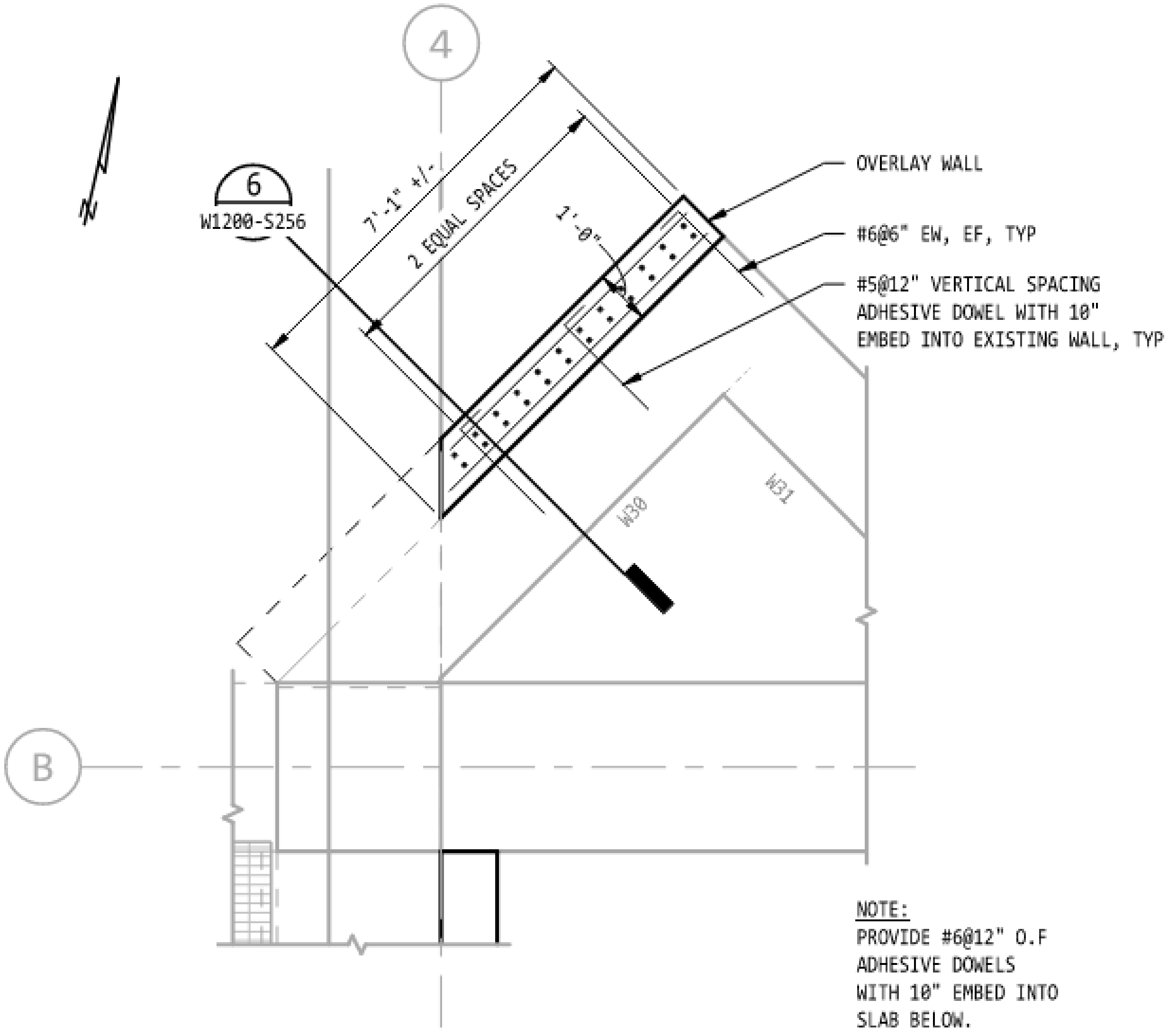


Figure 7-4. Example Concrete Overlay Wall

7.3.3 FRP Reinforcing

FRP reinforcing strengthens the existing wall for shear and tension. The reinforcing consists of FRP fabric adhered to the existing concrete wall with epoxy. The fabric can provide either unidirectional or bi-directional reinforcement. Bi-directional reinforcement is typically used where there is a deficiency in shear capacity, while unidirectional reinforcement can be used where bending capacity in one direction needs to be supplemented. Figure 7-5 shows a case where unidirectional reinforcement is used to add bending capacity in both directions using discrete strips oriented with the existing reinforcing to be supplemented.

Multiple sections of wall are overstressed because there is insufficient reinforcement in the section to preclude a brittle failure, which results in the section being considered force controlled. The addition of FRP reinforcing can help recategorize the section as deformation controlled by adding post-cracking capacity to the point where that capacity exceeds the force that caused the cracking to occur. In addition to the added capacity of the FRP, categorization as a deformation-controlled element also results in additional increased capacity from the use of the expected material strengths of the existing concrete and reinforcing bars.

Figure 7-5. Example FRP Reinforcing: seblog.strongtie.com

Typically, either concrete overlays or FRP reinforcing can be utilized to retrofit the existing concrete walls and piers. Concrete overlays have been shown as the preference where space is available and significant relocation of existing wall-mounted conduits or equipment is not required. FRP reinforcing has greater flexibility for installation around existing attachments. For either system, adhesive anchorage in drilled holes is provided where continuity through slabs and beams is required.

7.3.4 Fabric Reinforced Cementitious Matrix

The inlet channels have divider walls that were added as part of the SD170 project. The walls are thick and attract significant load, but have minimal reinforcing to resist that load resulting in a need to retrofit the walls. Retrofits in the inlet channels are subjected to a harsh environment that would greatly reduce the life of conventional FRP retrofits. Adding significant concrete thickness to the inlet channels would also restrict the flow and affect the hydraulics. Fabric reinforced cementitious matrix (FRCM) is a retrofit that overcomes the limitations of more conventional retrofit methods. FRCM uses a fiber reinforcing grid that is bonded to the existing concrete with a repair mortar instead of epoxy. The reinforcing grid is then covered with repair mortar that protects the fiber reinforcing from harsh environments and erosion. FRCM is a system that can strengthen the influent channel walls while taking up minimal space to avoid restricting flows. Refer to Figure 7-6 for an example of an FRCM retrofit.

Figure 7-6. Example FRCM Reinforcing: sika.com

7.4 Brick Veneer and Precast Wall Retrofits

The existing connection between the concrete walls and the brick veneer in the Grit Facility is unknown. The concrete and veneer need a positive connection to prevent the veneer from falling off during an earthquake. Retrofit helical ties will be provided to positively attach the brick veneer to the underlying concrete. Helical ties are intended for this sort of retrofit. Their installation is minimally invasive and they do not affect the finished appearance of the wall.

The existing precast concrete walls in the Grit Facility have multiple locations where the connection from original construction is missing the bolt. A new clip will be provided to connect the precast panels to the columns; refer to Figure 7-7.

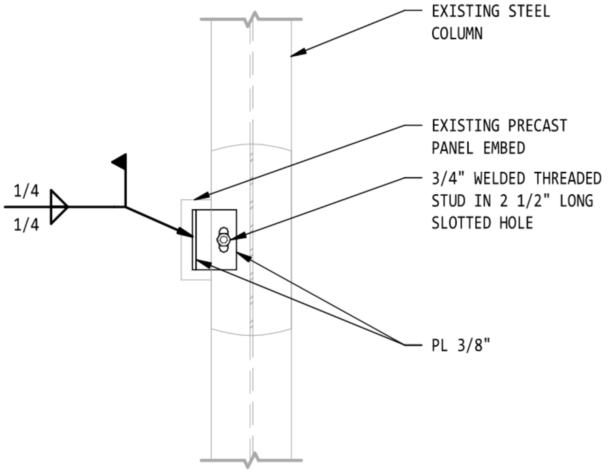


Figure 7-7. Retrofit Clip at Precast Panel

The precast panels are also missing a connection to the floor adjacent to the door openings. A post with connections to the panel and the floor will be added to provide out of plane capacity at the wall where it connects to the slab as shown in Figure 7-8.

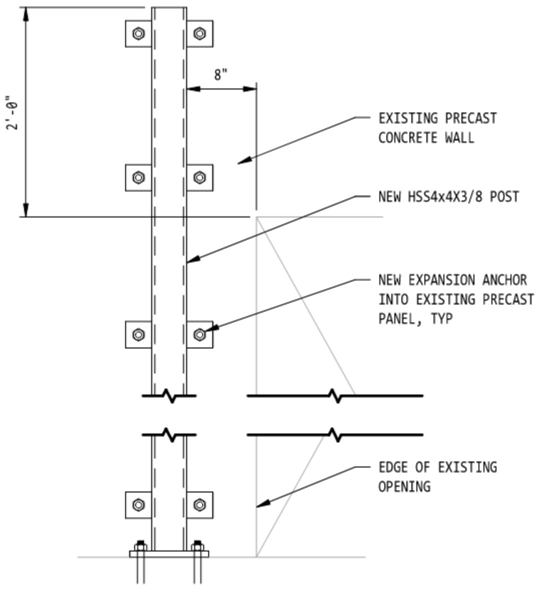


Figure 7-8. Retrofit Connection at Precast Panel

7.5 Concrete Slab Retrofits

The Pump Station lower roof slab is overstressed for both the Basic and Enhanced performance objectives. New concrete beams will be provided on top of the existing roof to add additional vertical slab capacity as well as distribute diaphragm loads around the HVAC openings that were placed in the original roof slab.

The concrete beams are shown in the Figure 7-9. Similar beams are added to the upper roof slab to address deficiencies around openings added after the original construction.

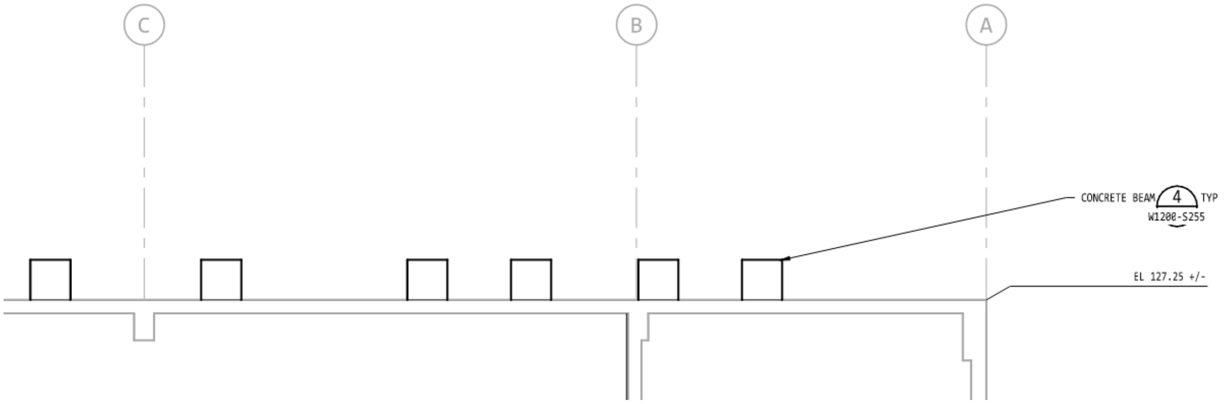


Figure 7-9. Pump Station Roof Retrofit Beams

7.6 Pile Retrofits

To minimize impacts to operations, pile retrofits at the Grit Facility are best constructed outside the footprint of the existing building.

7.6.1 Pile Deficiencies

As indicated in Section 6.2.9 above, Grit Facility piles are typically overstressed when considering lateral shear resistance given group effects on the lateral capacity. Some piles around the perimeter, particularly at the north side, are also overstressed under vertical seismic loads due to overturning of the structure. The piles at the Skybridge and the Fine Screenings Handling Building are generally acceptable, with slight overstress under the Enhanced Performance Objectives criteria from overturning forces. No retrofits are recommended at those two facilities as the use of calculated rather than tabulated C_1 and C_2 factors per Section 7.4.1.2.1 of ASCE 41 (ASCE 2017) would be expected to reduce demands to acceptable levels.

7.6.2 Pile Retrofits

Addressing the deficiencies in the existing piles can be performed by either improving the strength of the soil surrounding the piles or by addition of new piles. Soil improvements such as deep soil-mixing and jet grouting may be considered in future phases of design, but the current retrofit scheme based on available geotechnical recommendations is focused on the addition of new piles around the perimeter of the Grit Facility where space is available.

Base shears reported from the Grit Facility SAP2000 model are 16,067 kips for the BSE-2E seismic hazard and 10,796 kips for the BSE-1E seismic hazard. Average lateral demands for the 177 existing piles as reduced by the m-Factors of ASCE 41 (ASCE 2017) on soil resistance are shown in Table 7-1.

Table 7-1. Average Grit Facility Pile Shear Demands

Performance Level	Demand Capacity Ratios			
	BSE-1E Seismic Hazard		BSE-2E Seismic Hazard	
	m-Factor	Average Pile Shear Force	m-Factor	BSE-2E Life Safety
Basic Performance Objectives	2.5	24.4 kips	3.5	25.9 kips
Enhanced Performance Objectives	2	30.5 kips	3	30.2 kips

The available lateral resistance for the existing piles is shown in Figure 7-10, while Figures 7-11 and 7-12 show the resistance to vertical loads. To preclude group effects reducing the lateral pile capacity, piles are required to be spaced a minimum of 6 diameters apart on center (AASHTO LRFD Bridge Design Specifications as referenced in AGS 2020a). The existing piles within a pile cap do not meet that requirement, but piles in separate pile caps do. Therefore, the average of Rows 1 and 2 is used for the average capacity of piles in pile caps typically having four piles in two rows of two. To achieve that average capacity, the lead Row 1 piles are required to have the structural capacity to develop the 45 kip force. For the vertical loads, the same assumption of a 28-foot pile length is used as was reported by AGS based on available evidence from the original construction.

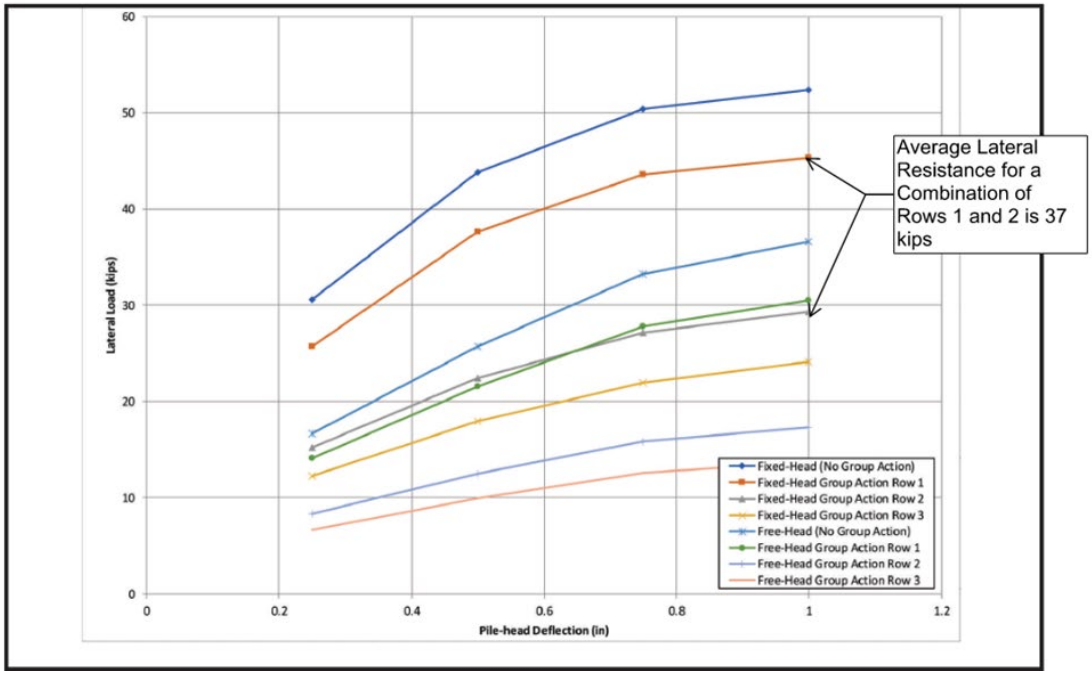


Figure 7-10. Existing Pile Lateral Capacity (AGS 2020a)

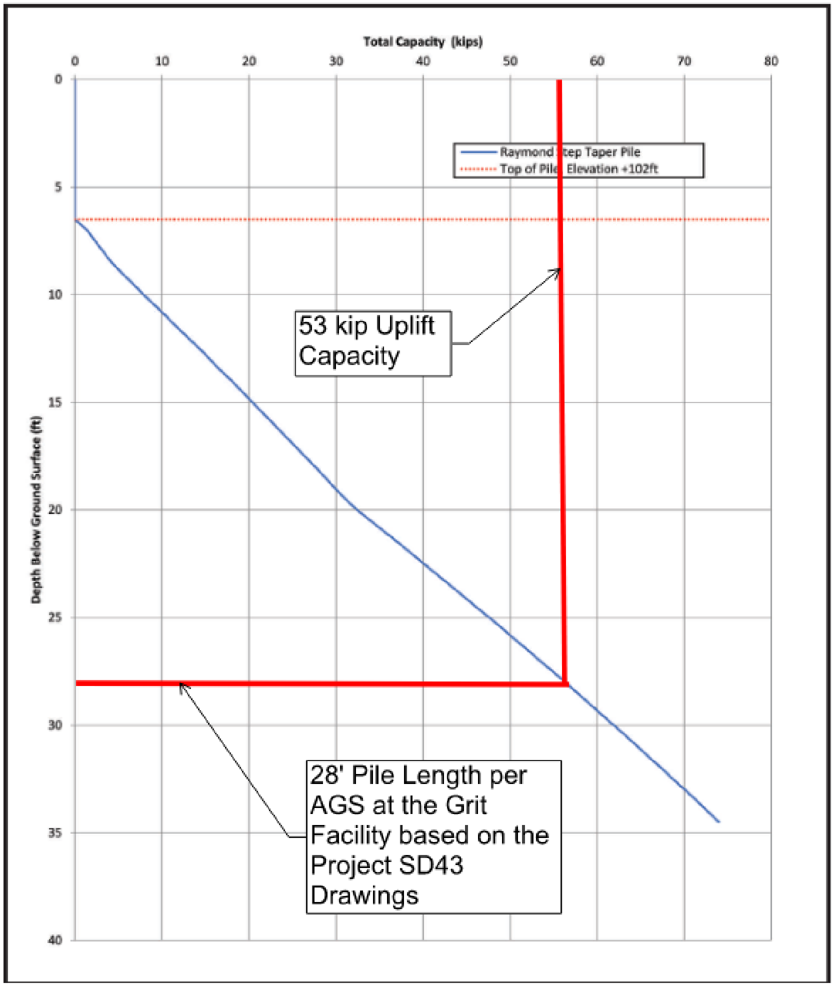
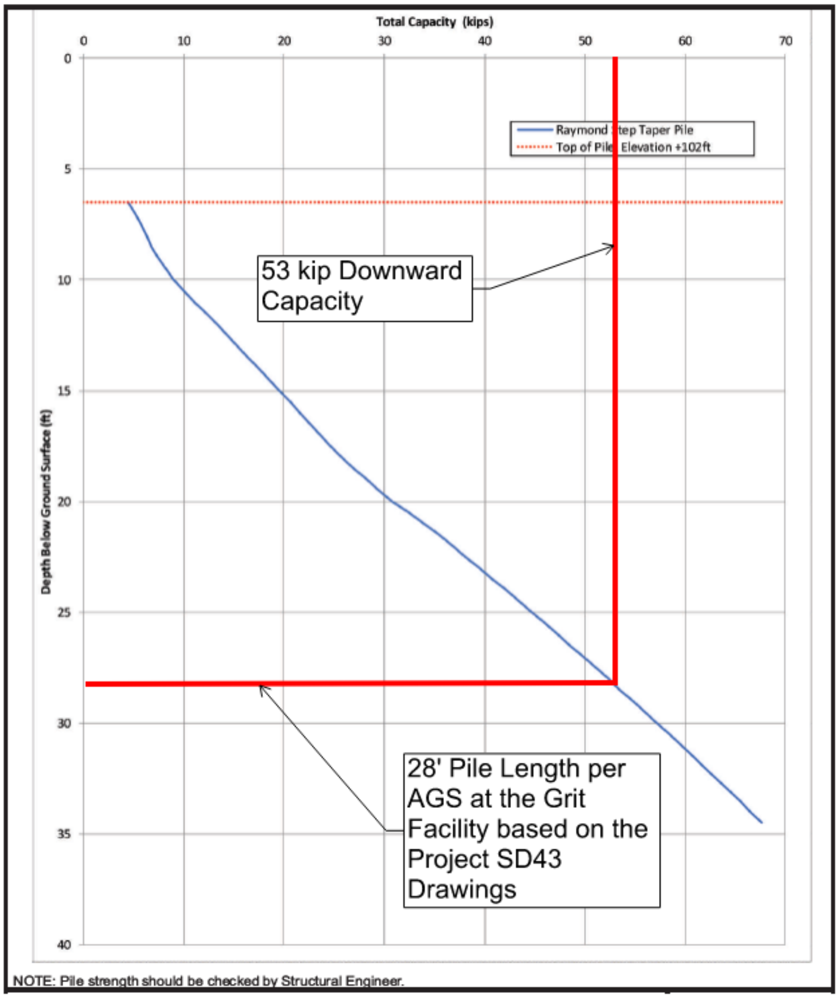


Figure 7-11. Existing Pile Uplift Capacity (AGS 2020a)



NOTE: Pile strength should be checked by Structural Engineer.

Figure 7-12. Existing Pile Downward Capacity (AGS 2020a)

Since the existing piles have sufficient lateral capacity at 37 kips to resist the base shear adjusted by the allowed m-Factors, the retrofit is designed to help resist the overall structure rotation and concentrated forces at existing shear walls. Figure 7-13 shows the locations for new pile caps selected to avoid existing structural elements other than the stair landing at the north.

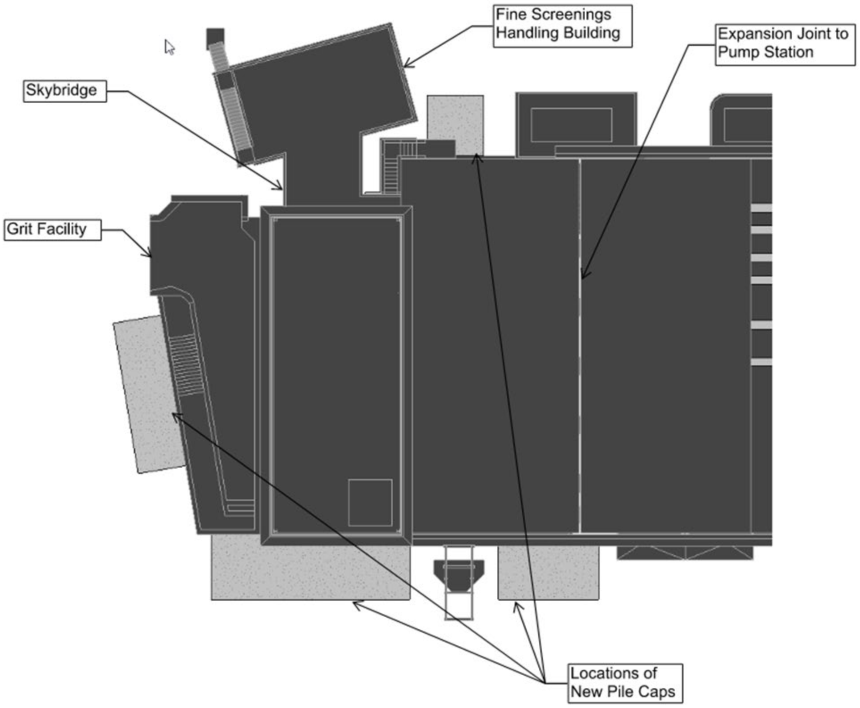


Figure 7-13. Added Pile Cap Locations

To best utilize the available space for the pile caps, larger diameter 24" cast-in-drilled-hole (CIDH) piles are selected similar to those supporting the duct bank to the main substation. Figure 7-14 shows the lateral capacity of the duct bank piles.

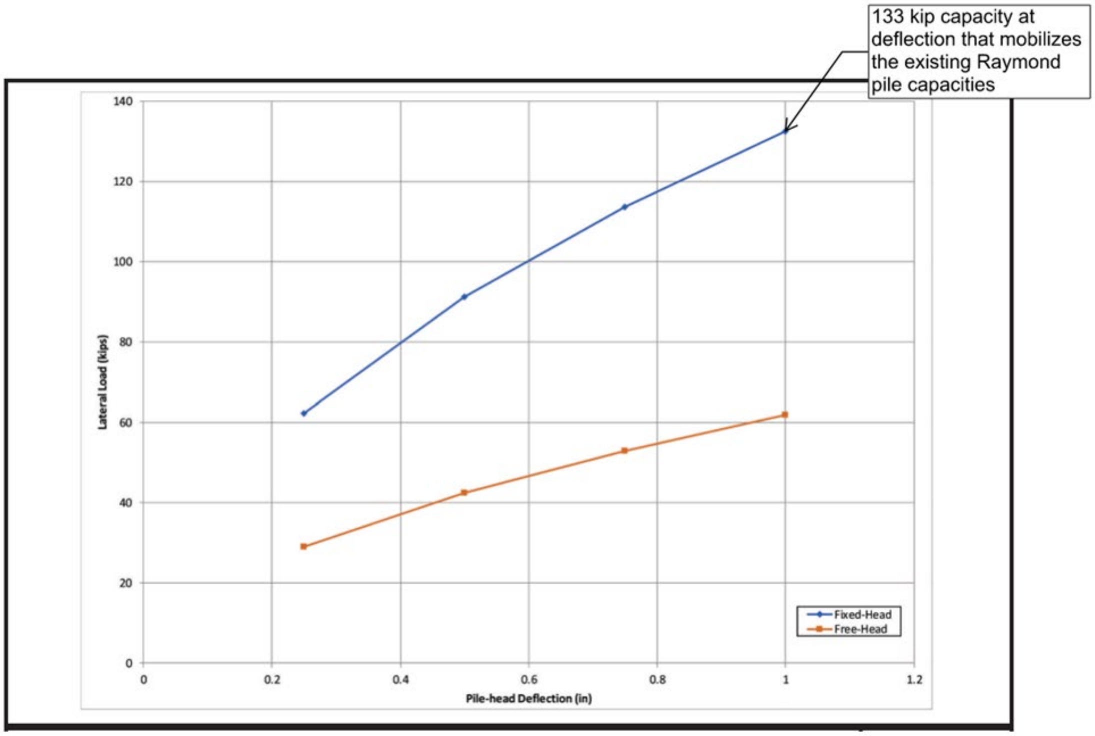
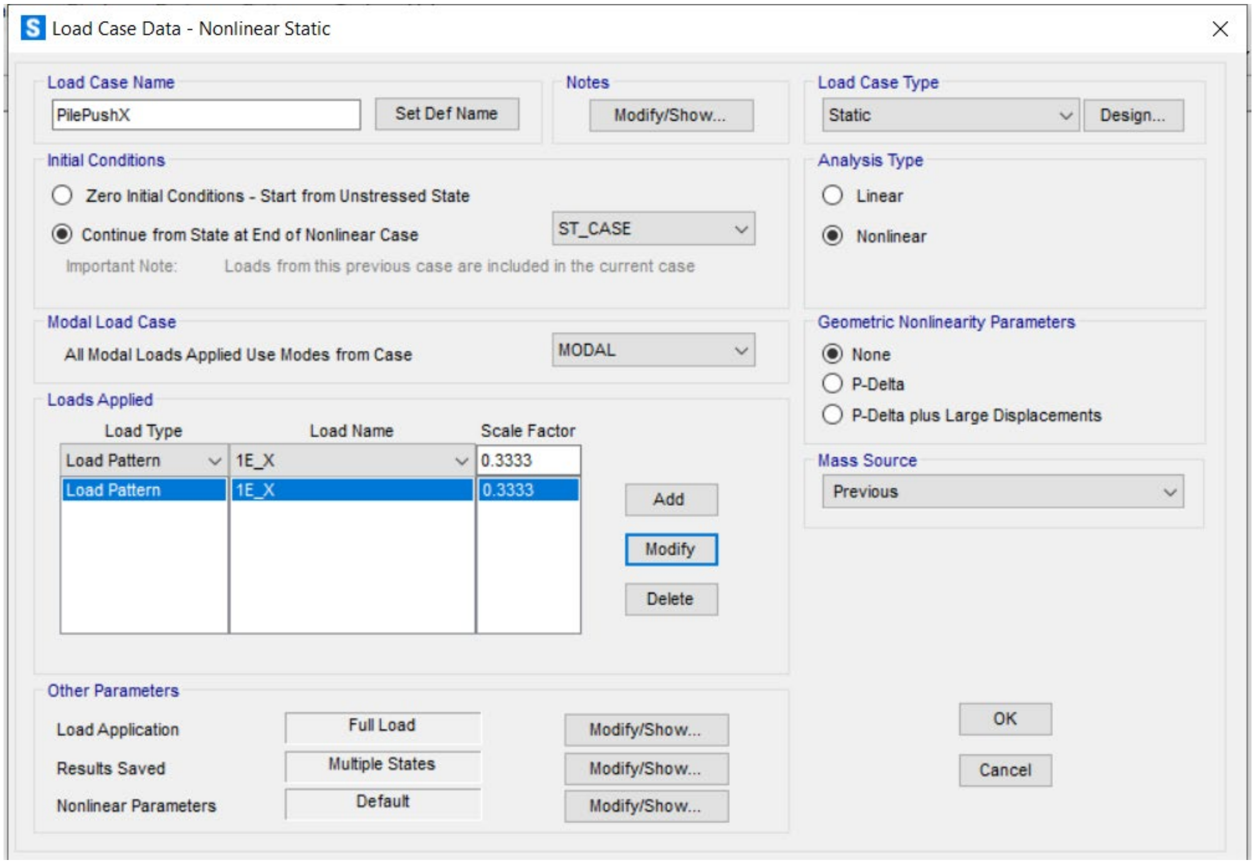


Figure 7-14. Lateral Capacity of 24" CIDH Piles (AGS 2020a)

According to the AASHTO LRFD Bridge Design Specifications (AASHTO 2020), the capacity of the first row of piles is unreduced at a spacing of 5 diameters on center. The second row is reduced to 85 percent capacity, while the remaining rows are reduced to 70 percent capacity. That is substantially improved over 80, 40, and 30 percent, respectively, for a 3-diameter pile spacing. Therefore, the layout for the new piles maintains 10-foot spacings in the direction parallel to wall and resisting structure rotation where added capacity is needed. Sixteen piles contributing 1,729 kips at 10-foot spacing each direction fit within the available pile caps on the south side of the Grit Facility, while 8 piles at 10-foot spacing parallel to the wall and 6-foot spacing perpendicular to the wall can be located on the west. Due to the presence of odor control equipment on the north side of the facility in addition to the Fine Screenings Handling Building, only 4 piles at 10-foot spacing in each direction are added.

To evaluate the effectiveness of adding piles at the locations shown in Figure 7-13, nonlinear static pushover analyses were defined in SAP2000 as indicated in Figure 7-15.



Load Case Name: PilePushX

Load Case Type: Static

Initial Conditions

○ Zero Initial Conditions - Start from Unstressed State

● Continue from State at End of Nonlinear Case: ST_CASE

Important Note: Loads from this previous case are included in the current case

Analysis Type

○ Linear

● Nonlinear

Modal Load Case

All Modal Loads Applied Use Modes from Case: MODAL

Geometric Nonlinearity Parameters

● None

○ P-Delta

○ P-Delta plus Large Displacements

Mass Source: Previous

Loads Applied

Load Type	Load Name	Scale Factor
Load Pattern	1E_X	0.3333
Load Pattern	1E_X	0.3333

Other Parameters

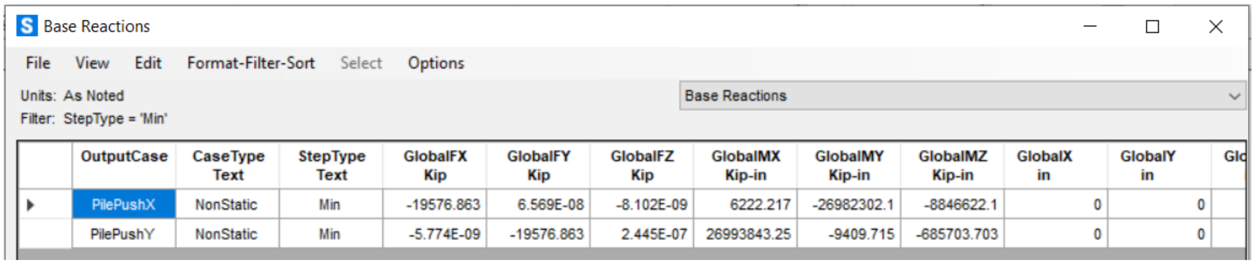
Load Application: Full Load

Results Saved: Multiple States

Nonlinear Parameters: Default

Figure 7-15. East-West Static Pushover Case Definition

The applied load patterns ("1E_X" and "1E_Y") were scaled to orient the combined maximum base shear in each orthogonal reaction as indicated in Figure 7-16 conservatively assuming the north-south and east-west base shears indicated above occur at the same time and represent the peak base shear in any direction from the seismic hazard considered. The scale factor in Figure 7-15 reflects the m-Factor of 3 for Life Safety performance under the Enhanced performance criteria per Section 8.4.5.1.1 of ASCE 41.



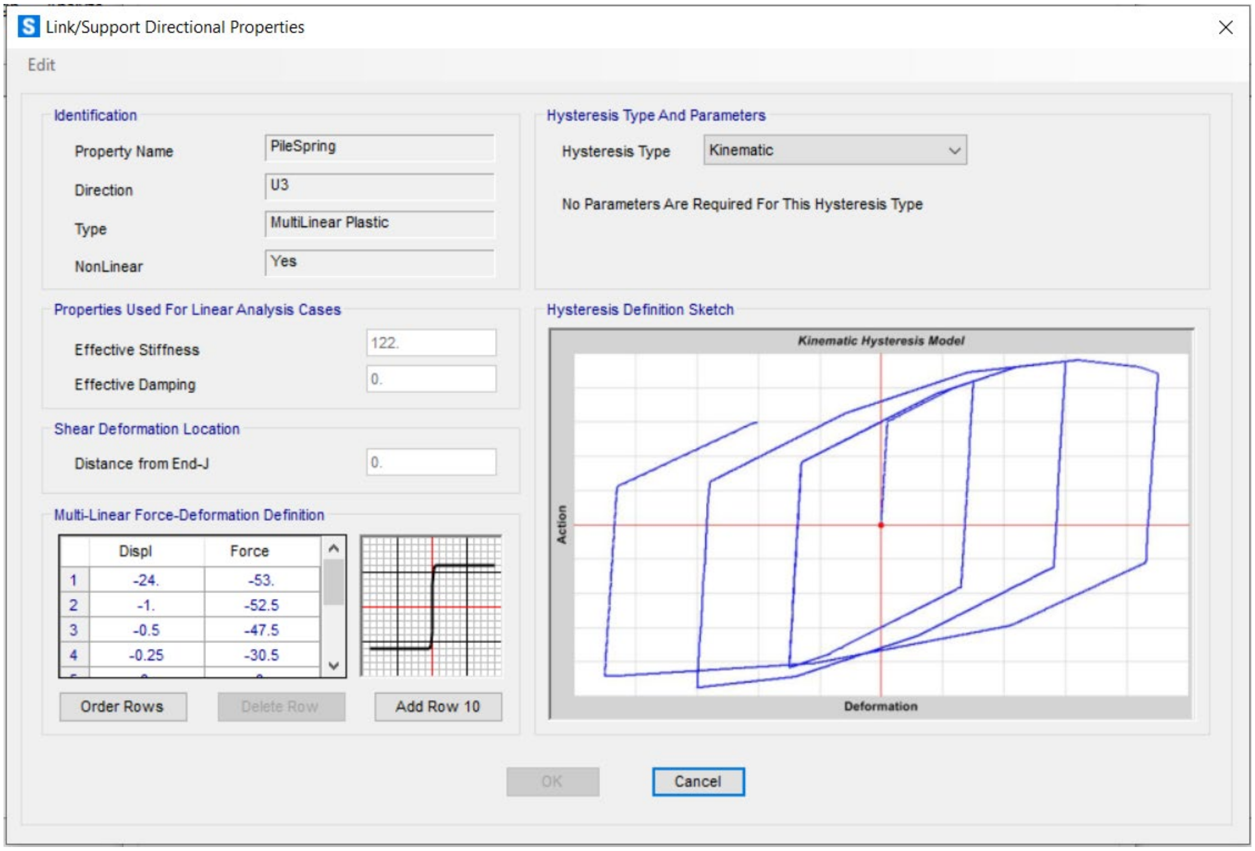
Units: As Noted
Filter: StepType = 'Min'

Base Reactions

OutputCase	CaseType Text	StepType Text	GlobalFX Kip	GlobalFY Kip	GlobalFZ Kip	GlobalMX Kip-in	GlobalMY Kip-in	GlobalMZ Kip-in	GlobalX in	GlobalY in
PilePushX	NonStatic	Min	-19576.863	6.569E-08	-8.102E-09	6222.217	-26982302.1	-8846622.1	0	0
PilePushY	NonStatic	Min	-5.774E-09	-19576.863	2.445E-07	26993843.25	-9409.715	-685703.703	0	0

Figure 7-16. Nonlinear Static Pushover Case Base Shears as Reported by SAP2000

Only the piles were considered to be nonlinear in these analyses, with Figure 7-17 showing the lateral pile spring definition for the existing piles and Figure 7-18 showing the definition for the added piles. A force level of 52.5 kips is included in the spring definition to provide for better convergence for plastic demands. Nonlinear springs were also incorporated for the piles in the vertical direction, so that the piles could not supply resistance above their uplift and downward capacities.



Identification

Property Name: PileSpring
Direction: U3
Type: MultiLinear Plastic
NonLinear: Yes

Hysteresis Type And Parameters

Hysteresis Type: Kinematic

No Parameters Are Required For This Hysteresis Type

Properties Used For Linear Analysis Cases

Effective Stiffness: 122.
Effective Damping: 0.

Shear Deformation Location

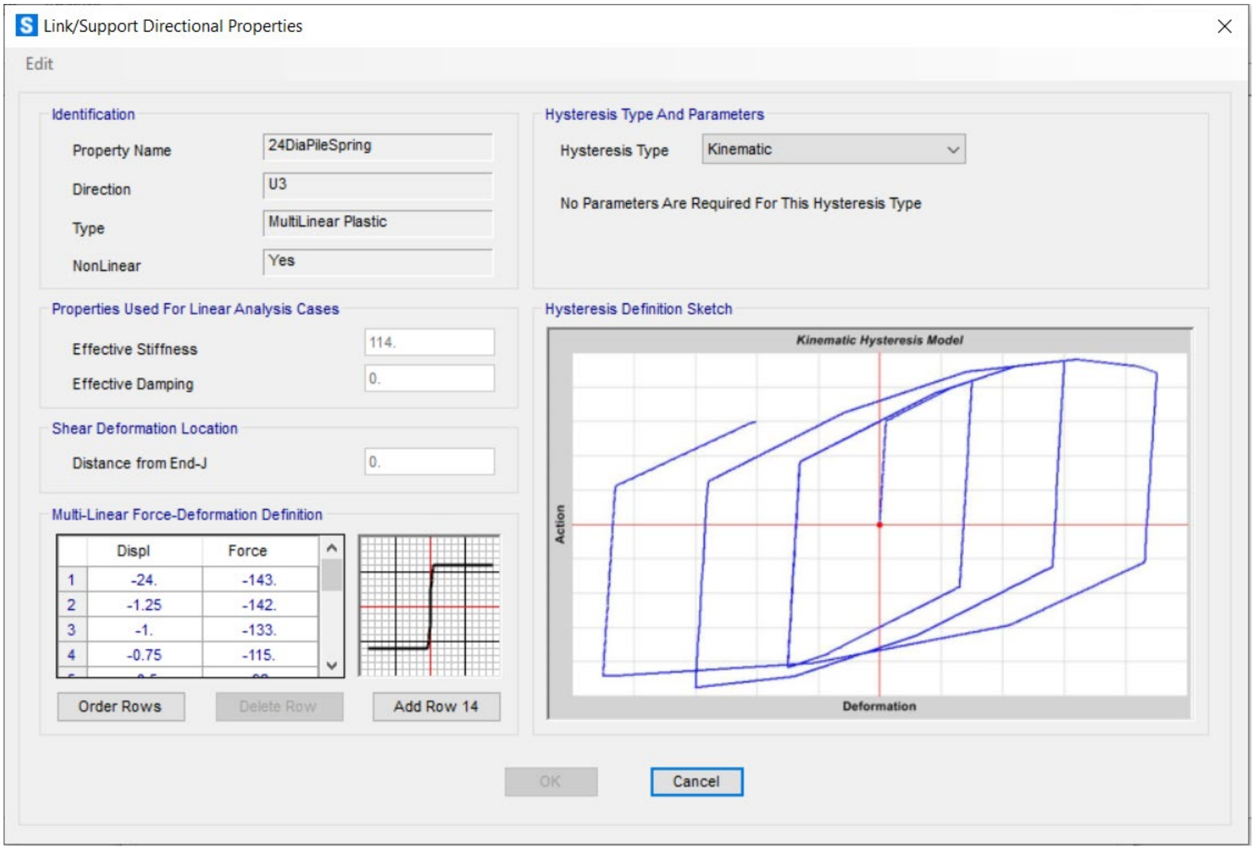
Distance from End-J: 0.

Hysteresis Definition Sketch

Multi-Linear Force-Deformation Definition

	Displ	Force
1	-24.	-53.
2	-1.	-52.5
3	-0.5	-47.5
4	-0.25	-30.5

Figure 7-17. Nonlinear Lateral Force-Displacement Definition for Existing Pile Spring



Identification

Property Name: 24DiaPileSpring

Direction: U3

Type: MultiLinear Plastic

NonLinear: Yes

Hysteresis Type And Parameters

Hysteresis Type: Kinematic

No Parameters Are Required For This Hysteresis Type

Properties Used For Linear Analysis Cases

Effective Stiffness: 114.

Effective Damping: 0.

Shear Deformation Location

Distance from End-J: 0.

Multi-Linear Force-Deformation Definition

	Displ	Force
1	-24.	-143.
2	-1.25	-142.
3	-1.	-133.
4	-0.75	-115.

Hysteresis Definition Sketch

Figure 7-18. Nonlinear Lateral Force-Displacement Definition for Added Pile Spring

Figures 7-19 and 7-20 show the resulting pile forces pushing to the required base shear in the east-west and north-south directions, respectively, for the controlling BSE-2E demands. As seen, the lateral loads on the existing Raymond piles are reduced below 37 kips as required when reduced by an m-Factor of 3 for the Enhanced Performance Objectives. The added 24" CIDH piles are highlighted in red in these figures.

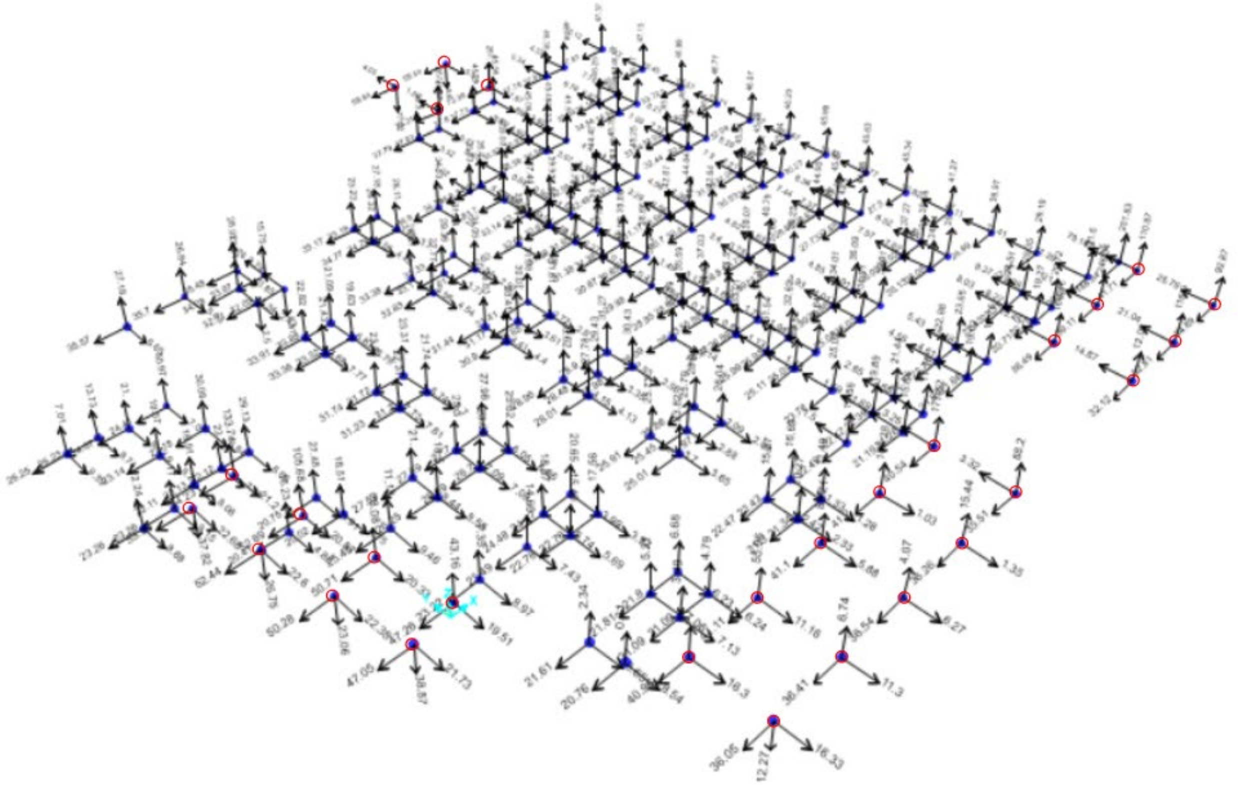


Figure 7-19. Pile Reaction Forces for East-West Base Shear Case

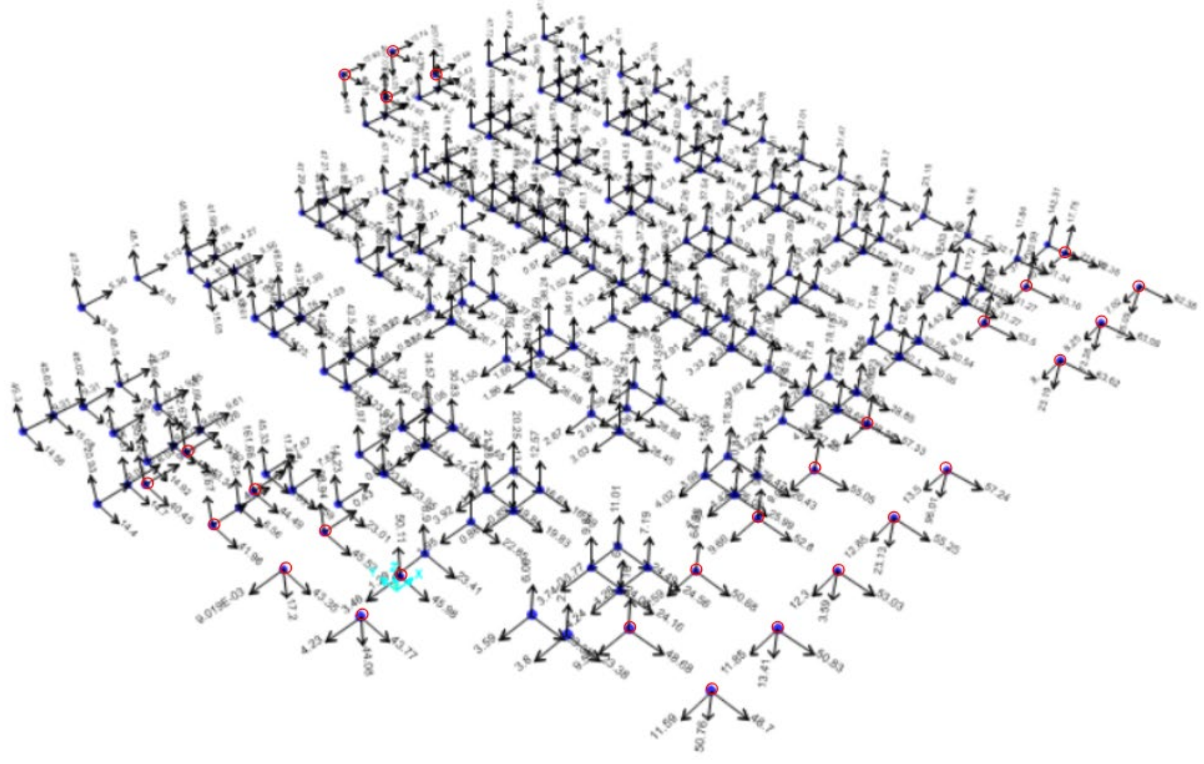


Figure 7-20. Pile Reaction Forces for North-South Base Shear Case

7.7 Seismic Joints

Displacements across the existing expansion joints were evaluated based upon the SAP2000 analysis results. Table 7-2 summarizes the movements anticipated at the BSE-2E seismic hazard level after pile retrofits to the Grit Facility. The large movement at the Grit Facility to Skybridge connection is the unreinforced precast panel deflection over the large opening. Joint movements at the connecting channels are estimated based upon the IPS results with similar displacements expected at the piles and fairly rigid response from the structure. No analyses were performed for those adjoining facilities in this phase of the design.

Table 7-2. Joint Movements at Existing Expansion Joints

Location	Joint Movement			
	Grit Facility End		Opposite End of Joint	
	Perpendicular to Joint	Parallel to Joint	Perpendicular to Joint	Parallel to Joint
Grit Facility to SD43 Sedimentation Basins Influent Channel Top Slab	0.88 inch	0.53 inch	1.00 inch	1.00 inch
Grit Facility to SD43 Sedimentation Basins Influent Channel Bottom Slab	0.50 inch	0.32 inch	0.75 inch	0.75 inch
Grit Facility to SD118 Grit Tanks Influent Channel Top Slab	0.75 inch	0.88 inch	1.00 inch	1.00 inch
Grit Facility to SD118 Grit Tanks Influent Channel Bottom Slab	0.53 inch	0.79 inch	0.75 inch	0.75 inch
Grit Facility to SD170 Skybridge Top Slab	5.78 inch	1.81 inch	1.70 inch	1.34 inch
Grit Facility to SD170 Skybridge Bottom Slab	0.81 inch	0.86 inch	0.40 inch	0.30 inch

Required movement capacities at the expansion joints are conservatively considered as additive of the two ends, as each separate structure may be moving in a different direction at a given time. This exceeds the requirements of ASCE 41 Section 7.2.13.1 as structures on the opposite sides of the joints have not been scoped for analysis. These required movements exceed the 5/8-inch joint width to the Sedimentation Basins Influent Channel, the 1-inch joint width to the Grit Tanks Influent Channel, and the 3/4-inch joint width to the Skybridge.

7.7.1 Joint Retrofits

Seismic joint systems with sufficient capacity to handle the relative movements indicated are available, such as the Sika EMSEAL SJS-Seismic Joint System shown in Figure 7-21. Standard joint widths of 3.5, 8, and 16 inches are available, with allowable relative movement equal to half of the joint width. For the joint at the Skybridge connection to the Grit Facility, installation of a seismic joint requires saw cutting and removal of the end of the concrete bridge structure sufficient for the joint width. Modifications to the conveyor supports straddling the joint are also required to allow for the joint movement.

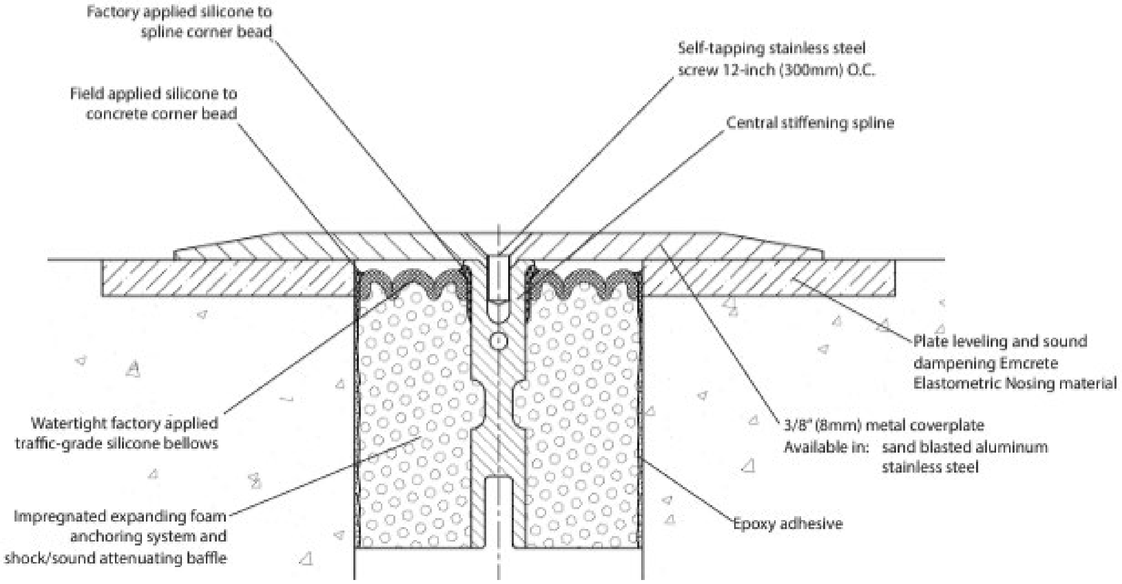


Figure 7-21. Seismic Joint System

At other locations, maintaining channels in service exiting the Grit Facility does not allow for saw cutting of a larger joint width. At these locations, it is recommended that a secondary joint be constructed outside of the existing joint to maintain watertightness if concrete spalling occurs at the existing joint interface due to pounding together of the two adjacent structures. Excavating under the pile-supported influent channels to the downstream facilities is required to install a seismic joint with higher movement capacity below those structures.

At the expansion joint between the Pump Station and the Grit Facility, the walls and roof structures are provided with 4-inch gaps between the concrete elements as shown in Figure 7-22. The sliding plates in these joints lose contact with the adjoining facility after a 3-inch relative movement, which is more than required.

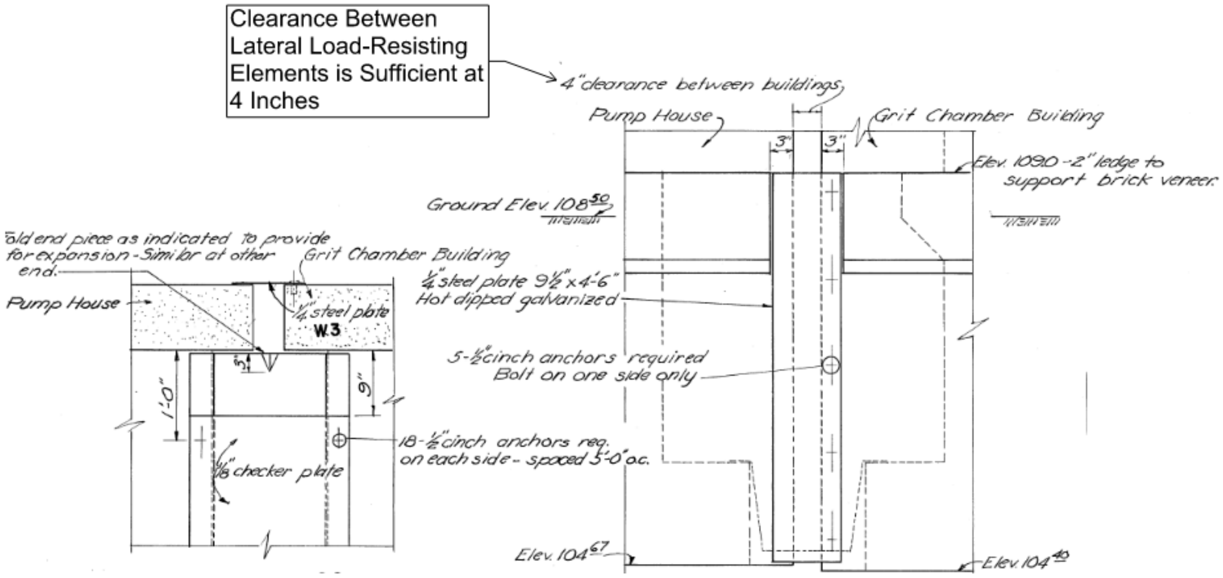
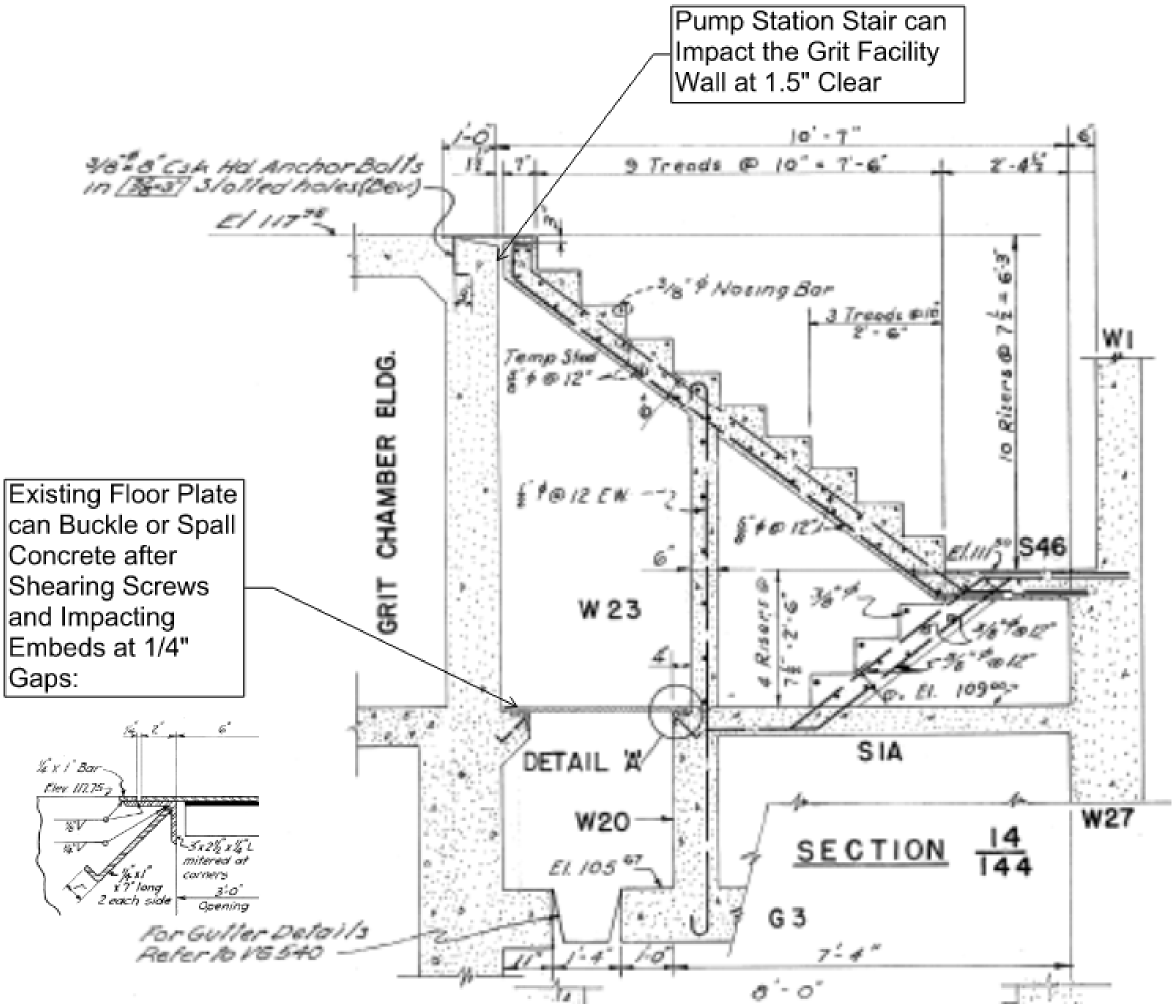


Figure 7-22. Original Expansion Joint Details at the Elevation 109 Ground Floor Level

There are elements that cross this expansion joint, however, that do not have the requisite movement capacity and may experience damage in a seismic event. Figure 7-23 shows two elements that could be impacted at the northwest corner of the Pump Station. Retrofits to provide additional movement capacity will be incorporated as this stair can provide emergency egress from the Grit Facility out a nearby door to the exterior in the Pump Station.


Pump Station Stair can Impact the Grit Facility Wall at 1.5" Clear

Existing Floor Plate can Buckle or Spall Concrete after Shearing Screws and Impacting Embeds at 1/4" Gaps:

Figure 7-23. Elements with Insufficient Capacity for Expansion Joint Displacements

The 48-inch pipes crossing between the Pump Station and Grit Facility are also not provided with sufficient movement capacity to prevent damage. Figure 7-24 shows three of the five pipes crossing the expansion joint and the locations of the flexible couplings at the joint. Typical flexible couplings offer a 1/2" axial movement capacity, which is not sufficient to prevent the ends of the pipes from contacting in an earthquake. When that movement is exceeded at the IPS, the pipe can push into the end plate shown in Figure 7-25 and buckle the steel plate and/or fracture the anchor bolts. That could potentially result in significant leakage from the open channel. Flexible coupling replacement is required to provide sufficient movement capacity to preclude damage.

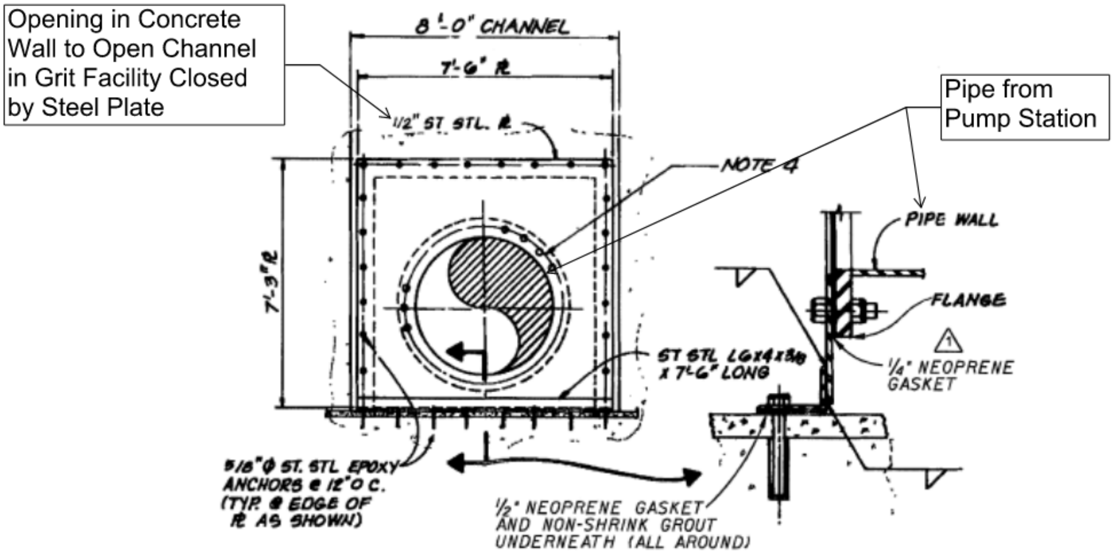


Figure 7-24. Transition from Pipe to Open Channel at Grit Facility

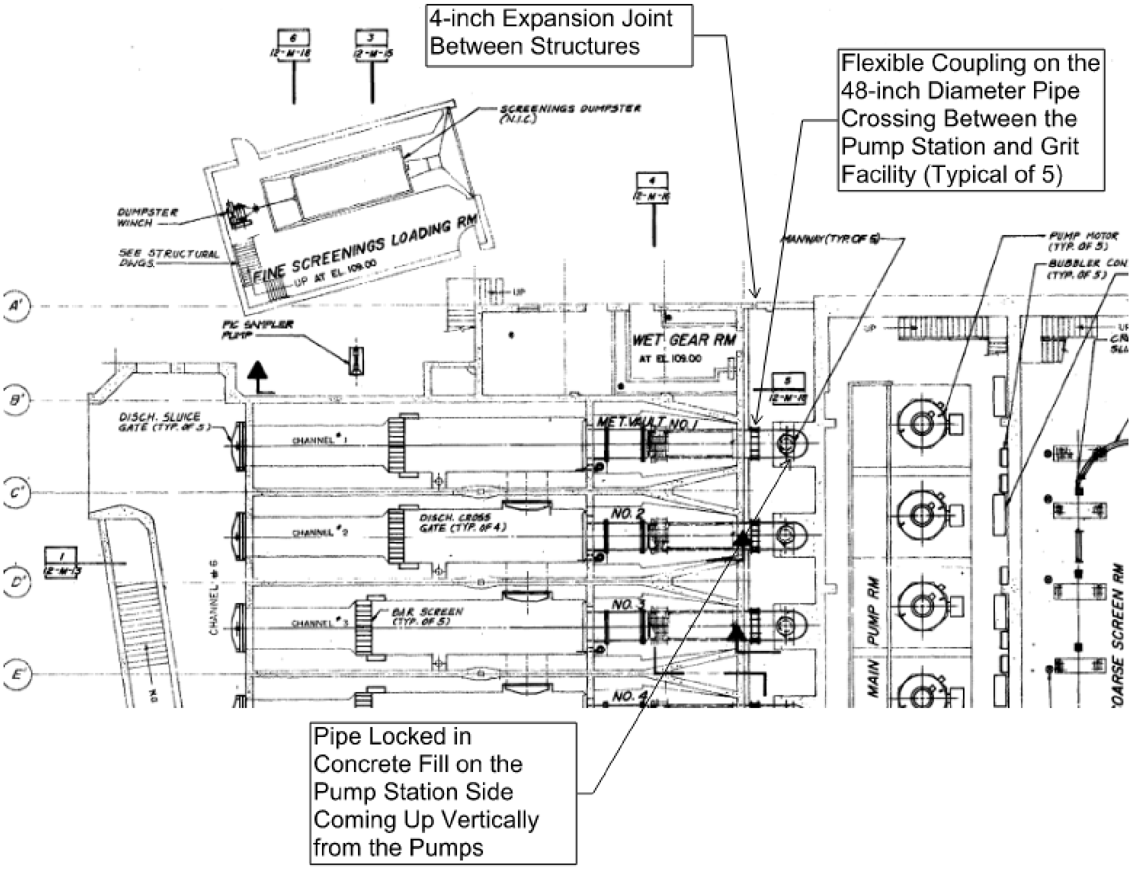


Figure 7-25. Pipes Crossing the Expansion Joint

7.8 Anchorage of Nonstructural Equipment

Anchorage of several nonstructural components is required to meet the performance objectives. The anchorage of the five main pumps in the Pump Station requires improvement to meet the Position Retention requirements under the BSE-1E seismic hazard level and the Hazards Reduced requirements under the BSE-2E. Figure 7-26 shows planned retrofits to the pump anchorages.

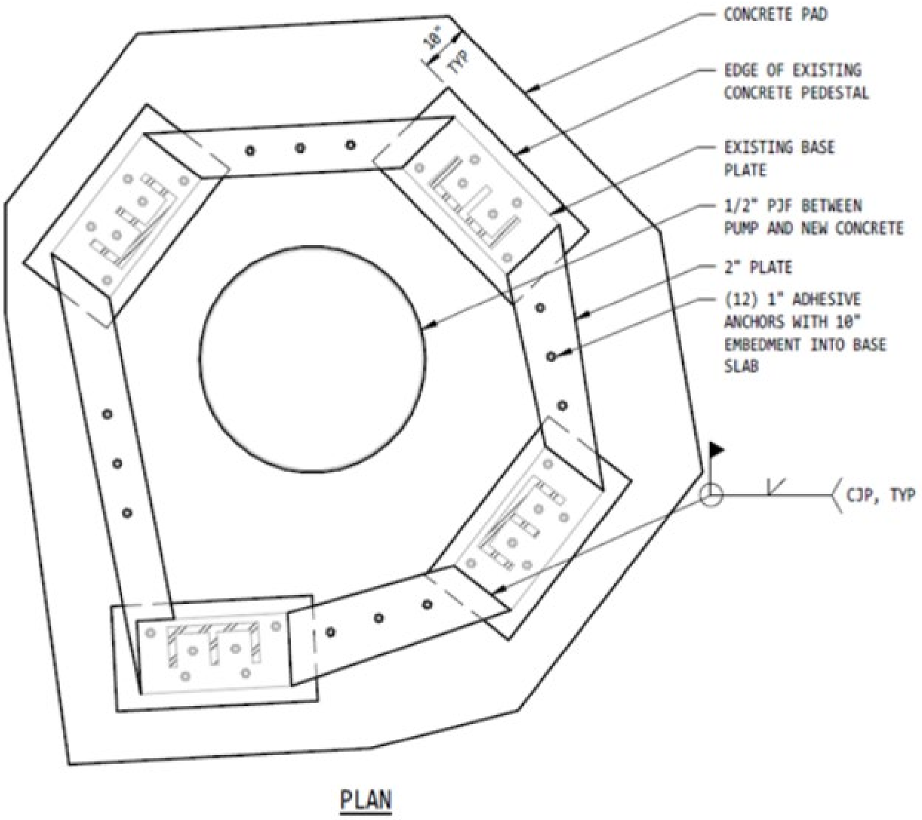


Figure 7-26. Main Pump Anchorage Retrofit

The anchorage of the fine screens in the Grit Facility also requires improvements to meet the performance objectives. Figure 7-27 shows the planned retrofit for the screens to supplement the existing anchorage.

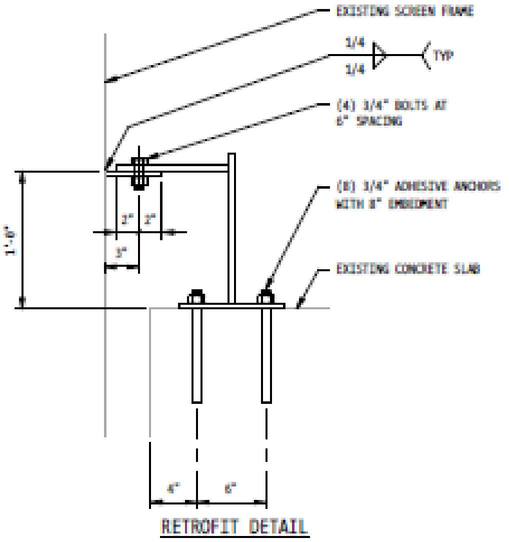


Figure 7-27. Fine Screens Anchorage Retrofit

7.9 Construction Considerations

The construction considerations include the following:

1. The IPS facility must remain operable and occupied during all construction activities.
2. Contractor access to the site is limited to Monday through Friday 8-hour day shifts for scheduling and construction cost determination. Extended hours may be utilized if required to manage the shut down periods allowed due to changed conditions.
3. No shutdowns or reduction in pumping capacity between October 1 and April 30.
4. Two pump trains can be shut down in the dry weather period (three trains remain operable) during May 1 to September 30.
5. Solids conveyor and load out facility needs to remain operable during construction.
6. Exterior Pile Cap and Pile Installation Criteria:
    a. No abnormal noise controls.
    b. No known contaminated soils.
    c. Auger Cast Piles will be used for pile installation.
    d. Buried utilities will need to be relocated.
7. Pumping Channel Retrofit Criteria (strengthening around the cross gates):
    a. Only two channels can be taken down during dry weather; no channels can be shut down during wet weather.
    b. Contractor will need to dewater and clean the channels as part of the work.
8. Concrete wall overlay will require localized removal and replacement of the exterior EIFS.
9. Braced Frame Requirements and Assumptions:
    a. Retrofit at braced frames will require temporary power, controls, piping, and wiring setup so that the conduit can be removed to complete the retrofit.

b. Existing paint will need to be cleaned and prepped for recoated, coating will need to be removed where field welding occurs. Paint system will be a high build epoxy.

c. Braced frames (columns, beams, connections, braces) will need to be repainted where accessible. Coatings that are not accessible due to the presence of brick veneer or precast panels will not be replaced.

d. No known lead paint in the IPS.

e. Braced frame columns and beams need to remain in place during the retrofit. For example the new column base plate would be installed in two pieces, welded together, and overlaid on top of the existing baseplate.

7.10 Cost Estimate Summary

A class 5 cost estimate was completed for the seismic retrofit. The major costs for the retrofit are the piles and associated earthwork followed by the concrete overlays. A detailed cost estimate is included in Attachment 2. The costs assume work would be completed in 2 years from 2024-2025. The estimated range of costs are:

Class 5 Estimate	Low Range	Estimate	High Range
Seismic Retrofit	$5,810,000	$11,610,000	$23,220.00

Cost estimate has been escalated to the mid-point in construction. Add 5% per year, compounded annually for years beyond 2024.

7.11 Construction Schedule

The construction schedule is driven by the construction considerations described in Section 7.7. The key drivers are maintaining operation of the IPS during construction, limiting reduction of pumping capacity to the dry weather season, and providing enough float in the schedule to accommodate unforeseen delays that if not appropriately planned for could result in reduced pumping capacity during the wet weather season. The following is a high-level construction schedule. A more detailed construction schedule is included in Attachment 3. Note that the retrofit work will require the pumping channels to be shut down as part of the work and the proposed construction schedule is for an independent structural retrofit project. In the task 8 implementation plan a construction schedule is presented that combines these outages with the required outages for replacing the pump drives and motors. For example pump trains 3 and 4 would be shut down to replace the drives, motors, and seismically retrofit channels 3 and 4 to meet the limitation of two pump trains maximum shut down during the dry period.

Table 7-3. Construction Sequencing

Retrofit Task	Year 1 January – April	Year 1 May – September	Year 1 October – December	Year 2 January – April	Year 2 May – September	Year 2 October – December
Submittals	X					
Prep for Channel Retrofits	X	X				
Retrofit Channel 1,2,3,4		X				
Retrofit Channel 5,					x	
Braced Frame Retrofit	x	x				
Concrete Wall Overlays		X	X	X	x	
Roof Diaphragm Retrofits				X	x	
Relocate Buried Utilities		X				
Piles and Pile Cap		x	x	X		
Punch List Items and Project Closeout						X

8. Further Numerical Analysis Recommendations

Once the seismic performance criteria are established for the final retrofit design, the maximum ASCE 41-17 DCR remaining with the capacity not modified by an m-Factor can be utilized to calculate new C_1 and C_2 factors. Those calculations then validate that the expected reductions from the tabulated values for typical treatment plant structures are achieved as considered in the preliminary retrofit design.

In order to refine the retrofit design of the pile-supported Grit Facility, the use of nonlinear pushover analyses in the north-south and east-west directions are recommended for the next phase of design. Those analyses would more accurately reflect force redistributions in the structure resulting from addition of 24" CIDH piles to the exterior of the existing structure. The nonlinear pushover analyses would also verify the force redistribution between the lightly reinforced channel training walls and the original walls between the channels to verify that no significant damage would occur.

Nonlinear pushover analyses can also be utilized to develop fragility curves for the structure. Figure 8-1 shows a typical curve (FEMA 2018), which relates the probability of collapse (or other damage levels) to the seismic demand in the form of the acceleration imparted at the fundamental period of vibration of the structure. Fragility curves can be useful in evaluating the overall seismic risks to the wastewater system that includes the influent piping.

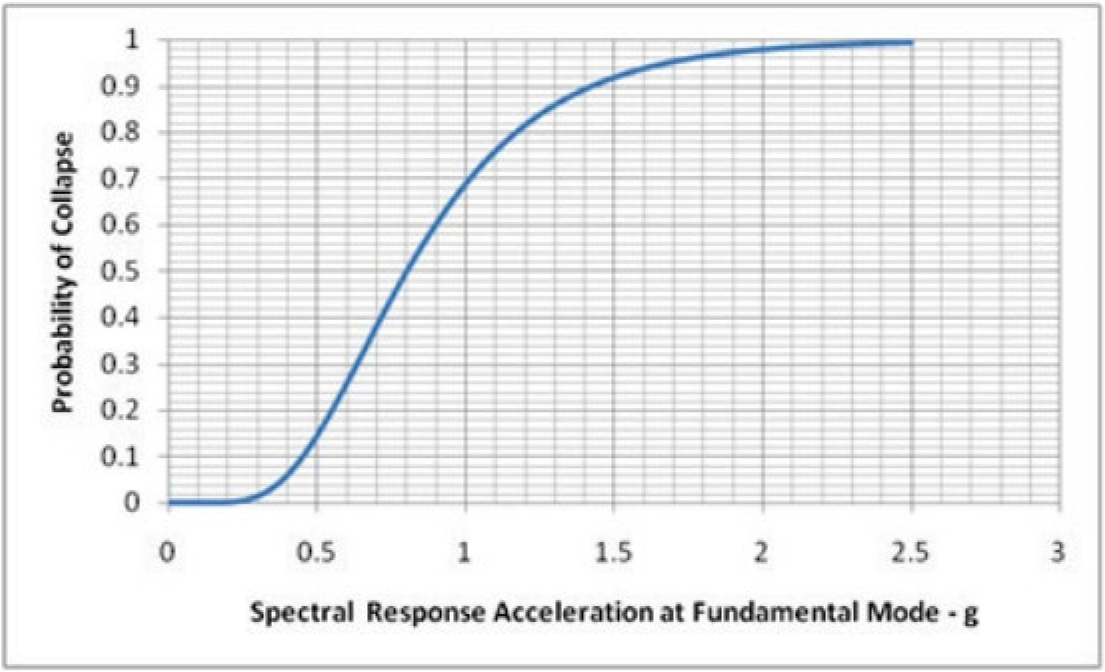


Figure 8-1. Example Fragility Curve

For the other structures comprising the IPS, nonlinear pushover analyses provide less benefit to refining the retrofit design. They can be used to verify the deformation-controlled response of the slabs in the Pump Station, with the impacts of the original one-way bending design on the concrete reinforcing explicitly accounted for in the modeling.

9. Other Alternatives Considered

Other alternatives were considered to mitigate the foundation deficiencies including deep soil-mixing and jet grouting to improve the resistance afforded by the existing piles and passive soil resistance against the existing pile caps. It was concluded that ground improvements would not reduce the loads to the piles enough to mitigate damage without significantly impacting operation of the facility and costing more than the piles shown in the retrofit concept therefore, this concept was dismissed. Consideration of additional shear walls outside of the building and in line with the Grit Facility braced frames was also considered and determined to more be costly than the steel frames retrofit.

10. Conclusions

The proposed seismic retrofit of the IPS will bring the IPS up to an Enhanced performance objective level. As the response spectra are the same for low periods between new and existing building hazard levels, it is recommended that those performance criteria be stated to apply to the BSE-1N and BSE-2N hazard levels in the next phase of design. With calculated structural periods in that range for the IPS, no changes to the preliminary retrofit design are expected. That retrofit consists of the following elements:

1. Strengthening the roof diaphragm around the existing roof openings, including added concrete and steel beams and additional welding of metal roof deck
2. Applying concrete wall overlays on select concrete shear walls
3. Applying fiber-reinforced polymer strengthening systems on select concrete shear walls
4. Applying fiber-reinforced polymer confinement of main Pump Station columns
5. Strengthening of the braced frame connections, columns, and collector beams in the fine screening room
6. Strengthening of the pumping channel walls at the cross gates with FRCM
7. Adding anchors to secure the brick veneer to the supporting concrete wall
8. Adding anchors and strengthening the connections between the precast concrete walls and braced frames in the fine screening room
9. Placing pile caps and piles at the northern, western, and southern sides of the IPS Grit Facility
10. Increasing capacity for movement at expansion joints, including in the five pipes exiting the pumps into the Grit Facility
11. Modifying equipment anchorages and adding bracing

While the IPS could be retrofitted to the lower-level Basic performance objective, only modest increases were required to achieve the Enhanced performance objective. To utilize the Basic performance objective, the following retrofits could be eliminated:

1. Strengthening ground floor beams at the Pump Station with FRP
2. Adding roof beams at the upper roof of the Pump Station
3. FRP retrofit of the pier at Grid Line C3 at the Pump Station

While the added piles at the Grit Facility could be slightly reduced in number for the Basic performance objective, those piles add to the control of displacements at the expansion joints between structures so that eliminating some is not recommended. Overall, the cost reductions down to the Basic performance objective would be expected to be less than $70,000.

The retrofits have been designed to be constructed in an operating facility but will require shutdown of the pumping channels. The work is being proposed to be scheduled so that two channels are shutdown at a time during the dry season. The channel shutdowns are coordinated with pump train shutdowns during the Load Commutated Inverter Drives and pump motor replacements. The work also requires relocation of conduit to allow access to the braced frames as well as relocation of buried conduit to clear the area for new piles and pile caps on the outside of the IPS.