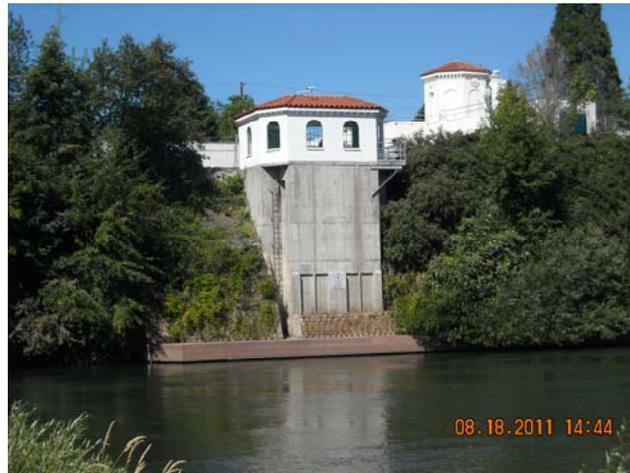


Structural/Seismic Evaluation, Grants Pass Water Treatment Plant, Grants Pass, Oregon

Final June 28, 2012



Contents

Definitions

Executive Summary

Executive Summary	ES-1
-------------------------	------

Chapter 1 General

1.1 Introduction	1-1
1.2 Purpose	1-1
1.2.1 Scope of Work	1-2
1.3 Documents Reviewed and Basis of Information	1-2
1.4 Limitations	1-3

Chapter 2 Structural Wind and Seismic Analysis

2.1 Existing Drawing Review	2-1
2.2 Seismic Geotechnical Evaluation Results	2-1
2.3 Seismic Analysis Results	2-2
2.4 Seismic Evaluation Criteria	2-7
2.5 Seismic Hazards and Potential for Damage	2-7

Chapter 3 Visual Observations

3.1 On-site Inspection/ Review	3-1
3.2 Seismic Vulnerabilities	3-8

Chapter 4 Summarized Recommendations

4.1 Seismic Retrofits	4-1
4.2 Seismic Retrofit Rough Order of Magnitude Cost Estimate	4-3
4.3 Maintenance Retrofits	4-5
4.4 New Plant Alternative	4-5

Attachments

Attachment A	Building Performance Data
Attachment B	Original Drawings & Field Photographs

Definitions

Accelerations-Sensitive Nonstructural Component: A nonstructural component that is sensitive to, and subject to, damage from inertial loading.

Beam: A structural member whose primary function is to carry loads transverse to its longitudinal axis.

Braced Frame: A vertical lateral-force-resisting element consisting of vertical, horizontal, and diagonal components joined by concentric or eccentric connections.

Building Occupancy: The purpose for which a building, or part thereof, is used or intended to be used, designated in accordance with the applicable building code.

Chord: See Diaphragm Chord.

Cross Tie: A component that spans the width of the diaphragm and delivers out-of plane wall forces over the full depth of the diaphragm.

Diaphragm: A horizontal (or nearly horizontal) structural element used to transfer inertial lateral forces to vertical elements of the lateral-force-resisting system.

Diaphragm Chord: A boundary component perpendicular to the applied load that is provided to resist tension or compression due to the diaphragm moment.

Diaphragm Collector: A component parallel to the applied load that is provided to transfer lateral forces in the diaphragm to vertical elements of the lateral-force-resisting system.

Diaphragm Tie: A component parallel to the applied load that is provided to transfer wall anchorage or diaphragm inertial forces within or across the diaphragm. Also called diaphragm strut.

Element: An assembly of structural components that act together in resisting forces, including gravity frames, moment-resisting frames, braced frames, shear walls, and diaphragms.

In-Plane Wall: See Shear Wall.

Landslide: A down-slope mass movement of earth resulting from any cause.

Lateral-Force-Resisting System: Those elements of structure that provide its basic lateral strength and stiffness.

Liquefaction: An earthquake-induced process in which saturated, loose, granular soils lose shear strength and liquefy as a result of increase in porewater pressure during earthquake shaking.

Maximum Considered Earthquake (MCE): An extreme earthquake hazard level defined by MCE maps which are based on combination of mean 2%/50-year probabilistic spectra and 150% of median deterministic spectra at a given site.

Out-of-Plane Wall: A wall that resists lateral forces applied normal to its plane.

Pilaster: A rectangular column with a capital and base projecting only slightly from the wall as an ornamental motif.

Seismic Evaluation: An approved process or methodology of evaluating deficiencies in a building which prevent the building from achieving a selected Rehabilitation Objective.

Shear Wall: A wall that resists lateral forces applied parallel with its plane. Also known as an in-plane wall.

Sheathing: Lumber or panel products that are attached to parallel framing members, typically forming wall, floor, ceiling, or roof surfaces.

Executive Summary

This report focuses on a seismic and structural review of the Grants Pass Water Treatment Plant (WTP). A review of geotechnical studies conducted at the plant site show that ground-shaking and slope stability along the Rogue River bluff are the two most-significant seismic geotechnical risks. A review of the construction documents of the plant shows that, overall, the structures appear to have been designed and detailed prior to consideration of seismic loads. Due to the lack of seismic design or inadequate design for seismic loads to prevent significant damage or potential collapse in the event of strong ground motion earthquakes, the facility is judged to have a high seismic risk with portions susceptible to collapse in a code-level seismic event or strong earthquake. In general, major structural elements and connections of the lateral-load carrying systems were not designed to be reasonable in terms of current code requirements. They were also not designed for expected wind load performance for this type of structure.

MWH has prepared a list of the potential retrofits for deficiencies identified during this review. This list is presented in Section 4. In addition to compiling this list, MWH has estimated a total project “rough order of magnitude” cost for these improvements. The project costs are itemized in Section 4 and total approximately \$8,500,000.

In addition to estimating project costs for improvements, MWH developed a high-level estimate for the construction of a new treatment facility for the City. These project costs are also itemized in Section 4 and totals \$49,000,000. While this number is significantly higher than the cost of improvements for the seismic deficiencies, additional benefits would be gained by the City such as more-advanced treatment processes than are currently being used today, increased energy efficiency, and room for expansion.

In MWH’s opinion, the valuable water treatment plant operations associated with this facility are practical provided a major earthquake, with strong ground motion, is not experienced. In the event that a major earthquake with strong ground motion did occur, loss of facilities would likely be incurred. The seismic retrofit of the facility will enhance seismic performance of the structure substantially, and is recommended if this plant will continue to be used into the future.

Chapter 1 General

1.1 Introduction

This report presents a structural and seismic engineering review of the Grants Pass Water Treatment Plant (WTP) in Grants Pass, Oregon and “rough order of magnitude” estimated costs for recommended improvements and for construction of a new plant. A visual inspection of the WTP was conducted on August 18, 2011. The WTP is a unique reinforced concrete structure with some wood-framed roofs, steel-framed roofs, concrete block buildings, and multiple structural elements and systems. The existing 2-story tall, slender, reinforced concrete and block shear wall buildings were constructed in multiple phases over many years. The original headhouse, filters, flocculation and sedimentation basins were constructed in 1931. Additional filters, flocculation and sedimentation basins and a chemical feed area were constructed in 1950. A chemical feed area for the original filters and chlorine storage building were added in 1961. In 1980, a new reinforced concrete raw water intake adjacent to the Rogue River was added along with a third sedimentation basin, clearwell and three filters. A new finished water pump room and wastewater solids basin were also added. These additions are presented graphically in **Figure 1**.

1.2 Purpose

The primary purpose of this project was to examine the ability of the WTP to resist seismic earthquake loads. The project is also intended to be informative in preparation of a long-term Capital Improvement Plan (CIP) in conjunction with the ongoing facility plans. The seismic forces considered for the site are governed by the International Building Code (IBC) 2009 edition which defines ground accelerations of 75 percent (%) of gravity for short period structures, and 38% of gravity for long period structures which equates to a 2,475 year return period earthquake per the current Oregon Structural Specialty Code (OSSC) 2007 edition. The OSSC is based upon the 2006 IBC. Additionally, the intent of this report is to assess the WTP's current structural condition, to identify observable deficiencies, and to discuss potential benefits as well as concerns produced by retrofitting recognized deficiencies. Also, we began the process of recording the state of each structure to address areas of concern and identify locations of potential future relevance. Thus, this report focuses on issues of seismic resistance and discusses material degradation likely to impact this WTP in the future.

1.2.1 Scope of Work

MWH was authorized on July 13, 2011 to perform this structural and seismic engineering review. The scope of work for this project included:

Task 1 – Assemble Project Information

- Provide the City of Grants Pass (City) with a written list of all data required to perform the engineering review including all available structural drawings and reports.
- Interface with appropriate personnel during the project to address the needs of City personnel.

Task 2 – Structural and Seismic Evaluation

- Utilizing primarily experience and judgment, evaluate the as-designed structures for IBC 2009 and American Society of Civil Engineers (ASCE) 7-05 seismic forces per the IBC and OSSC criteria for a 2,475 year return period earthquake of 75% of gravity lateral loading for these short period structures.

Task 3 – Visual Observations

- Visually inspect the structures in their current state. The visual inspection is to establish the condition of the structural elements, recognize any obvious deficiencies, and confirm the structures' conformance with the as-built drawings.

Task 4 – Document Analysis and Observations in this Report

- Create a draft report of the seismic evaluation and observations for City review.
- Upon receipt of City comments on the reviewed draft report, incorporate the comments and issue a Final Report.
- The report will also include recommendations for the following:
 - Any immediate structural life-safety issues recognized.
 - Retrofit or replacement measures needed for the facility to achieve an acceptable level of wind and seismic force resistance.

1.3 Documents Reviewed and Basis of Information

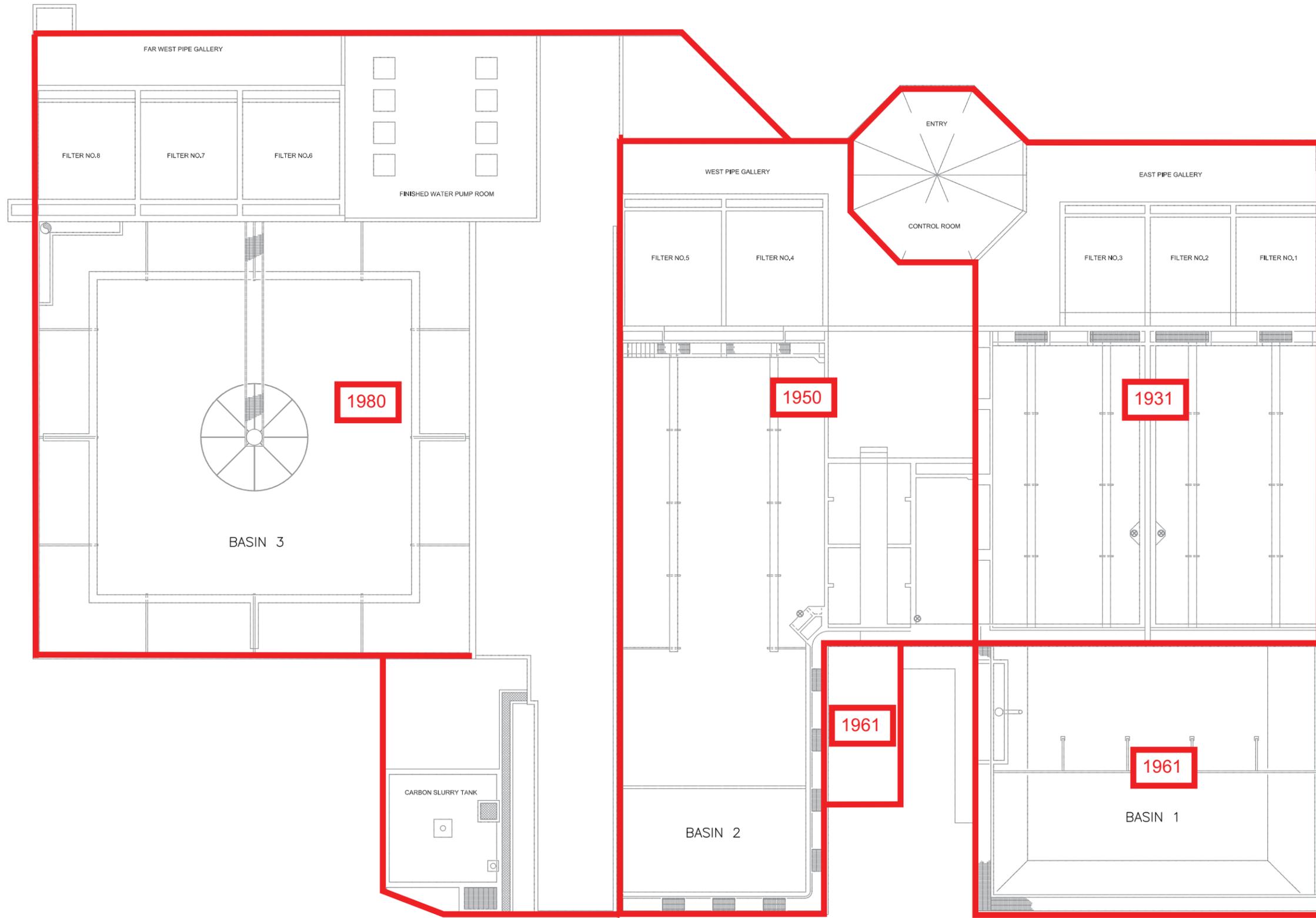
A review of existing documents related to the WTP was conducted. These documents consisted of geotechnical reports and original structural drawings and are listed below.

- Geotechnical reports
 - Geotechnical Engineering Consultation, West Yost & Associates, 2005
 - Site Visit Report, Geotechnical & Environmental Associates, 2006
 - Geotechnical Investigation Report The Galli Group, 2009
- The original structural drawings and reports by
 - Water Filtration Plant, Baar Cunningham, 1931
 - Addition to Filter Plant, John Cunningham, 1950
 - Water Treatment Plant 1961 Expansion, Cornell, Howland, Hayes and Merryfield, 1961
 - Water System Improvements, CH2MHill, 1980
 - Water Treatment Intake Improvements, West Yost & Associates, 2007
 - Structural and Seismic Upgrades for the Proposed Basin Improvements, MWH, 2009

1.4 Limitations

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable engineers practicing in the structural field in this or similar localities at this time. No other warranty, express or implied, is made as to the professional advice included in this report. Although The City of Grants Pass may share this report with other parties, this report was prepared for the City to be used solely for its evaluation of the subject buildings. The report was not prepared for use by other parties and may not contain sufficient information for the purposes of other parties or other uses. The original design engineers remain the engineer of record for their project work and nothing in the report shall be construed as limiting this status. Our statements of opinion are as follows in the body of this report and as stated above in the Executive Summary.

\$\$\$\$\$DATE\$\$\$\$\$
\$\$\$\$\$USER\$\$\$\$\$



\$\$\$\$\$FILENAME\$\$\$\$\$

REV	DATE	BY	DESCRIPTION

SCALE
1" = 10'

WARNING
0 ? 1
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

DESIGNED ARL
DRAWN ARL
CHECKED PK

SUBMITTED BY
(PROJECT MANAGER'S NAME) LICENSE NO. DATE
(COMPANY OFFICER'S NAME) LICENSE NO. DATE



WATER TREATMENT PLANT
GENERAL ARRANGEMENT

SHEET
Fig 1
Job No. 1530683

Chapter 2 Structural/Seismic Evaluation

2.1 Existing Drawing Review

The documents reviewed for this evaluation included copies of the original structural drawings of the existing Grants Pass WTP and reports as listed in Chapter 1. The WTP seismic and structural evaluation was completed by MWH and is discussed herein.

Multiple connected structures on the site are commonly referred to as the City of Grants Pass Water Treatment Plant. The main structure is an assembly of multiple interconnected structures constructed over many years. The original reinforced concrete sedimentation basins, filters, and clearwell building (with headhouse and tower) were constructed in 1931. These were followed with added filters and sedimentation basins in 1950. The plant was expanded again in 1961 with a chemical feed area, motor control, chlorine storage, new southeast basin, and piping improvements. In 1980, a new reinforced concrete raw water intake from the Rogue River was added along with the 80-foot square sedimentation basin, new filters and northwest clearwell as well as other improvements. Many common walls interconnect adjacent elements of this facility and contribute to lateral stability of the structures. The availability of the existing drawings allows for significant understanding of many of the lateral load resisting elements.

The Grants Pass WTP Technical Memorandum titled “Structural and Seismic Upgrades for the Proposed Basin Improvements” (MWH - May 29, 2009) was also reviewed. This memorandum described existing plant conditions at the older sedimentation basins and baffle walls along with proposed seismic/structural improvements to these areas.

2.2 Seismic Geotechnical Evaluation Results

Geotechnical reports listed in Chapter 1 were reviewed. Significant findings from the geotechnical data review are:

1. Ground shaking is the primary hazard to the facility. The maximum considered earthquake for the site (which is the IBC code level design event) has an acceleration $S_s=0.754g$ which is substantial for the short period acceleration that will affect all of the site structures. The S_1 acceleration of $0.379g$, which is the longer period seismic wave, will have an additional impact on the structures by resonating with the contained water in the basins.
2. Landslide and slope stability are recognized as significant risks along the Rogue River bluff. The bluff under the adjacent bridge and along the area where the

treatment plant exists has been somewhat stabilized in the past 10 to 12 years by various means. Significant erosion and/or sloughing of the bank occurred in 1995 and during the flood of January 1, 1997. In 2006, a portion of the bank failed and a head scarp was documented. In 2001, another area of the slope failed during installation of soil nails. This bank is inherently unstable. During a significant seismic event, a slope failure presents a high vulnerability that could significantly damage and/or collapse portions of the facility adjacent to the slope depending on the depth of the failure plane.

3. Liquefaction is not a significant concern as the site is underlain by sandy gravel and cobbles to a depth of 89 feet where bedrock exists, and no groundwater is present in the soils. Much of the sandy gravel and cobbles are described as cemented.
4. Ground surface fault rupture is not a concern as no mapped faults run through the site.

2.3 Potential for Damage Due to a Seismic Event

In the event of a major earthquake, the current structural risk to the City of Grants Pass WTP is assessed to be high. Overall, the structures are assessed to be designed and detailed prior to consideration of seismic loads, per review of the design and construction documents. Due to the lack of seismic design or inadequate design for seismic loads to prevent significant damage or potential collapse in the event of strong ground motion earthquakes, the facility is judged to have a high seismic risk with portions susceptible to collapse in a strong earthquake. In general, major structural elements and connections of the lateral-load carrying system were not designed to be reasonable in terms of current code requirements. While they met codes that were in effect at the time of construction these codes were not very restrictive and not much was known then about seismic resistance, design and construction (and expected wind load performance for this type of structure).

The following areas of concern have been identified at the strong ground motion earthquake level. These deficiencies may lead to major structural damage and partial collapse of the structure: The seismic evaluation of the original drawings was able to determine multiple areas of seismic vulnerability. Beginning with the 1931 drawings, the following vulnerabilities are recognized:

1. The original round reinforced concrete mixing tank has a 12-inch thick wall and only a single curtain of reinforcing steel with 8 and 4 inches of concrete cover inside and outside, respectively.

2. The many reinforced concrete columns supporting the upper levels of the facility have tie spacing at up to two feet on-center and do not have cross ties. Columns such as these have limited confinement of their concrete cores due to the minimal #4 tie bar spacing that does not meet modern building codes. This lack of confinement can lead to spalling of the columns, loss of concrete coverage, and diminished column capacity with the lack of cross ties necessary to confine the concrete. The concrete within the ties can crack, spall, and burst as well, losing capacity during a strong ground motion earthquake. Loss of vertical capacity of the columns can cause redistribution of loading and potential partial collapse in the most vulnerable areas.
3. The sedimentation basin walls are thin and lightly reinforced and susceptible to significant hydrodynamic forces during a strong earthquake. Depending upon the acceleration of a strong ground motion earthquake, the results can likely be wall failure in bending. The acceleration of the earthquake can fail the walls in bending from the dynamic force of the water. The bending cracks, yielding of reinforcing and crushing of concrete at the base of the walls will result in permanent deformations of the walls and cause leaks through the cracks. Retrofitting the basin walls is possible and was outlined in the Grants Pass WTP Technical Memorandum titled "Structural and Seismic Upgrades for the Proposed Basin Improvements" (MWH - May 29, 2009). The retrofit described in the TM entailed the addition of 6-inch interior-face reinforced concrete walls to the existing walls.
4. The straight wood sheathed roof diaphragms have a very limited diaphragm shear capacity of only 100 pounds per lineal foot. The heavy clay tile roof and the heavy concrete walls greatly increase seismic demands on this straight sheathed diaphragm. The diaphragm is expected to shear during a strong ground motion earthquake leading to loss of out-of-plane wall anchorage and partial collapse of the roofs.
5. The gap between the straight wood sheathing and the top chord of the wood truss at the filter building does not allow lateral wall out of plane shear transfer from the truss top chord into the straight sheathed wood diaphragm. The straight sheathed wood diaphragm is also inadequate to transfer the high seismic load demands. The result of these multiple vulnerabilities is that the as-built roof structure is a collapse hazard in a strong ground motion earthquake.
6. The lack of adequate anchorage at the top of the pilasters to the roof trusses and all perimeter wood framing for seismic out-of-plane wall loads to resist the walls inertial forces falling away from the building creates a condition where the walls and roof can separate during a strong earthquake. This detachment can lead to

collapse potential of the roof in the filter areas. Attachment B sheet B-1 has the detail reproduced from the original 1930's and 1950's drawings that depicts this connection. The assembly of these elements without adequate connections was not recognized or designed for in the original design. Retrofit of the roof members and pilaster/wall connection for these large seismic loads is recommended to prevent excessive deformations of the roof relative to the walls resulting in collapse hazards during strong ground motion earthquakes.

7. The basement level clearwell was documented by MWH in the March 3, 2011 inspection as having multiple reinforced concrete beams with exposed bottom reinforcing steel. This bottom reinforcing steel is in various stages of rusting and corrosion. Upon performing structural calculations on these beams, it became apparent that the only loads these beams can support without cracking and yielding are vertical gravity code level loads. The addition of seismic demand is recognized as beyond the capacity of these beams as originally built. Recognizing the reinforced concrete beams do not have seismic code level capacity characterizes the structural horizontal supports as vulnerable to yielding and significant deformation during a code level seismic event. MWH does, however, believe the repair of these members should be completed to bring these critically important beams to a vertical load carrying capacity at service levels.

With the 1950 drawings, the following vulnerabilities are recognized. Many of these are identical to the 1930 original construction items as this was a duplication of much of the original design in adding more filters and basins.

1. The many reinforced concrete columns supporting the upper levels of the facility have tie spacing at up to 12-inches on center. Columns such as these have limited confinement of their concrete cores due to the minimal #4 tie bar spacing that does not meet modern building codes. This lack of confinement can lead to spalling of the columns, loss of concrete coverage, and diminished column capacity with the lack of cross ties necessary to confine the concrete. The concrete within the ties can crack, spall, and burst as well, losing capacity during a strong ground motion earthquake. Loss of vertical capacity of the columns can cause re-distribution of loading and potential partial collapse in the most vulnerable areas.
2. The sedimentation basin walls are thin and lightly reinforced and susceptible to significant hydrodynamic forces during a strong earthquake. Depending upon the acceleration of the strong ground motion earthquake, the results can likely be wall failure in bending. The acceleration of the earthquake can fail the walls in bending from the dynamic force of the water. The bending cracks, yielding of

reinforcing and crushing of concrete at the base of the walls will result in permanent deformations of the walls and cause leaks through the cracks. Retrofitting the basin walls is possible and was outlined in the Grants Pass WTP Technical Memorandum titled "Structural and Seismic Upgrades for the Proposed Basin Improvements" (MWH - May 29, 2009). The retrofit described in the TM entailed the addition of 6-inch interior face reinforced concrete walls to the existing walls.

3. The gap between the straight wood sheathing and the top chord of the wood truss at the filter building does not allow lateral wall out of plane shear transfer from the truss top chord into the straight sheathed wood diaphragm. The straight sheathed wood diaphragm is also inadequate to transfer the high seismic load demands. The result of these multiple vulnerabilities is that the as-built roof structure is a collapse hazard in a strong ground motion earthquake.
4. The straight wood sheathed roof diaphragms have a very limited diaphragm shear capacity of only 100 pounds per lineal foot. The heavy clay tile roof and the heavy concrete walls greatly increase seismic demands on this straight sheathed diaphragm. The diaphragm is expected to shear during a strong ground motion earthquake leading to loss of out-of-plane wall anchorage and partial collapse of the roofs.
5. The lack of adequate anchorage at the top of the pilasters to the roof trusses and all perimeter roof framing for seismic out-of-plane wall loads to resist the walls inertial forces falling away from the building creates a condition where the walls and roof can separate during a strong earthquake. This detachment can lead to collapse potential of the roof in the filter areas. Attachment B sheet B-1 has the detail reproduced from the original 1930's and 1950's drawings that depicts this connection. The assembly of these elements without adequate connections was not recognized or designed for in the original design. Retrofit of the roof members and pilaster/wall connection for these large seismic loads is recommended to prevent excessive deformations of the roof relative to the walls resulting in collapse hazards during strong ground motion earthquakes.

With the 1961 plant expansion drawings, the following vulnerabilities are recognized, many of which are identical to the 1930 and 1950 original construction items. This commonality and interdependency of the structures is due to the existing structures being interconnected with common walls. The vulnerabilities are as follows:

1. The east sedimentation basin has a common wall with the existing basin wall that forms the north wall of this basin. The hydrodynamic forces both pushing and pulling on this interior "divider" wall were never designed for or retrofitted for

these loads. The thin and lightly reinforced sedimentation basin walls are susceptible to significant hydrodynamic forces during a strong earthquake. Depending upon the acceleration of the strong ground motion earthquake two results are likely. The lower level acceleration earthquake can fail the walls in bending from the dynamic force of the water causing bending cracks at the base of the walls and resulting in permanent deformations of the walls and even leaks through the cracks. The larger level earthquake can result in shear failures at the base of the walls from the foundation. Shear failures at the base of the wall will result in rapid loss of basin contents and render the basins unusable. Retrofitting the basin walls may not be practical due to their original thin wall construction that is vulnerable to catastrophic shear failure.

2. At the base foundation of this inertia, an expansion joint was placed between the basins to accommodate movement and potential differential settlement. The detrimental effects of this joint can be realized during a strong ground motion earthquake with two likely results. The lower level acceleration earthquake can fail the slab-to-foundation connection by deformation of the slab in bending from the dynamic force of the water causing bending cracks at the resulting in permanent deformations of the slab and leaks through the cracks. The larger level earthquake can result in shear failures at the slab from the foundation. Shear failures at the slab will result in rapid loss of basin contents and render the basins unusable. Retrofitting the basin slabs is likely difficult due to their original thin slab construction that is vulnerable to catastrophic shear failure.

With the 1980 plant expansion drawings, the following vulnerabilities are recognized, many of which are identical to the 1930, 1950, and 1961 original construction items. This commonality and interdependency of the structures is due to the existing structures being interconnected with common walls. The vulnerabilities are as follows:

1. The 80-foot square sedimentation basin with the single platform walkway out to the center of the sedimentation reaction well does not offer adequate seismic resistance to the hydrodynamic loads that can damage and collapse these elements into the basin during strong ground motion earthquakes. The 7-foot square foundation for the center column is too small to resist the large hydrodynamic forces. Strengthening of the platform by spanning clear across the basin with a new bridge/platform is a common method of seismically retrofitting this deficiency. Alternatively, process decisions can dictate the complete removal of the reaction well and platform to eliminate this seismic collapse risk.
2. The thin and lightly reinforced square (in plan) sedimentation basin walls are susceptible to significant hydrodynamic forces during a strong earthquake. The basin walls are thin and lightly reinforced and susceptible to significant

hydrodynamic forces during a strong earthquake. Depending upon the acceleration of the strong ground motion earthquake, the results can likely be wall failure in bending. The acceleration of the earthquake can fail the walls in bending from the dynamic force of the water. The bending cracks, yielding of reinforcing and crushing of concrete at the base of the walls will result in permanent deformations of the walls and cause leaks through the cracks. Retrofitting the basin walls is possible. The retrofit described in the May 29, 2009 memo referenced earlier entailed the addition of 6-inch interior face reinforced concrete walls to the existing walls which will likely be appropriate for these walls as well.

3. The Rogue River intake structure is very tall and laterally loaded by the soil on one side only. During a seismic event, the soil lateral pressure can increase substantially and result in deformation and movement of the intake facility toward the river. This movement can cause failure to the pumps rendering water removal from the river useless until the pump cans are replaced or straightened.
4. The clearwell and filters reinforced concrete are thin and lightly reinforced.
5. The chemical tank and electrical equipment areas have metal deck on steel joist framed roof supported on reinforced concrete block and reinforced concrete walls. The seismic in-plane shear connection from the roof to the walls is detailed for lighter seismic conditions. The out-of-plane wall to roof is detailed for lighter seismic conditions and can lead to separation of the wall from the roofs during a strong ground motion earthquake.

The deficiencies described above could lead to catastrophic failure at the strong ground motion levels that can occur during a code level seismic event. These deficiencies can lead to progressive failure and some partial collapses in the affected areas in code level seismic events.

2.4 Seismic Evaluation Criteria

The primary design seismic evaluation criteria are the IBC with Oregon state amendments, which are based upon both the 2009 edition of the IBC and the ASCE 7-05 building code. The seismic loads are horizontal force ground accelerations of 75% of gravity for short period structures and 38% of gravity for long period (tall or long water basins) structures which equate to 2,475 year return period earthquakes per the current IBC. The 2009 IBC level forces have a 2% chance of exceedance in any 50-year period. Two-thirds of these ground level accelerations are then used for the actual structure seismic acceleration calculations due to the code prescribed procedures. The Importance factor for the facility is 1.5, meaning that the intent of seismic design is to have all structures up and running following a major earthquake event. For design,

seismic loads will be increased by 1.5 to keep structural elements in the elastic range of stress such that they are not yielding, cracking, crushing or leaking and becoming unusable after the earthquake.

2.5 Seismic Hazards and the Potential for Damage

In the event of a major earthquake, the current structural risk to the building is assessed to be high. Overall, the structures have not been adequately designed and detailed to prevent global collapse and major damage in the event of strong ground shaking. In general, major structural elements and connections of the lateral-load carrying system are missing in terms of code requirements in effect at this time and for the expected seismic load performance for this type of structure.

The deficiencies described in the previous sections can lead to catastrophic failure and are a concern at the code level design earthquake. These deficiencies can lead to progressive failure in these areas as evidenced by previous performance of these types of structures in past seismic events.

Chapter 3 Visual Observations

3.1 On-site Inspection/Review

A visual inspection of the Grants Pass Water Treatment Plant was conducted on August 18, 2011. Both the entire external perimeter of the structure at grade level and the interior of accessible areas were viewed. The adjacent Rogue River Intake structure was observed. The north and south filter enclosures were covered with architectural finishes on most surfaces, so limited observations were made to the filter exterior walls and roof. Many of the observed deficiencies of the WTP are captured in Attachment B photographs. The applicable photographs are referenced as Figures contained in Attachment B.

The WTP and Rogue River Intake had the following observed deficiencies with the numbering below referencing the Figure numbers in Attachment B:

1. Both the 1931 and 1950 filter roof structures have multiple seismic deficiencies at the roof level. The wall-to-roof anchorage is the most significant of these vulnerabilities.
2. The blocking for out-of-plane anchorage will likely need to be added at four feet on-center to carry the heavy lateral out-of-plane wall loads. The split blocking will also require replacement.
3. Split truss bottom chord bracing at the bolt line does not offer adequate interconnection for lateral loading and should be retrofitted with new connections. This indicates loss of ability to resist lateral loading in these areas that are now more vulnerable to seismic forces.
4. The bottom chord of the truss is split and not capable of resisting the lateral load required during a strong ground motion seismic event. Retrofit of the truss with interconnected side plates can likely remediate this deficiency.
5. In addition to the deficient wall-to-roof anchorage, this photograph exhibits the gap between the straight wood sheathing and the top chord of the wood truss that does not allow lateral wall out-of-plane shear transfer from the truss top chord into the straight sheathed wood diaphragm. The straight sheathed wood diaphragm is also inadequate to transfer the high seismic load demands. The result of these multiple vulnerabilities is that the as-built roof structure is a collapse hazard in a strong ground motion earthquake.
6. The plan offset in the south wall of the filter building creates a horizontal irregularity on the roof diaphragm concentrating large lateral loads into the wall that is not

extended further southward. These lateral force concentrations were not recognized or designed for in the original design. Retrofit of the roof and south walls for these large seismic loads is recommended to prevent excessive deformations of the south walls relative to the roof resulting in collapse hazards during strong ground motion earthquakes.

7. The combination of lack of anchorage at the top of this column to the roof truss and the lack of the chord/collector connection around the back of the column make this a very seismically unstable interconnection. The assembly of these elements without adequate connections were not recognized or designed for in the original design. Retrofit of the roof members and column connection for these large seismic loads is recommended to prevent excessive deformations of the roof over the column resulting in collapse hazards during strong ground motion earthquakes.
8. The single platform walkway out to the center of the square sedimentation reaction well does not offer adequate seismic resistance to the hydrodynamic loads that can damage and collapse these elements into the basin during strong ground motion earthquakes. Strengthening of the platform by spanning clear across the basin with a new bridge/platform is a common method of seismically retrofitting this deficiency. Alternatively, process decisions can dictate the complete removal of the reaction well and platform to eliminate this seismic collapse risk.
9. The thin and lightly reinforced sedimentation basin walls are susceptible to significant hydrodynamic forces during a strong earthquake. The acceleration of the earthquake can fail the walls in bending from the dynamic force of the water. The bending cracks, yielding of reinforcing and crushing of concrete at the base of the walls will result in permanent deformations of the walls and cause leaks through the cracks. Retrofitting the basin walls is possible and was outlined in the Grants Pass WTP Technical Memorandum titled "Structural and Seismic Upgrades for the Proposed Basin Improvements" (MWH - May 29, 2009). The retrofit described in the TM entailed the addition of 6-inch interior face reinforced concrete walls to the existing walls.
10. The cracked CMU wall of the chemical building with the filter building is indicative of stress concentrations being exceeded, resulting in permanent deformation of the wall and cracking. During a seismic event, further stress concentration at this interconnected vulnerable area can be expected resulting in these cracks opening up further and additional cracks extending out from these existing cracks.
11. A vertical steel wide flange column is cantilevered from the foundation to laterally support the redwood baffle walls in the sedimentation tanks. Both the redwood baffles and the steel columns can collapse, bend and fail when subjected to

hydrodynamic loads requiring replacement of these members after a seismic event provided the basins are still functional. The steel column can be braced back to the walls to provide greater seismic load resistance and be reinforced per the Grants Pass WTP Technical Memorandum titled "Structural and Seismic Upgrades for the Proposed Basin Improvements" (MWH - May 29, 2009).

12. The deteriorated concrete around the filter pipe penetrations have corroded pipe elements and reinforcing elements that are susceptible to brittle failure during a major earthquake's strong ground motion.
13. The below-grade filter gallery north wall has multiple locations where vertical cracks below grade are seeping water. The water coming through the wall is apparently contacting steel through the thickness of the crack and depositing rust stains on the concrete. These cracks are expected to be the weakened plane of the concrete that will be further yielded during strong ground motion.
14. The filter gallery piping passing through the structural wall jamb interrupts the vertical reinforcing steel. With the vertical steel interrupted seismic lateral out-of-plane loads in the wall focus on this highly stressed area and can cause severe cracking and shear failures in this zone during an earthquake.
15. The filter gallery piping passing through the structural wall jamb to the head area interrupts the vertical and horizontal reinforcing steel. With the vertical and horizontal steel interrupted seismic lateral out-of-plane loads in the wall focus on this highly stressed area and can cause severe cracking and shear failures in this zone during an earthquake.
16. In the electrical room, a substantial amount of the roof steel angle ledgers do not have any visible anchor bolts to the concrete block masonry walls. The seismic in-plane and out-of-plane loads needed to be transferred in an earthquake are likely not capable of being transmitted through this ledger angle that does not have anchorage that is visually verifiable. The resulting lack of support can lead to loss of support and partial collapse of roof elements framing into this wall.
17. In the electrical room, a substantial amount of the roof metal deck does not have any visible anchor bolts or ledgers to the concrete block masonry walls. The seismic in-plane and out-of-plane loads needed to be transferred in an earthquake are likely not capable of being transmitted through this bearing connection that does not have anchorage that is visually verifiable. The resulting lack of support can lead to loss of support and partial collapse of roof elements framing into this wall.
18. In the chemical storage area, some of the diagonal bracing for the trusses do not have any visible anchor bolts to the concrete block masonry walls. The seismic out-

of-plane loads needed to be transferred in an earthquake are likely not capable of being transmitted through this connection angle that does not have anchorage that is visually verifiable. The resulting lack of support can lead to loss of support and partial collapse of roof elements framing into this wall. This connection is also vulnerable to wind uplift buckling the bottom chord of the trusses during strong winds.

19. In the chemical storage area, some of the diagonal bracing for the trusses have narrow vulnerable plates to the concrete block masonry walls. The seismic out-of-plane loads needed to be transferred in an earthquake are likely not capable of being transmitted through this thin plate that does not have much capacity. The resulting lack of support can lead to loss of support and partial collapse of roof elements framing into this wall. This connection is also vulnerable to wind uplift buckling the bottom chord of the trusses during strong winds.
20. In the chemical storage area, the steel beam connecting into the shear wall appears to not have much capacity to transfer lateral loads. In addition,, in the direction 90 degrees to this connection, there is no visible seismic collector connection to the roof metal deck. The seismic in-plane and out-of-plane loads needed to be transferred in an earthquake are likely not capable of being transmitted through this light and non-existent connection that does not have much capacity. The resulting lack of support can lead to loss of support and partial collapse of roof elements framing into this wall.
21. In the chemical area, the steel truss to concrete block wall grout pockets have honeycombed grout that will likely limit the out-of-plane load transfer from the truss to the walls during strong ground motion earthquakes. The seismic out-of-plane loads needed to be transferred in an earthquake are likely not capable of being transmitted through this honeycombed bearing connection that does not have adequate shear capacity and edge distance on the anchorage. The resulting lack of support can lead to loss of support and partial collapse of roof elements framing into this wall during an earthquake.
22. In the chemical area, a substantial amount of the roof steel angle ledgers do not have any visible anchor bolts to the concrete block masonry walls. The seismic in-plane and out-of-plane loads needed to be transferred in an earthquake are likely not capable of being transmitted through this ledger angle that does not have anchorage that is visually verifiable. The resulting lack of support can lead to loss of support and partial collapse of roof elements framing into this wall.
23. In the chemical area, the steel beam connecting into the wall appears to not have much capacity to transfer lateral loads due to limited edge distance of the anchorage. The seismic out-of-plane loads needed to be transferred in an earthquake are likely

not capable of being transmitted through the connection that does not have much capacity. The resulting lack of support can lead to loss of support and partial collapse of roof elements framing into this wall.

24. The river intake pump station top-level housing is a three-sided structure susceptible to torsional (twisting) loads in a seismic event. The torsional demands can result in damage to the top-level walls and roof structure during a strong ground motion earthquake.
25. The river intake pump station wood roof framing connections to the walls for out-of-plane loading are very light and not capable of transferring the high seismic forces that can be developed by the structure. The resulting lack of support can lead to loss of support and partial collapse of roof elements framing into this wall.
26. This rigid pipeline locked into the basin wall reinforced concrete and the sidewalk concrete does not allow differential movement of this element. Differential settlement of the basin at the wall relative to the surrounding grade can shear off this pipe at this location during strong ground motion earthquakes.
27. The overhead interconnecting reinforced concrete water channel from the 1931 to 1950s era basins is susceptible to high stress and differential movement during seismic events. The operation of this channel post earthquake may not be possible due to the cracking, breaking and leakage.
28. The east wall of the 1930's era basin was observed to have vertical cracks at several locations. These cracks were also well documented in the Grants Pass WTP Technical Memorandum titled "Structural and Seismic Upgrades for the Proposed Basin Improvements" (MWH - May 29, 2009). These cracks are indicative of many items such as initial drying shrinkage, differential settlement, insufficient horizontal and vertical reinforcing for gravity and seismic loads. During the hydrodynamic loading from a strong ground motion earthquake we believe these walls can be severely damage in bending or shear at their base resulting on loss of contents and use.
29. The east wall of the 1930's era basin was observed to have vertical cracks at several locations. There is also vertical offset of these walls well documented in the Grants Pass WTP Technical Memorandum titled "Structural and Seismic Upgrades for the Proposed Basin Improvements" (MWH - May 29, 2009). These cracks are indicative of many items such as initial drying shrinkage, differential settlement, insufficient horizontal and vertical reinforcing for gravity and seismic loads. During the hydrodynamic loading from a strong ground motion earthquake we believe these walls can be severely damage in bending or shear at their base resulting on loss of contents and use.

30. The east wall of the 1930's era basin interconnection to the filter building wall was observed to have vertical cracks. These cracks are indicative of many items such as initial drying shrinkage, differential settlement, insufficient horizontal and vertical reinforcing for gravity and seismic loads. During the hydrodynamic loading from a strong ground motion earthquake we believe these walls can be severely damage in bending or shear at their base resulting on loss of contents and use. The acceleration of the earthquake can fail the walls in bending from the dynamic force of the water. The bending cracks, yielding of reinforcing and crushing of concrete at the base of the walls will result in permanent deformations of the walls and cause leaks through the cracks. Retrofitting the basin walls is possible and was outlined in the Grants Pass WTP Technical Memorandum titled "Structural and Seismic Upgrades for the Proposed Basin Improvements" (MWH - May 29, 2009). The retrofit described in the TM entailed the addition of 6-inch interior face reinforced concrete walls to the existing walls.
31. The northeast corner of the filter building wall was observed to have horizontal cracks at several exterior locations. These cracks are indicative of many items such as initial drying shrinkage, differential settlement, insufficient horizontal and vertical reinforcing for gravity and seismic loads. During the seismic loading from a strong ground motion earthquake we believe these walls can be severely damaged in bending or shear with potential loss of use.
32. The adjacent Rogue River's proximity to the treatment facility produces a steep slope on the south side of the facility. The intake structure appears to have rotated toward the river due to the unbalanced loading from the soil embankment. During a seismic event, the soil lateral pressure can increase substantially and result in further movement of the intake facility toward the river. The river's proximity has lead to previous erosion of the embankment that could be further exacerbated with a strong ground motion earthquake accelerating this unrestrained steep edge toward the river.
33. The slope stabilization along the river embankment is a recognition that the erosion of the embankment is proceeding with time at the end of this bend in the river. The proximity of the entire facility being so close to the embankment creates a vulnerably top surcharge-loaded slope. During strong ground shaking, any weakened planes in the slope can be vulnerable to stability concerns.
34. The electric transformer did not have visible anchorage to its foundation. To prevent sliding of the transformer during significant lateral loading from severing, the electrical connections positive mechanical anchorage should be installed.

35. The incoming natural gas line pipe diameter necks down to a very narrow pipe section. The narrow pipe section does not offer much structural capacity to resist cyclical seismic loading of the meter moving back and forth like a pendulum. The narrow section can bend and fail by cracking that can not only lead to loss of gas service but be easily ignited by any surrounding ignition source causing a fire to the adjacent wood structure. Correction of the incoming gas line's diameter and independent support of the meter to prevent breaking the line and feeding a fire are prudent measures to reduce risk. Additionally, an automatic seismic acceleration shut off valve could be added for added safety.
36. The polymer tanks held in the stands lack anchorage to prevent sliding or potential overturning. With the high center of gravity of these tanks, they can topple in a large earthquake and the bases can certainly slide (or walk over) and fall off the raised concrete slab leading to collapse of the tanks in even a moderate earthquake.
37. The stainless steel polymer tanks lack mechanical anchorage to prevent sliding or potential overturning. With the high center of gravity of these tanks they can topple in a large earthquake and the bases can certainly slide (or walk over) and fall off the raised concrete slab leading to collapse of the tanks in even a moderate earthquake.
38. The aluminum sulfate and T-Floc tanks lack anchorage at their bases to prevent sliding. Sliding of the tanks can sever their inlets/outlets causing evacuation of the contents of the tanks and rendering the treatment process inoperable.
39. The electrical equipment has limited anchorage on narrow concrete pads that likely have little to prevent sliding or potential overturning. With the high center of gravity of the electrical equipment, they can topple in a large earthquake and the bases can certainly slide (or walk over) and fall off the raised concrete slab leading to collapse of the equipment in even a moderate earthquake. This destruction of the equipment can render the treatment plant inoperable.
40. The t-bar ceiling, lighting and HVAC ducts do not have lateral splayed wires or compression struts to adequately resist seismic loads. The result of this deficiency is that during strong ground motion the ceilings, lights and duct work can move significantly and collapse creating a space below that is unusable.
41. The lighting over the filters does not have lateral splayed wires or compression struts to adequately resist seismic loads. The result of this deficiency is during strong ground motion the lights can move significantly and collapse creating a space below that is unusable.

3.2 Seismic Vulnerabilities

In order to reduce the risks of the observed deficiencies, we recommend consideration of seismic and wind resistant structural upgrades for the facility that are further discussed in Chapter 4. Many of the deficiencies described above and in Chapter 2 can have progressive domino-type effects on the failure of structural elements during strong ground motion earthquakes. By allowing this structure to have so many deficiencies during a seismic event, devastating effects to not only the performance of the local element but also to the entire structure can be the result. A partial collapse that occurs at one of the operational elements, rendering the operation of that function to be lost, can have a chain reaction effect on the entire facility rendering the treatment plant ineffective at treating the water to fight fires or provide drinking water after a seismic event. Continued progressive failure of adjacent structures can also result once one area has been severely weakened by the original earthquake such that aftershocks of a lower magnitude event can cause failures as well.

Chapter 4 Summarized Recommendations

4.1 Conceptual Seismic Retrofits

The following list of retrofit items are focused on the effect of strong ground motion from a major seismic event that can be categorized as a code-level seismic performance of the various plant structures. The list is a prioritized ranking of the retrofits deemed most significant to the structure's lateral load resistance at this time. The next phase of the retrofit development process would be to perform seismic calculations to confirm the size and extent of these retrofit items, and to refine cost estimates.

1. Installation of a tieback or soil nail wall along the steep bank of the Rogue River that has steel reinforced shotcrete placed between a grid of deep tiebacks. Figure 4.1.1.
2. Install tiebacks through the Rogue River Intake North wall of the structure to connect the tall reinforced concrete structure to the steep embankment and resist the large seismic lateral load. Figure 4.1.2.
3. Tile roof removal and installation of the new plywood sheathing nailed through the framing. Reinstallation of the roof tile upon completion of all new plywood. Figure 4.1.3 a and b.
4. Installation of new wood roof wall-to-roof framing and wall-to-roof anchorage at four feet on-center around the entire roof perimeter at the filters and tower. Figure 4.1.4.
5. Strengthen the sedimentation basin walls with a combination of new reinforced concrete retrofit as well patching and repairing to prevent shear bending failures in the Basin 1 and 2 walls. Remove, inspect, repair and replace baffle walls per the Grants Pass WTP Technical Memorandum titled "Structural and Seismic Upgrades for the Proposed Basin Improvements" (MWH - May 29, 2009). Figures 4.1.5 a-e.
6. Strengthen the mixing tank and clearwell walls and columns with a combination of new reinforced concrete retrofit as well as fiber-reinforced polymer fiber wrap additions to prevent shear and bending failures in the basin walls and columns. Figures 4.1.6 a and b.
7. Strengthen the 80-foot square sedimentation basin walls and center column with a combination of new reinforced concrete retrofit as well as steel platform, column and fiber-reinforced polymer additions to prevent shear and bending failures in the basin walls. Figures 4.1.7 a and b.

8. Installation of new steel anchorage for the wall-to-roof in-plane shear and framing and wall-to-roof anchorage at 4 feet-on-center around the entire roof perimeter at the chemical and electrical room structures. Figures 4.1.8 a – c.
9. Install Chemical tank anchorage to prevent sliding and toppling of chemical tanks. Figures 4.1.9 a – m.
10. Install Electrical Equipment anchorage to prevent sliding and toppling of chemical tanks. Figures 4.1.9 a – m.
11. Water Pipe anchorage at Filter Galleries and Chemical Pipe anchorage at the Chemical Areas. Figures 4.1.11 a and b.

Table 4.1. Retrofit Summary Table

Number	Retrofit
1	New tiebacks and shotcrete of embankment entire face
2	New tiebacks through North wall of the intake
3	Tile roof removal and installation of the plywood sheathing and replacement of tile roof.
4	Installation of new roof wall to roof framing and wall to roof anchorage
5	Strengthen sedimentation basin walls and baffle walls and leak repairs with reinforced concrete. Patching, repairing per the Grants Pass WTP Technical Memorandum titled “Structural and Seismic Upgrades for the Proposed Basin Improvements” (MWH - May 29, 2009) for Basins 1 and 2
6	Strengthen mixing tank and clearwell walls and columns with a reinforced concrete and FRP
7	Strengthen 80 foot square sedimentation walls and column with steel, reinforced concrete and FRP
8	Installation of new steel anchorage for the wall at the chemical and electrical rooms
9	Install chemical tank anchorage
10	Install electrical equipment anchorage
11	Pipe anchorage and support

Proposed modifications are expected to provide more-positive, redundant transfer of seismic loads through the structures and prevent separations that could lead to progressive, severe damage. **(Although the modifications are not expected to bring the facility into complete compliance with current building code requirements, strengthening modifications are expected to reduce the possibility of severe damage in a large strong ground motion earthquake.)** Retrofits and design criteria will be negotiated with local building officials to find criteria that is reasonable and safe. Often the ASCE Manual 41 is used on old structures and is accepted by many jurisdictions.

4.2 Seismic Retrofit Cost Estimates

The following retrofit costs presented in Table 4.2 are focused on the effect of seismic retrofits as described above. All the listed retrofits are assumed to be completed in one contract for cost efficiencies. Should the retrofits be completed as individual separate contracts, the cost estimates listed below should be factored accordingly for less efficiency. The listed cost estimates are rough order-of-magnitude only and do not include the cost of overtime, internal management, off-shift work, interruption of process, any unusual safety requirements or project coordination with Treatment Plant functions and these cost elements should be factored accordingly. The estimates are based on present knowledge, which is in large part based upon experience and judgment. Once actual preliminary and detailed engineering designs for the retrofits are available, more-accurate estimates can be developed.

Table 4.2. Seismic Retrofit Cost Estimate Summary Table

Number	Retrofit	Rough Order of Magnitude Cost
1	Tiebacks and shotcrete at embankment	\$1,000,000
2	Tiebacks at Intake	\$200,000
3	Plywood roof sheathing	\$200,000
4	Wall-to-roof anchorage	\$400,000
5	Basin wall strengthening	\$1,500,000
6	Strengthen tank, clearwell walls, roof and columns	\$300,000
7	Strengthen 80' sq sed basin walls and columns	\$500,000
8	Strengthen Chemical and Elect Room wall-to-roof conn.	\$400,000
9	Anchor chemical tanks	\$100,000
10	Anchor electrical equipment	\$200,000
11	Pipe Anchorage	\$200,000
	Subtotal	<u>\$5,000,000</u>
	Mobilization and General Conditions (15%)	\$750,000
	Contractor Profit and Overhead (10%)	<u>\$500,000</u>
	Contingencies, Engineering, CMS, Legal, Admin (45%)	<u>\$2,250,000</u>
	Total	<u>\$8,500,000</u>

Once detailed engineering drawings are prepared, the construction costs can be more accurately estimated.

4.3 Maintenance Retrofits

The following simple internal retrofit items are more maintenance-related than directly affecting seismic performance of the plant.

1. Multiple telephone, communications, and electrical cabinets at many levels are not adequately attached to the concrete walls, floors, and ceilings to resist seismic loads. We recommend these pieces of equipment and cabinets be positively attached to all adjacent structural surfaces.
2. Similarly, file cabinets, bookcases, and some furnishings are not attached to the floor and wall structures they are supported on or are adjacent to. We recommend all these furnishings be positively attached to their adjacent structures to prevent falling and life safety hazards.
3. Also, desk mounted equipment and computer screens and monitors lack positive attachment to desktops. This critical equipment is fragile and will break during a strong ground motion earthquake and can be a life-safety hazard to tower occupants. We recommend all equipment be positively secured to the desks and furnishings.

4.4 New Plant Alternative

As an alternative to the extensive structural modifications detailed in this report, a budget-level construction cost estimate was prepared for a new, fully compliant Water Treatment Plant that is similarly sized to the existing plant in all major facilities. It is assumed this new plant would be located on a not-yet-identified, unoccupied site. The new treatment plant would make use of the following primary treatment processes:

- Rapid Mix
- Flocculation/Sedimentation
- Granular Media Filtration with a dual media bed of granular activated carbon (GAC) over sand
- Chlorine Disinfection
- Treated Water Storage (2.0 MG) and High Service Pumping

The new plant would also include the following ancillary facilities:

- Backwash Water Equalization and Clarification
- Sludge Drying Lagoons
- Backup Power Facilities

A new plant would provide many advantages over the existing plant. In addition to a longer useful life and structural/seismic code compliance, a new plant would provide a more robust, energy efficient, and higher performing treatment process. A new plant would likely feature a more operator-friendly work environment as well as space for future plant expansions. The flexibility afforded with a new plant would also be beneficial in the future as the water industry's regulations and standards for emerging contaminants and pathogens continue to be defined.

Table 4.3, provides the budget-level capital cost estimate for a new plant. The costs do not include provisions for land acquisition, a new intake, raw water pump station, or associated conveyance systems as these are provided in Table 4.4. It is expected that the treatment plant will need to occupy a site with an area of approximately 6 acres. The costs provided are based on our past experience designing and building new or expanded water treatment facilities throughout the region. This estimate is provided at an "order of magnitude" level only and, as such, should only be used for early budgeting and decision-making purposes.

Provisions for land acquisition, a new intake, raw water pump stations and associated conveyance systems are challenging to cost estimate at this early stage of planning as they can take many forms depending upon location, land availability and other factors. For example – If a suitable site were found near the existing treatment plant, it is possible that the existing high service pump station would be adequate to utilize the current intake and pump to the new plant. If a site were found farther away, but close to the river, it may be more feasible to construct a new intake near the new plant than to pump and convey water from the existing intake. For this reason, we did not include these highly variable elements in the Cost Projection for a New Treatment Plant (Table 4.3), but still want to give the City some guidance for cost estimating. Table 4.4 shows an "a la carte" menu of costs that can be manipulated as design progresses. To these raw construction costs, the customary additional percentages should be applied such as:

- 10% for Contractor Profit and Overhead
- 15% for Contractor Mobilization and General Conditions
- 45% for Contingencies, Engineering, CMS, Legal and Admin

Items that are unique to an a la carte menu item are included and listed in the Table.

Table 4.3. Order of Magnitude Capital Cost Projection for a New Treatment Plant Designed to Treat 20 mgd.

Facility	Estimated Cost, 2011 \$
Influent Flow Metering and Flash Mix Facilities	\$500,000
Flocculation/Sedimentation Basin (2 x 10 mgd with Plate Settlers)	\$2,500,000
Filters	\$7,500,000
Treated Water Storage (2.0 MG Volume)	\$3,000,000
High Service Pump Station and Metering	\$2,500,000
Chemical Storage and Feed Facilities	\$1,500,000
Backwash Equalization Basin and Return Pump Station	\$600,000
Backwash Wastewater Clarification	\$750,000
Sludge Drying Lagoons and Decant Pump Station	\$1,500,000
New Admin & Laboratory Facilities	\$1,000,000
Engine Generator/Backup Power Facilities	\$1,000,000
Landscaping	\$500,000
Subtotal	\$22,850,000
Mobilization and General Conditions (@15%)	\$3,400,000
Electrical (@ 12%)	\$2,700,000
Site Civil and Yard Piping (@ 15%)	\$3,400,000
Instrumentation (@ 6%)	\$1,400,000
Subtotal	\$10,900,000
Contingencies, Engineering, CMS, Legal, Admin (@ 45%)	\$15,250,000
TOTAL	\$49,000,000

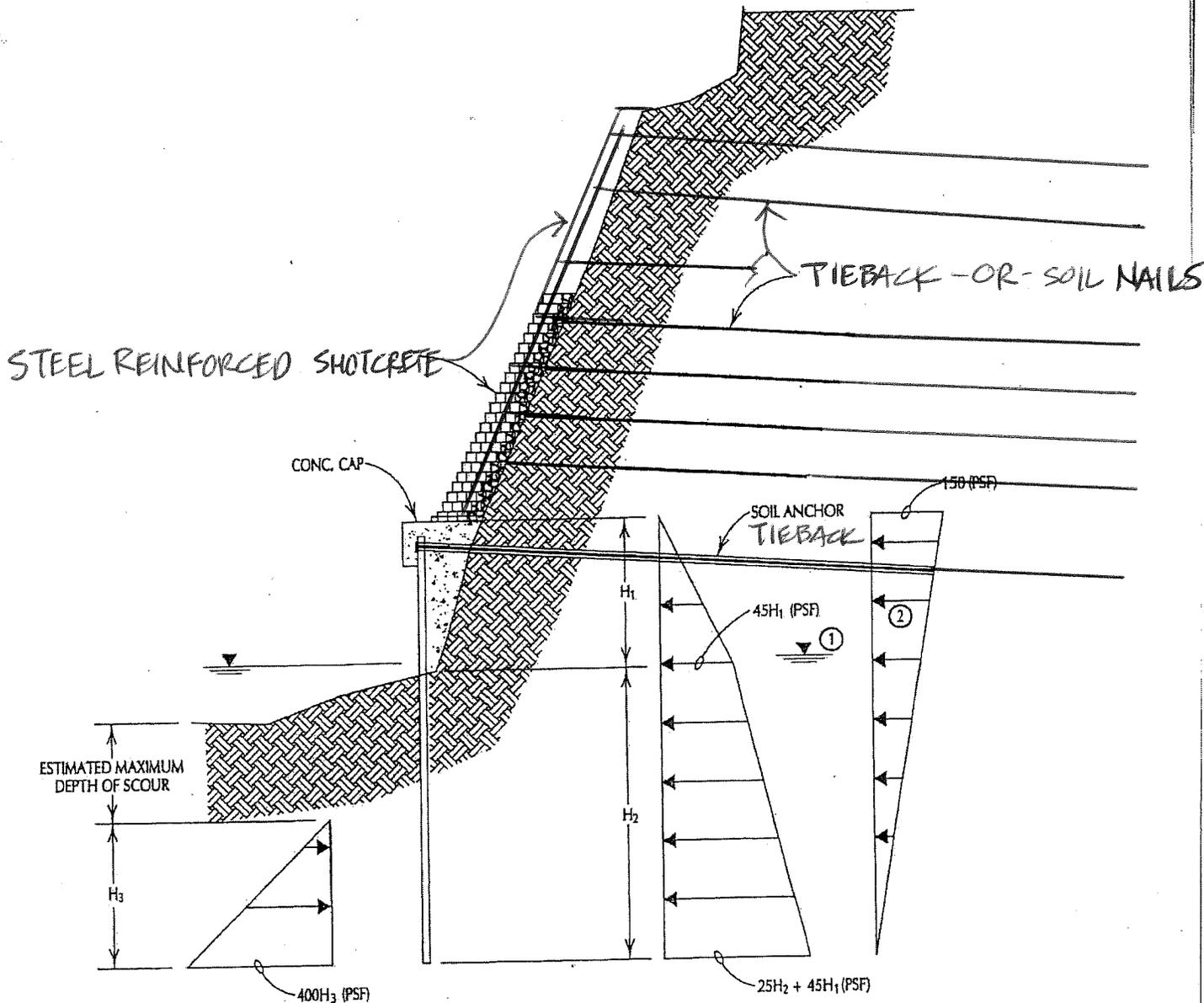
Cost shown do not include:

1. Intake, Raw Water Pump Station, Raw Water Piping.
2. Finished Water Transmission Piping.
3. Land Acquisition costs for new WTP site.
4. Land Use Permitting

Table 4.4. A la Carte Items for Consideration in Order of Magnitude Construction Cost Projections for a New Treatment Plant Designed to Treat 20 mgd.

	Estimated Construction Cost, 2011 \$
Piping	
20" diameter piping (\$/ft)	\$500
24" diameter piping (\$/ft)	\$600
30" diameter piping (\$/ft)	\$750
36" diameter piping (\$/ft)	\$900
48" diameter piping (\$/ft)	\$1,200
Assumptions: Pipes are in roadways and are of ductile iron	
Intake	
Environmental Assessment and Permitting	\$40,000
Structure	\$7,500,000
Pumping (\$/hp)	\$1,500
Finished Water Pumping	
Pumping (\$/hp)	\$1,500

- Permitting costs include approximately 6 meetings with State officials
- EA includes approximately 1 week per month of labor hours for a total of 5 months
- Structure estimates are based on similar structure costs in the region and have not been detailed out due to the wide range of variables involved in this work.



NO SCALE

NOTES:

- ① ESTIMATED LOW RIVER ELEVATION
- ② LATERAL LOAD DUE TO STACKED GEOCELL SURCHARGE

GRI WEST YOST & ASSOCIATES
GRANTS PASS RAW WATER INTAKE IMPROVEMENTS

TYPICAL SECTION WITH RECOMMENDED
LATERAL EARTH PRESSURES

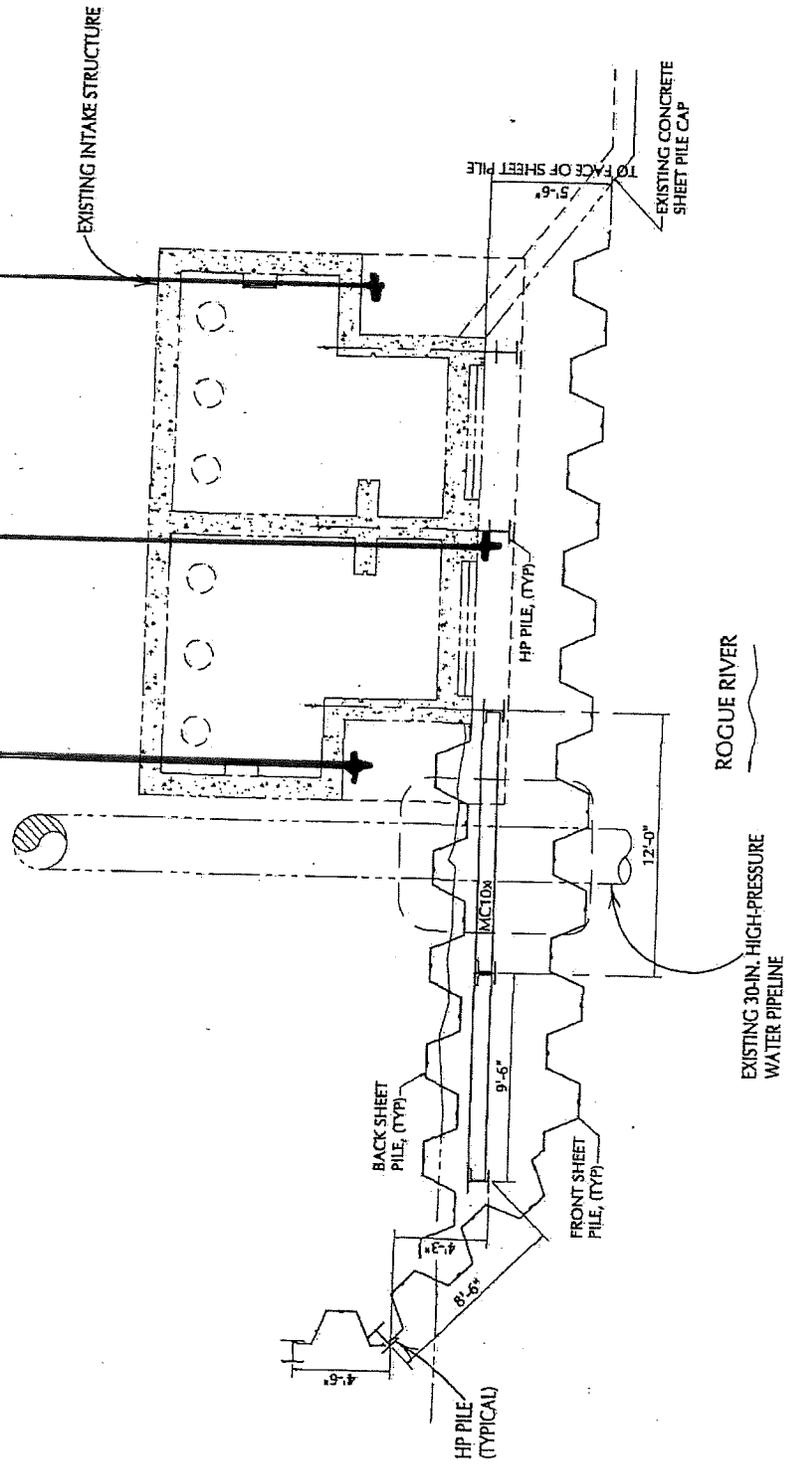
SEPT. 2005

JOB NO. 4289

FIG. 4

FIGURE 4.1.1 TIEBACK / SOIL NAIL WALL

TIEBACKS



SITE PLAN FROM FILE BY WEST YOST & ASSOCIATES (UNDATED)



WEST YOST & ASSOCIATES
GRANTS PASS RAW WATER INTAKE IMPROVEMENT

INTAKE STRUCTURE PILE PLAN
FIGURE 4.1.2 TIEBACKS THROUGH INTAKE

SEPT. 2005

JOB NO. 4289

FIG.

REMOVE & REINSTALL TILE
 ROOF OVER NEW PLYWOOD
 SHEATHING PER FIGURE 4.1.3b

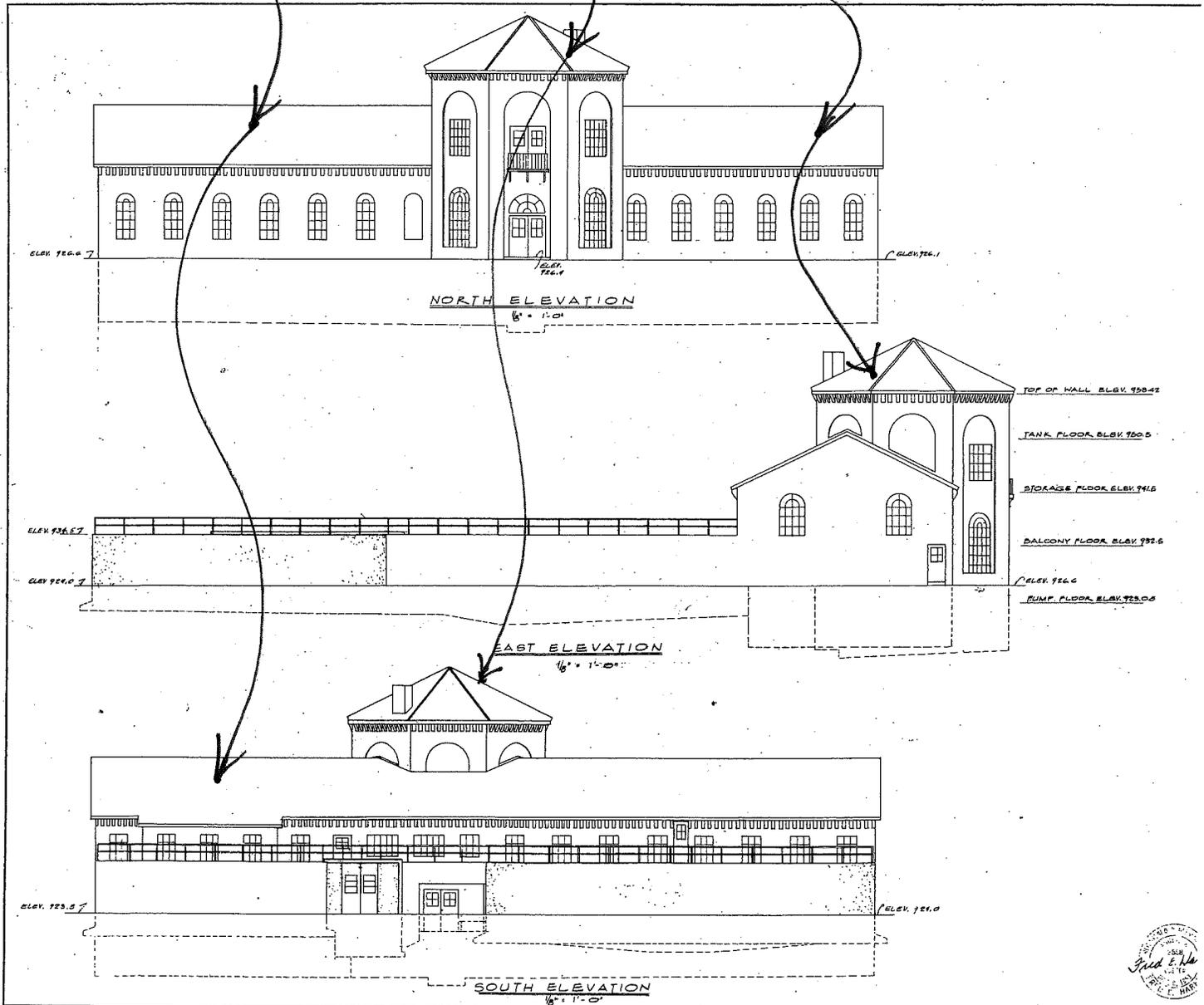
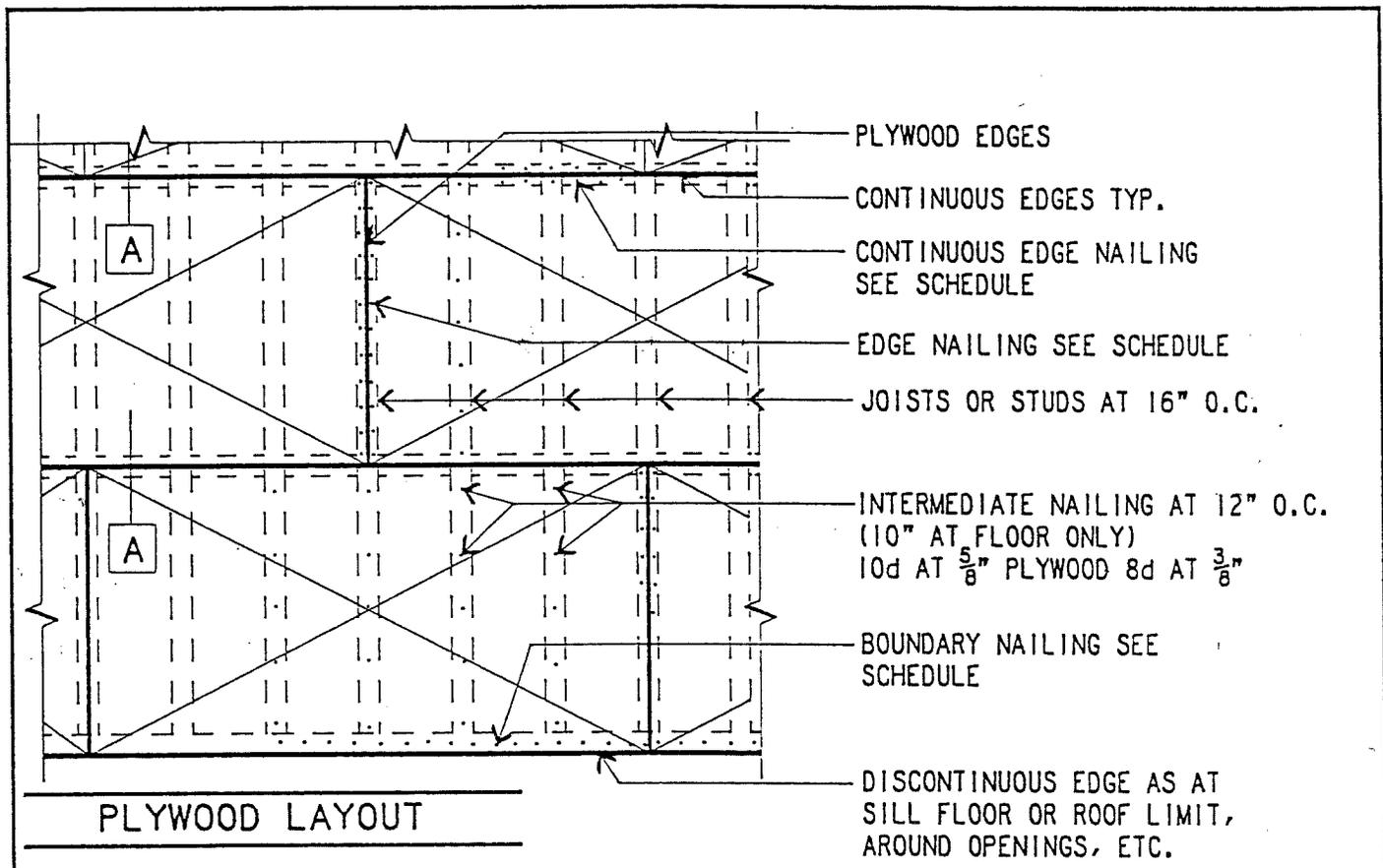
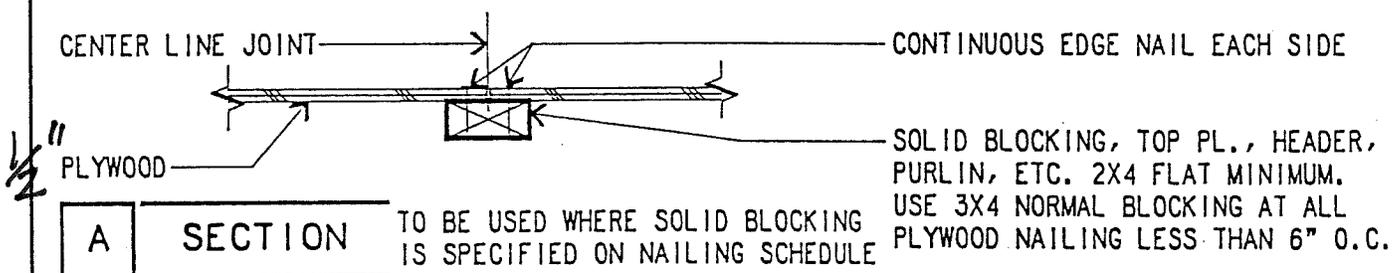


FIGURE 4.1.3a





PLYWOOD LAYOUT



A SECTION TO BE USED WHERE SOLID BLOCKING IS SPECIFIED ON NAILING SCHEDULE

NOTES:

1. SEE DRAWINGS FOR PLYWOOD THICKNESS.
2. PLYWOOD SHEETS SHALL BE 4'-0"X8'-0" EXCEPT EDGE PIECES SHALL NOT CONTAIN LESS THAN 12 SQ. FEET NOR BE LESS THAN 2 FEET WIDE.
3. WALL PLYWOOD MAY BE LAID WITH FACE GRAIN PARALLEL TO STUDS BUT SHALL BE SOLID BLOCKED OR EDGE SUPPORTED IN ACCORDANCE WITH SECTION 1 ABOVE.
4. FOR $1\frac{1}{8}$ " T & G PLYWOOD, BLOCKING MAY BE DELETED WHEN $1"$ X $\frac{3}{8}"$ X16 GA. STAPLES ARE USED AT $\frac{1}{2}$ NAIL SPACING.
5. B.F. GOODRICH PL400 ADHESIVE OR APPROVED EQUAL IS USED BETWEEN PLYWOOD AND FLOOR FRAMINGS.

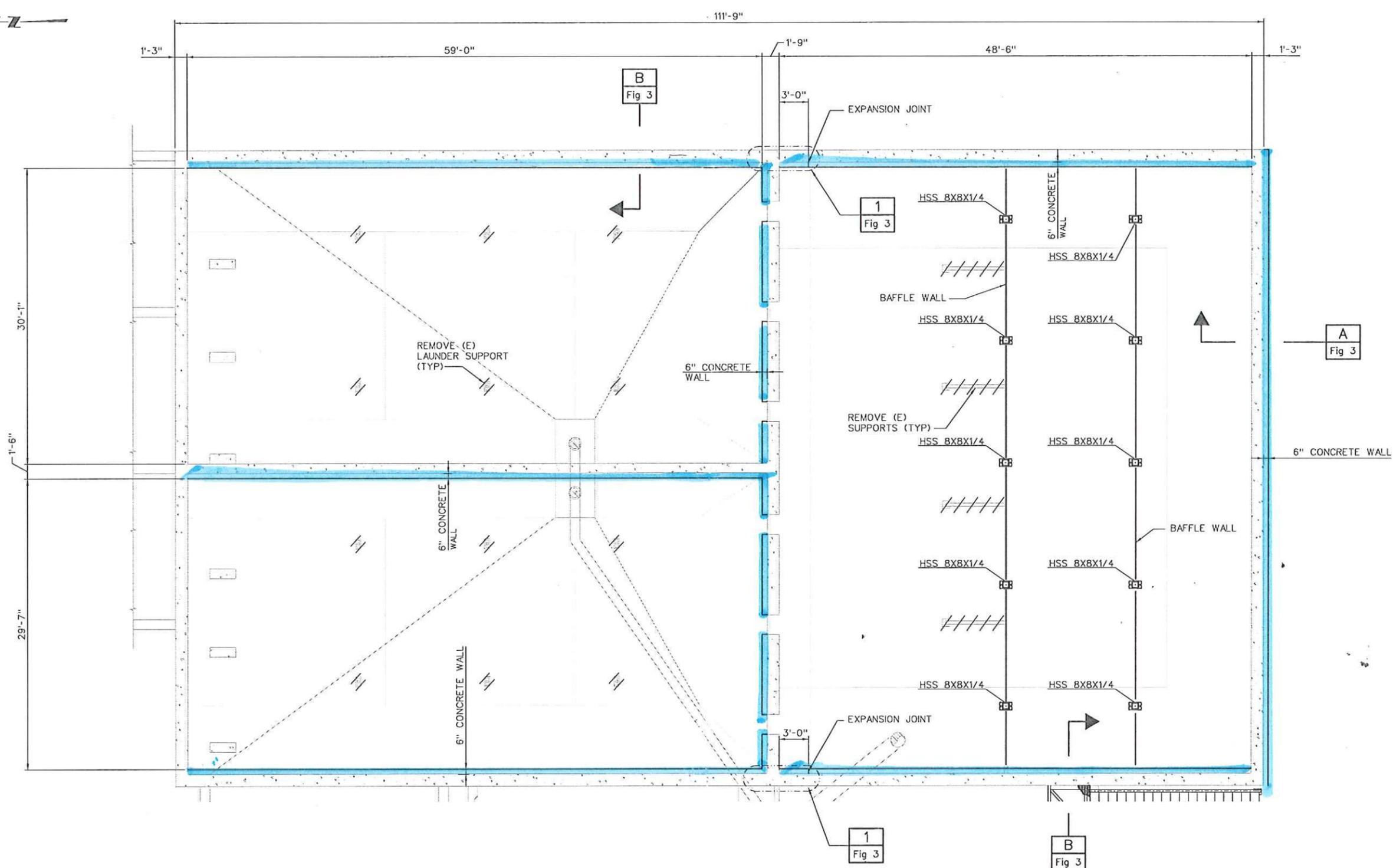


WF 11

PLYWOOD ROOF AND DIAPHRAGM LAYOUT

FIGURE 4.1.3b

File: ISO1.dwg Model: Layout1 ColorTable: bw.ctb DesignScript: MWH\plot_PenTable_VBS.pen PlotScale: 2:1 User: Crystal Roe Plot Date: 25-MAY-2009 08:40



FOUNDATION PLAN

FIGURE 4.1.5a

REV	DATE	BY	DESCRIPTION

SCALE 3/16" = 1'-0"	WARNING IF THIS BAR DOES NOT MEASURE 1' THEN DRAWING IS NOT TO SCALE	DESIGNED <u>CMR</u> DRAWN <u>CMR</u> CHECKED <u>CHECKED</u>
------------------------	---	---

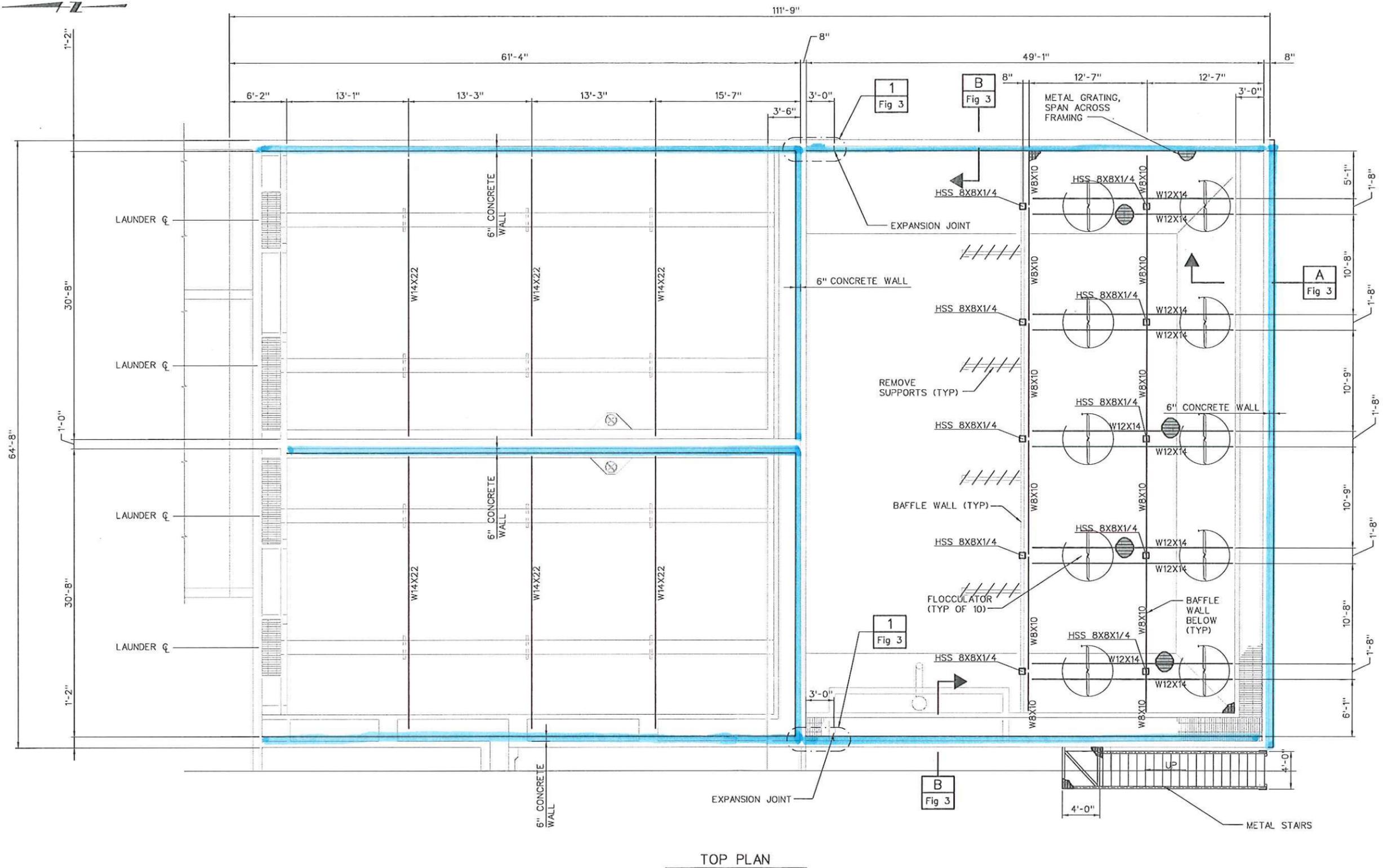
SUBMITTED BY		
(PROJECT MANAGER'S NAME)	LICENSE NO.	DATE
(COMPANY OFFICER'S NAME)	LICENSE NO.	DATE



City of Grants Pass

SEDIMENTATION BASIN 1
STRUCTURAL
FOUNDATION PLAN

SHEET
Fig 1
Job No.



TOP PLAN

FIGURE 4.1.5b

REV	DATE	BY	DESCRIPTION

SCALE
3/16" = 1'-0"

WARNING
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

DESIGNED CMR
DRAWN CMR
CHECKED CHECKED

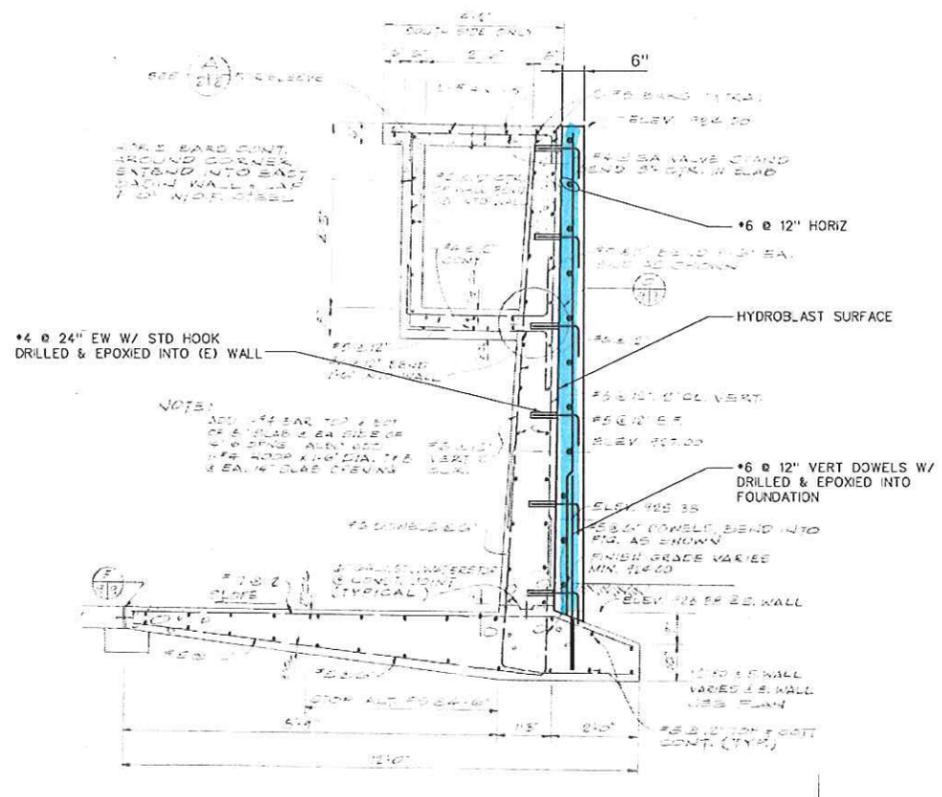
SUBMITTED BY
(PROJECT MANAGER'S NAME) LICENSE NO. DATE
(COMPANY OFFICER'S NAME) LICENSE NO. DATE



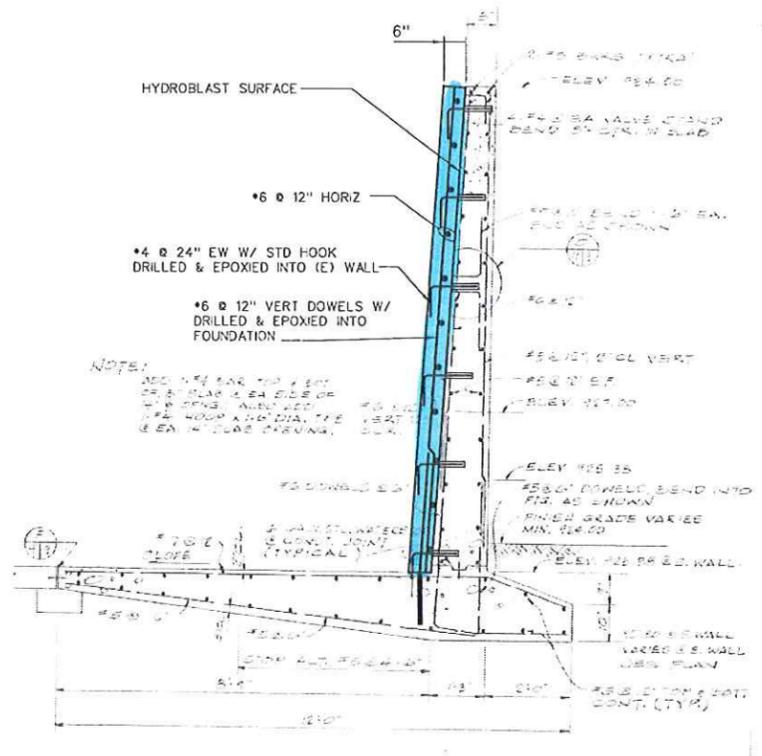
City of Grants Pass

SEDIMENTATION BASIN 1
STRUCTURAL
TOP PLAN

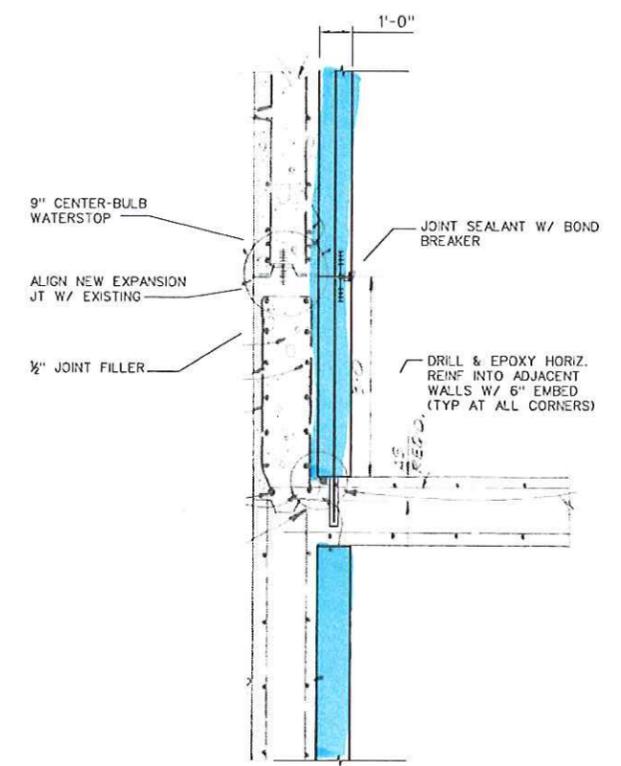
SHEET
Fig 2
Job No.



SECTION A
SCALE: 1/2" = 1'-0"
Fig 1
Fig 2



SECTION B
SCALE: 1/2" = 1'-0"
Fig 1
Fig 2

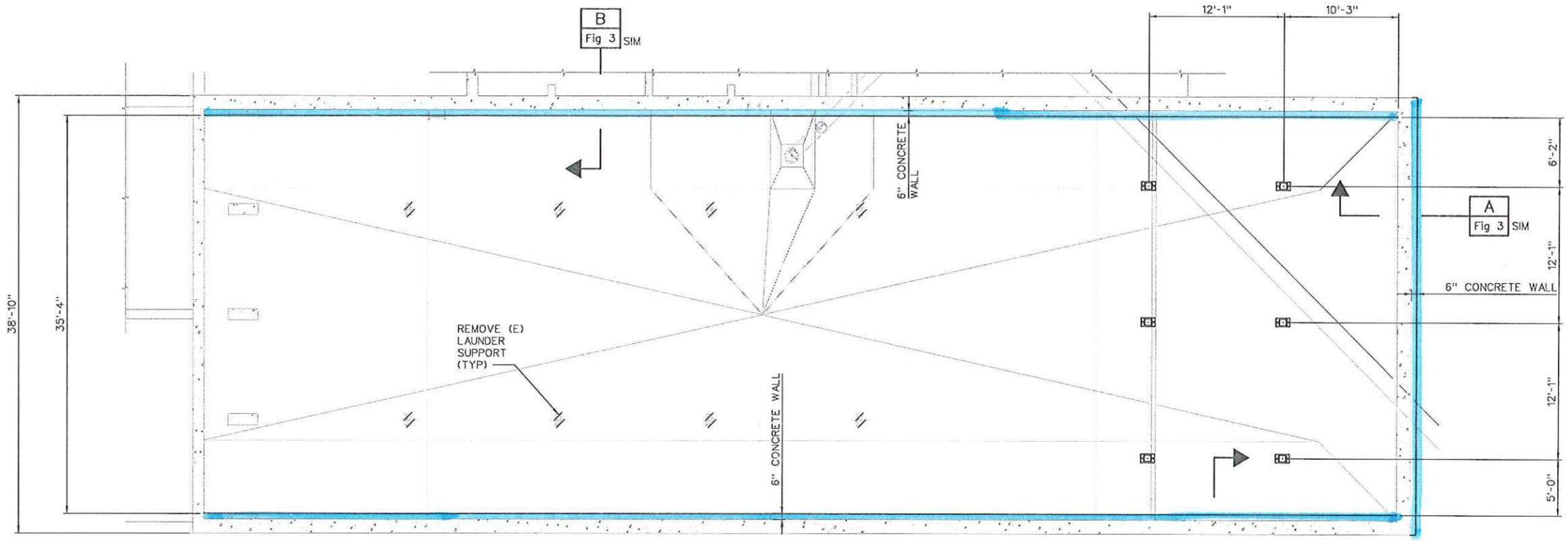


DETAIL 1
SCALE: 3/4" = 1'-0"
Fig 1
Fig 2

FIGURE 4.1.5c

REV	DATE	BY	DESCRIPTION	SCALE	AS NOTED	DESIGNED	CHECKED	SUBMITTED BY	(PROJECT MANAGER'S NAME)	LICENSE NO.	DATE		City of Grants Pass	SEDIMENTATION BASIN NO. 1 STRUCTURAL SECTIONS AND DETAIL	SHEET Fig 3 Job No.
						CMR	CHECKED	(COMPANY OFFICER'S NAME)	LICENSE NO.	DATE					

File: 2501.dwg Model: Layout1 ColorTable: bw.ctb DesignScript: MWH_plo_L_PenTable_v85.pen PlotScale: 2:1 User: Crystal Roe Plot Date: 29-MAY-2009 08:43



FOUNDATION PLAN

FIGURE 4.1.5d

REV	DATE	BY	DESCRIPTION

SCALE
3/16" = 1'-0"

WARNING
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

DESIGNED CMR
DRAWN CMR
CHECKED CHECKED

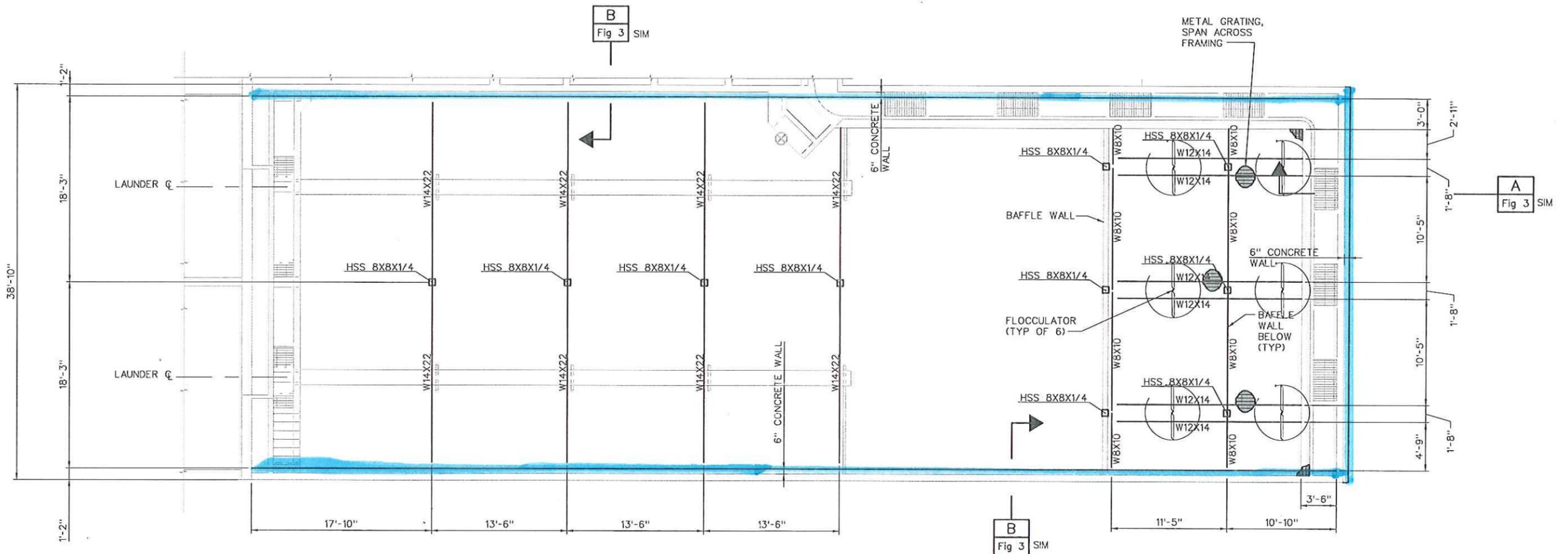
SUBMITTED BY
(PROJECT MANAGER'S NAME) LICENSE NO. DATE
(COMPANY OFFICER'S NAME) LICENSE NO. DATE



City of Grants Pass

SEDIMENTATION BASIN NO. 2
STRUCTURAL
FOUNDATION PLAN

SHEET
Fig 4
Job No.



TOP PLAN

FIGURE 4.1.5e

REV	DATE	BY	DESCRIPTION

SCALE
3/16" = 1'-0"

WARNING
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

DESIGNED CMR
DRAWN CMR
CHECKED CHECKED

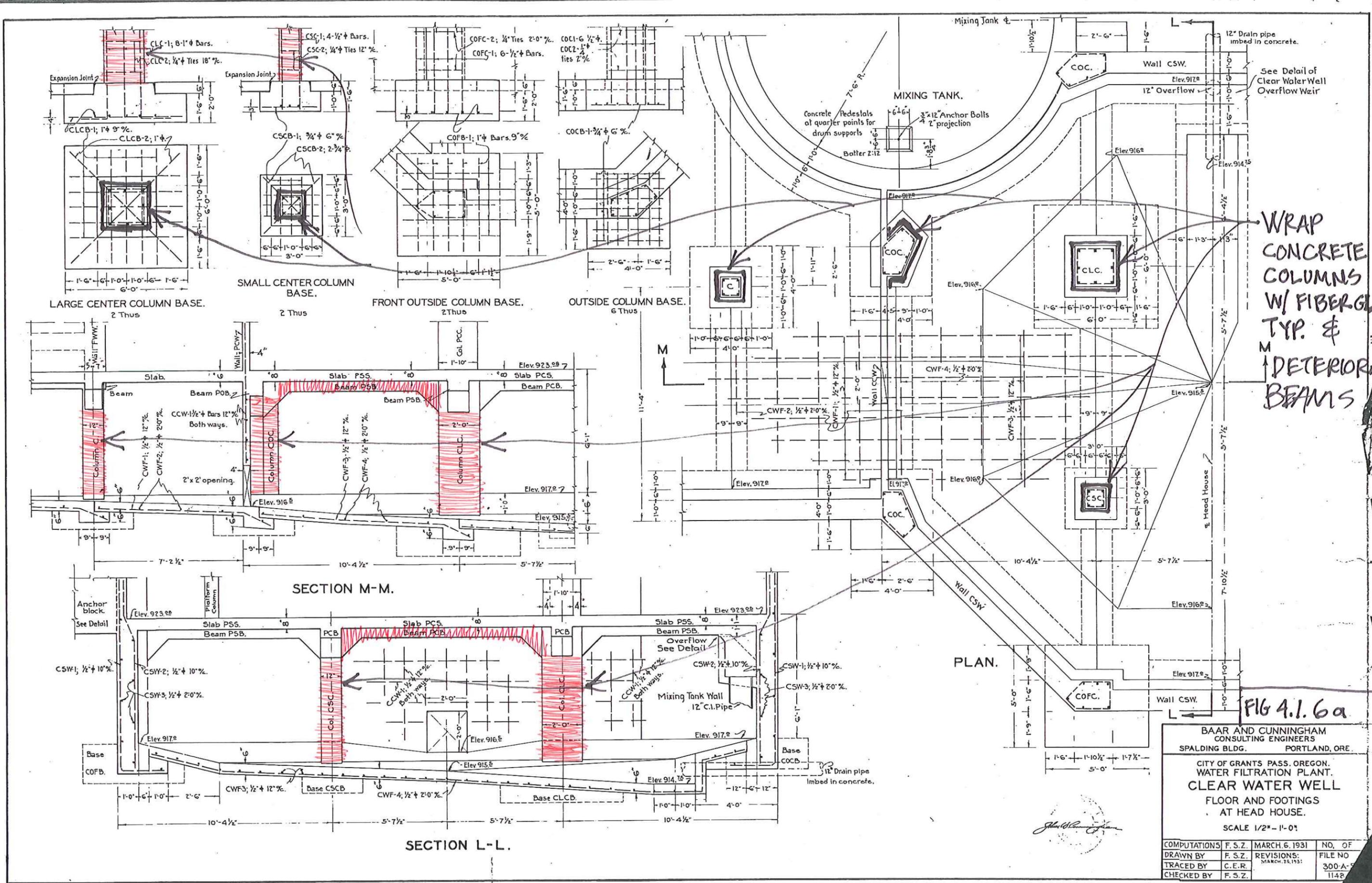
SUBMITTED BY
(PROJECT MANAGER'S NAME) LICENSE NO. DATE
(COMPANY OFFICER'S NAME) LICENSE NO. DATE



City of Grants Pass

SEDIMENTATION BASIN NO. 2
STRUCTURAL
TOP PLAN

SHEET
Fig 5
Job No.



WRAP
CONCRETE
COLUMNS
W/ FIBERGLASS
TYP. &
M
DETERIORATING
BEAMS

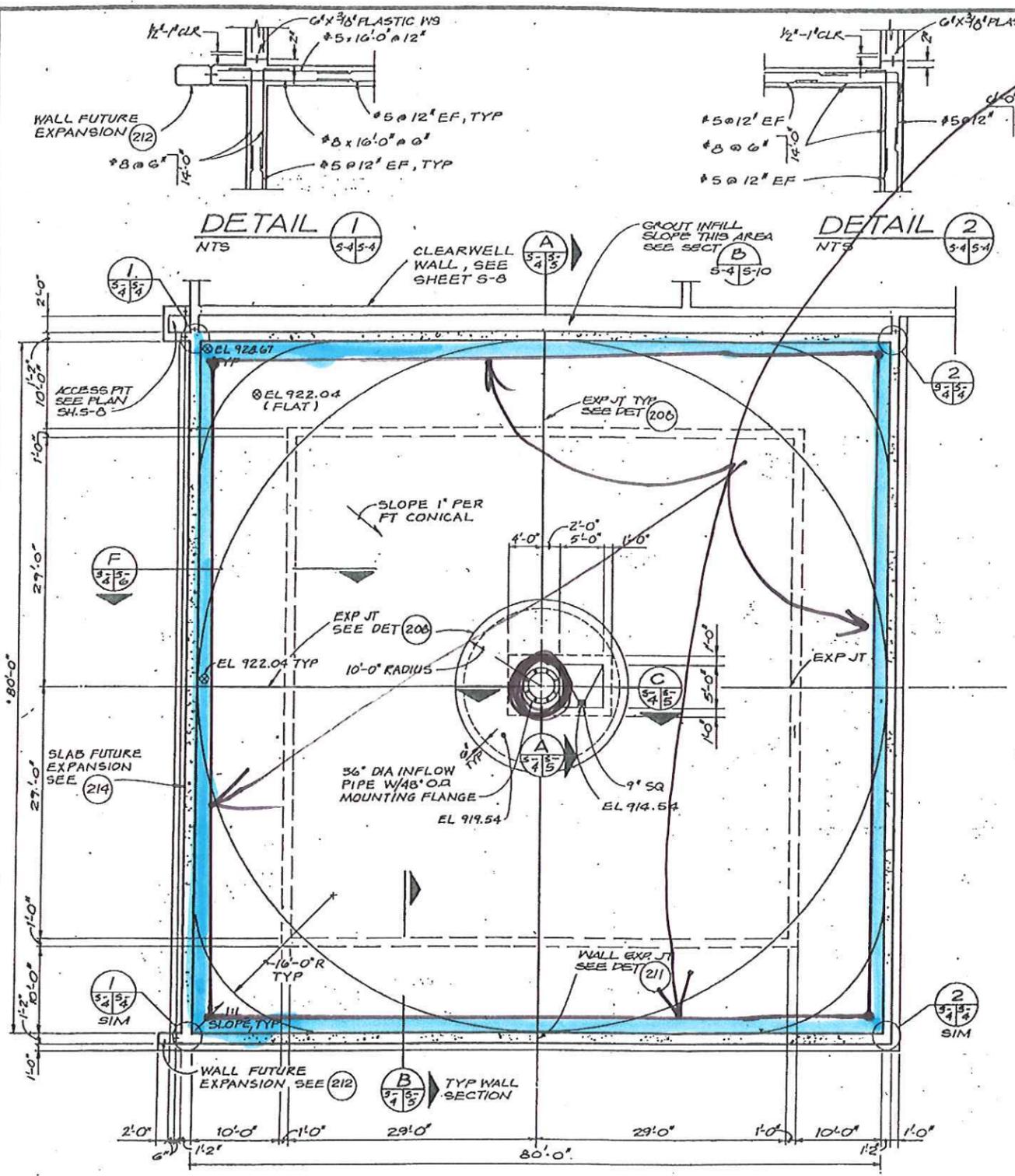
FIG 4.1.6a

BAAR AND CUNNINGHAM
CONSULTING ENGINEERS
SPALDING BLDG. PORTLAND, ORE.

CITY OF GRANTS PASS, OREGON.
WATER FILTRATION PLANT.
CLEAR WATER WELL
FLOOR AND FOOTINGS
AT HEAD HOUSE.

SCALE 1/2" = 1'-0"

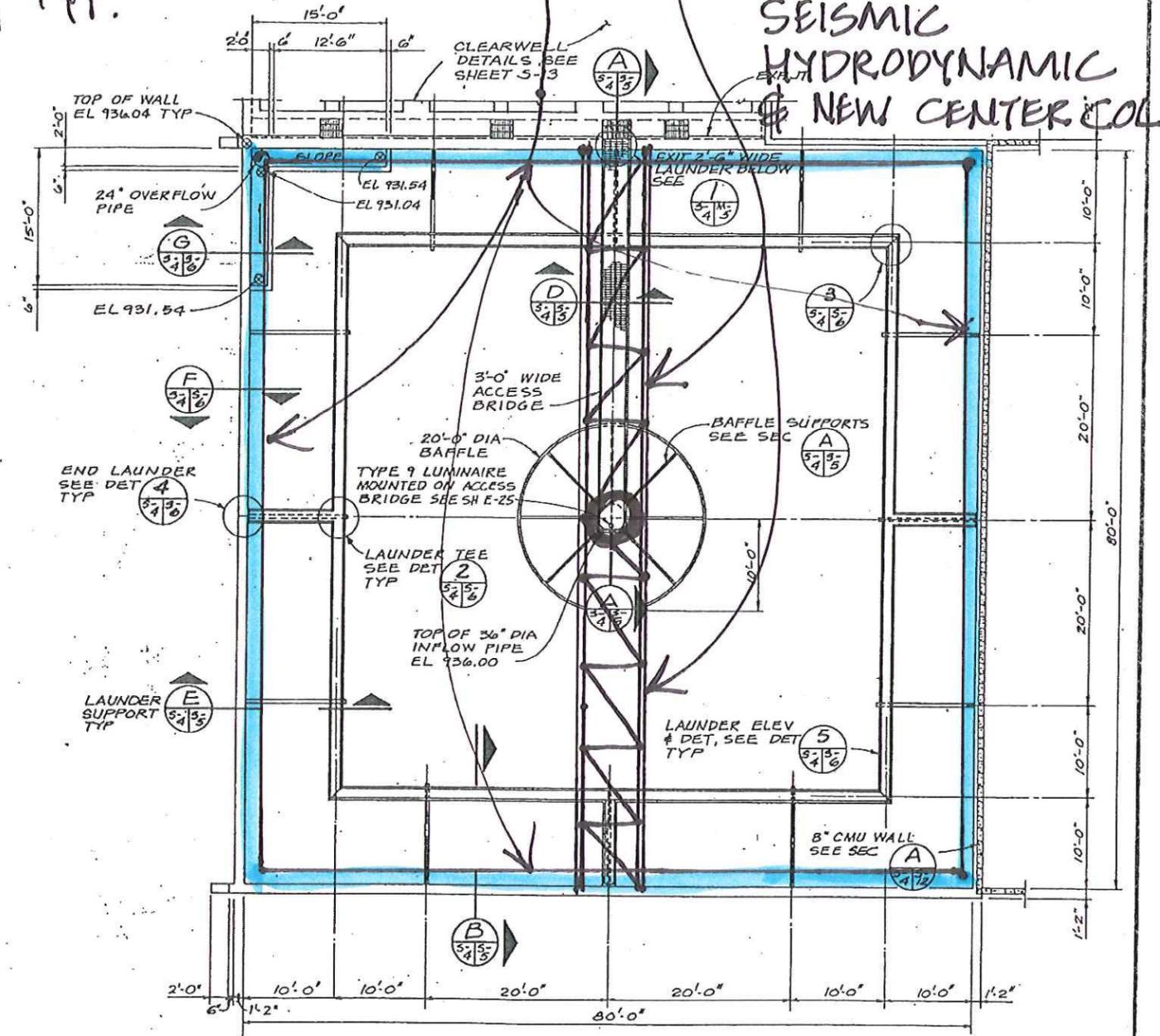
COMPUTATIONS	F. S. Z.	MARCH 6, 1931	NO. OF
DRAWN BY	F. S. Z.	REVISIONS:	FILE NO.
TRACED BY	C. E. R.	MARCH 25, 1931:	300-A-5
CHECKED BY	F. S. Z.		1148



FOUNDATION PLAN
1/8" = 1'-0"

NEW REINFORCED CONCRETE WALL STRENGTHENING TYP.

NEW CONTINUOUS BRIDGE TO TRANSFER SEISMIC HYDRODYNAMIC TO NEW CENTER COL.



PLAN ABOVE EL. 936.04
1/8" = 1'-0"



CH2M HILL	DES. RDV				
	DR. CRS				
	CHK. LJE				
	APPD. AEM	NO.	DATE	REVISION	BY

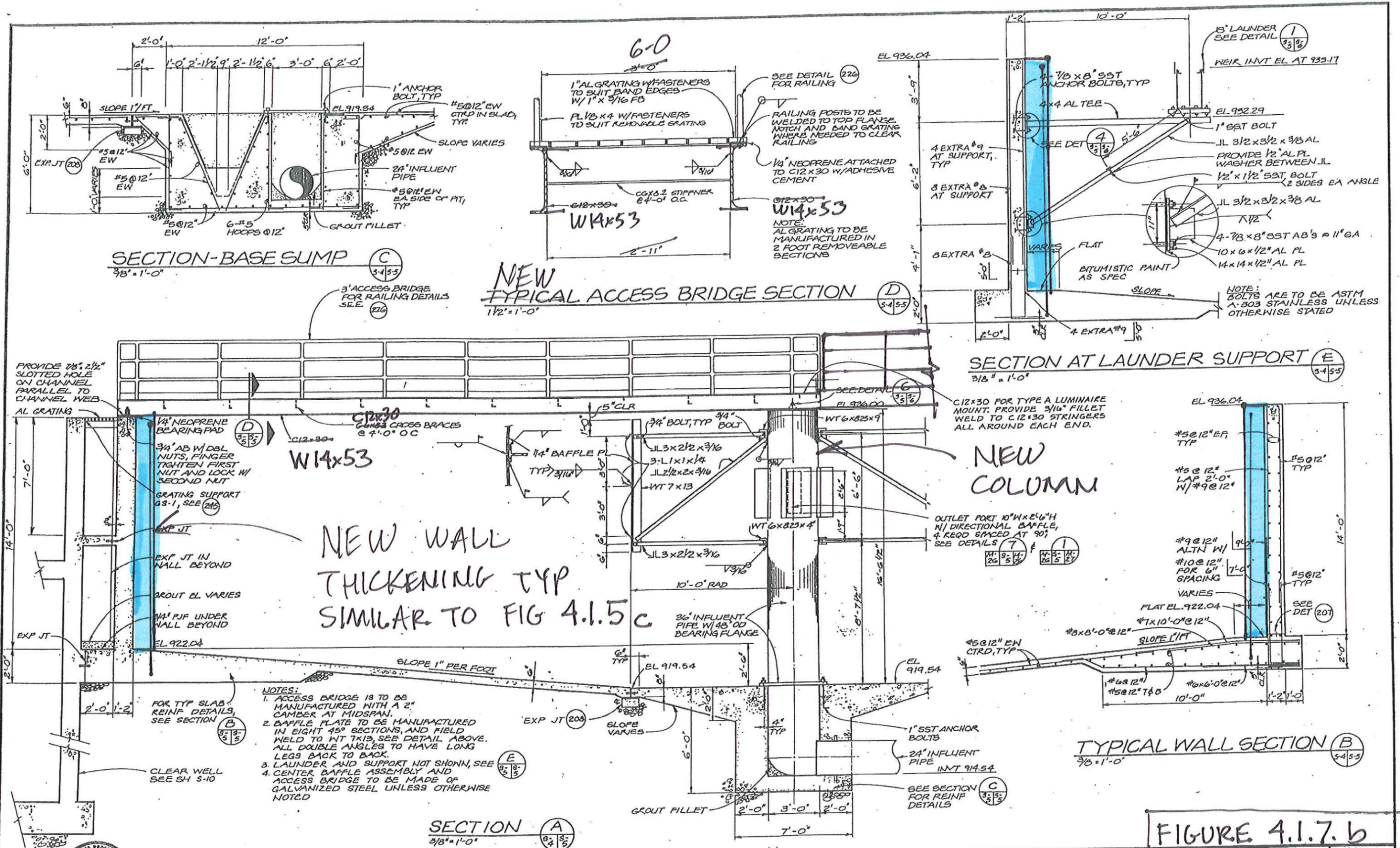
THIS PRINT IS REDUCED TO ONE-HALF OF THE ORIGINAL SCALE IF THE SCALE READS: 1" = 1'-0" USE 1/2" = 1'-0" OR 1" = 10' USE 1" = 20'

CITY OF GRANTS PASS, OREGON
WATER SYSTEM IMPROVEMENTS

STRUCTURAL
NEW SEDIMENTATION BASIN
PLANS AND DETAILS

SHL 31
DWG. NO. S-4
DATE: APR 1980
FILE NO. C13078.T1

FIGURE 4.17a



CH2M HILL	DES. RDV.				
	DR. MAY				
	CHK. L. JE				
APPD. AEM	NO.	DATE	REVISION	BY	APPO.

THIS PRINT IS REDUCED TO ONE-HALF OF THE ORIGINAL SCALE IF THE SCALE READS: 1" = 1'-0" USE 1/2" = 1'-0" OR 1" = 10' USE 1" = 20'

CITY OF GRANTS PASS, OREGON WATER SYSTEM IMPROVEMENTS

STRUCTURAL
NEW SEDIMENTATION BASIN
SECTIONS AND DETAILS

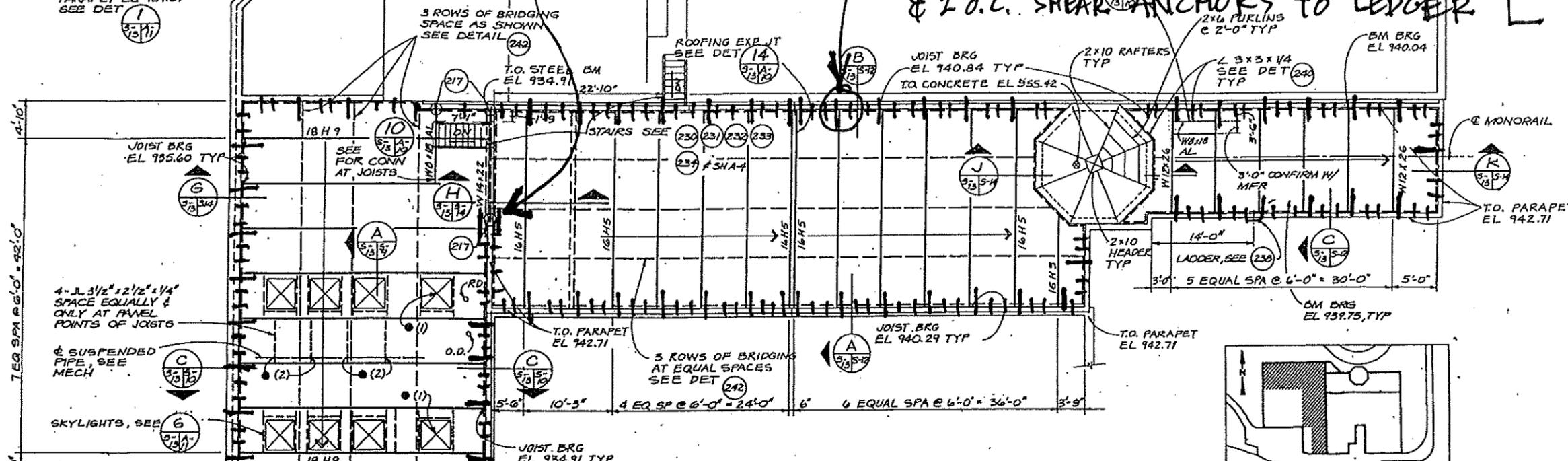
BH. 32
DWG. NO. S-5
DATE: APR 1980
FILE NO. C13078.T1



TOP OF MASONRY
PARAPET EL 931.57
SEE DET 1

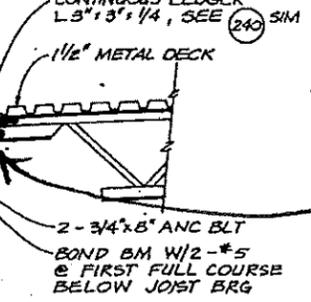
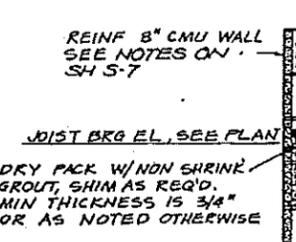
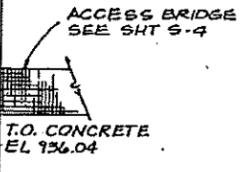
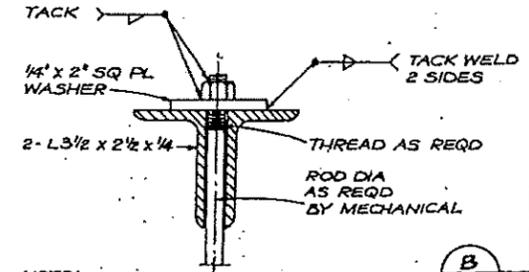
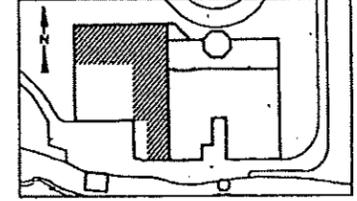
COLLECTOR
FIGURE 4.1.8c

ADDED WALL TO ROOF SHEAR TRANSFER &
TIES @ 6'-0" C, EPOXY ANCHORED THRU BOLTS
& 2'-0" C. SHEAR ANCHORS TO LEDGER L



ROOF FRAMING PLAN
1/8" = 1'-0"

- NOTES
1. SEE MECHANICAL DRAWINGS FOR PENETRATIONS NOT SHOWN
 2. SEE 6H 3-23 FOR GENERAL NOTES AND SH 5-7 FOR ROOF DESIGN NOTES



ADD L 3x4x1/4
SHEAR TRANSFER
L W/ EPOXY
ANCHORS
@ 2'-0" C.
CONT.

FIGURE 4.1.8a



CH2M HILL	DES. DEF				
	DR. CRS				
	CHK. LJE				
	APPD. AEM				
	NO.	DATE	REVISION	BY	APPD.

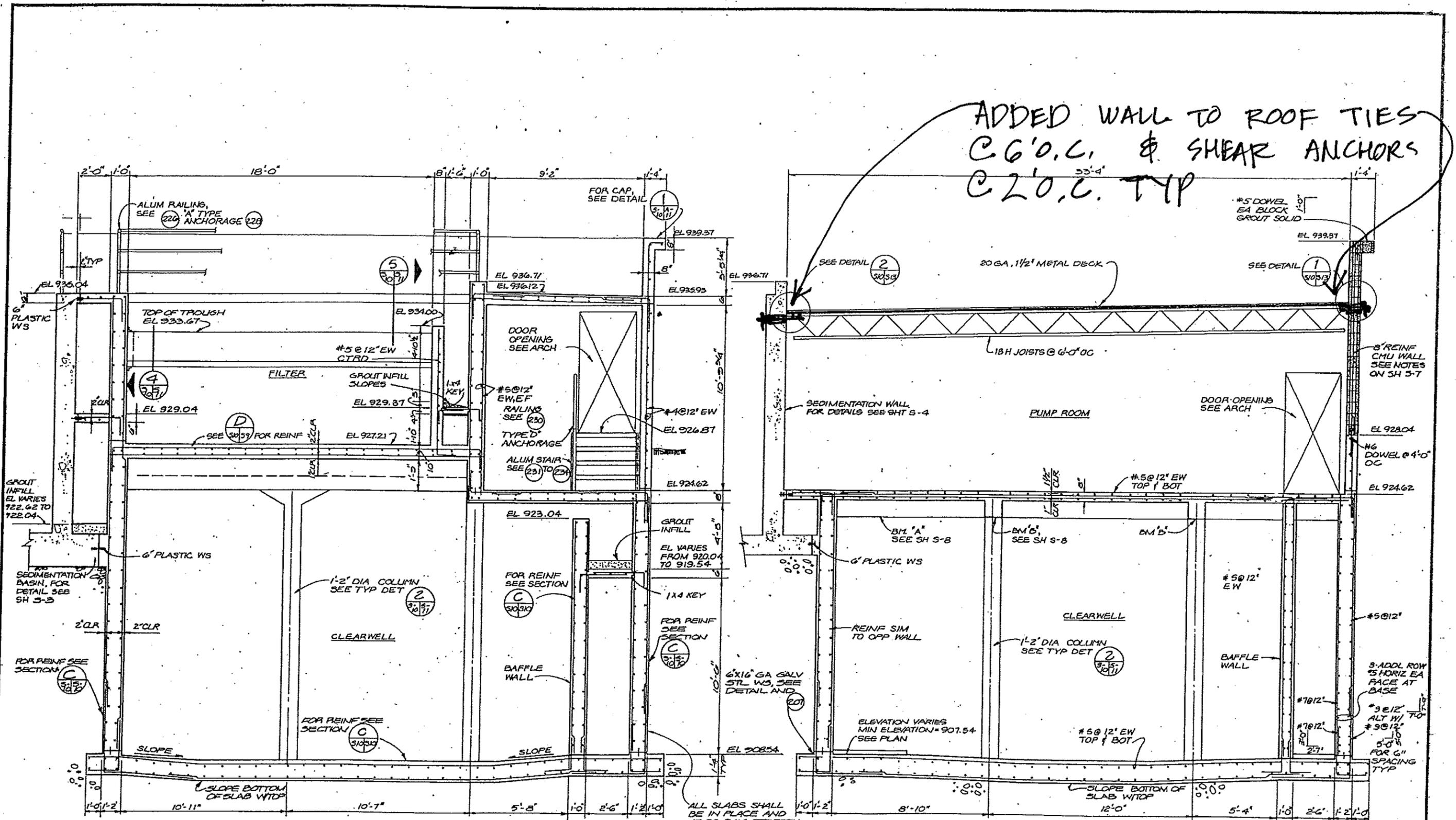
THIS PRINT IS REDUCED TO ONE-HALF OF THE ORIGINAL SCALE IF THE SCALE READS:
1" = 1'-0" USE 1/2" = 1'-0" OR 1" = 1'-0" USE 1" = 1'-0"

CITY OF GRANTS PASS, OREGON
WATER SYSTEM IMPROVEMENTS

STRUCTURAL
NEW BUILDING ADDITION
ROOF FRAMING PLAN AND DETAILS

SHEET NO.	40
DWG. NO.	S-13
DATE	APR 1980
FILE NO.	C1307&T1

ADDED WALL TO ROOF TIES
 @ 6' O.C. & SHEAR ANCHORS
 @ 2' O.C. TYP



DES. DEF					
DR. DLA					
CHK. LJE					
APPD. AEM					
NO.	DATE	REVISION	BY	APPD.	

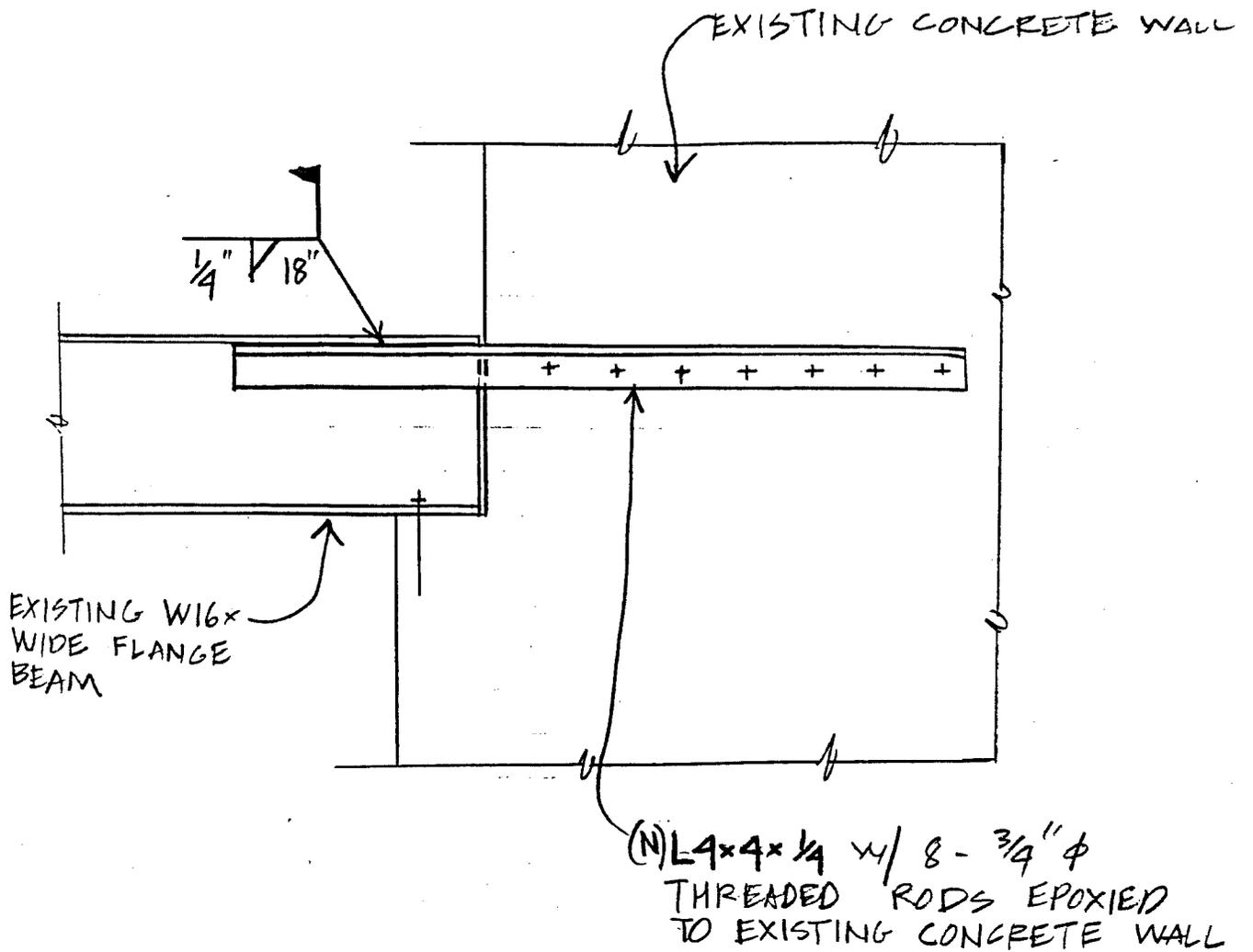
THIS PRINT IS REDUCED TO ONE-HALF OF THE ORIGINAL SCALE IF THE SCALE READS:
 1" = 1'-0" USE 1/2" = 1'-0" OR 1" = 10' USE 1" = 20'

CITY OF GRANTS PASS, OREGON
 WATER SYSTEM IMPROVEMENTS

STRUCTURAL
 FILTER, PUMP ROOM AND CLEARWELL
 SECTIONS

SHL	37
DWD. NO.	S-10
DATE	APR 1980
FILE NO.	C13076.T1

FIGURE 4.1.8b



ELEVATION

SECTION

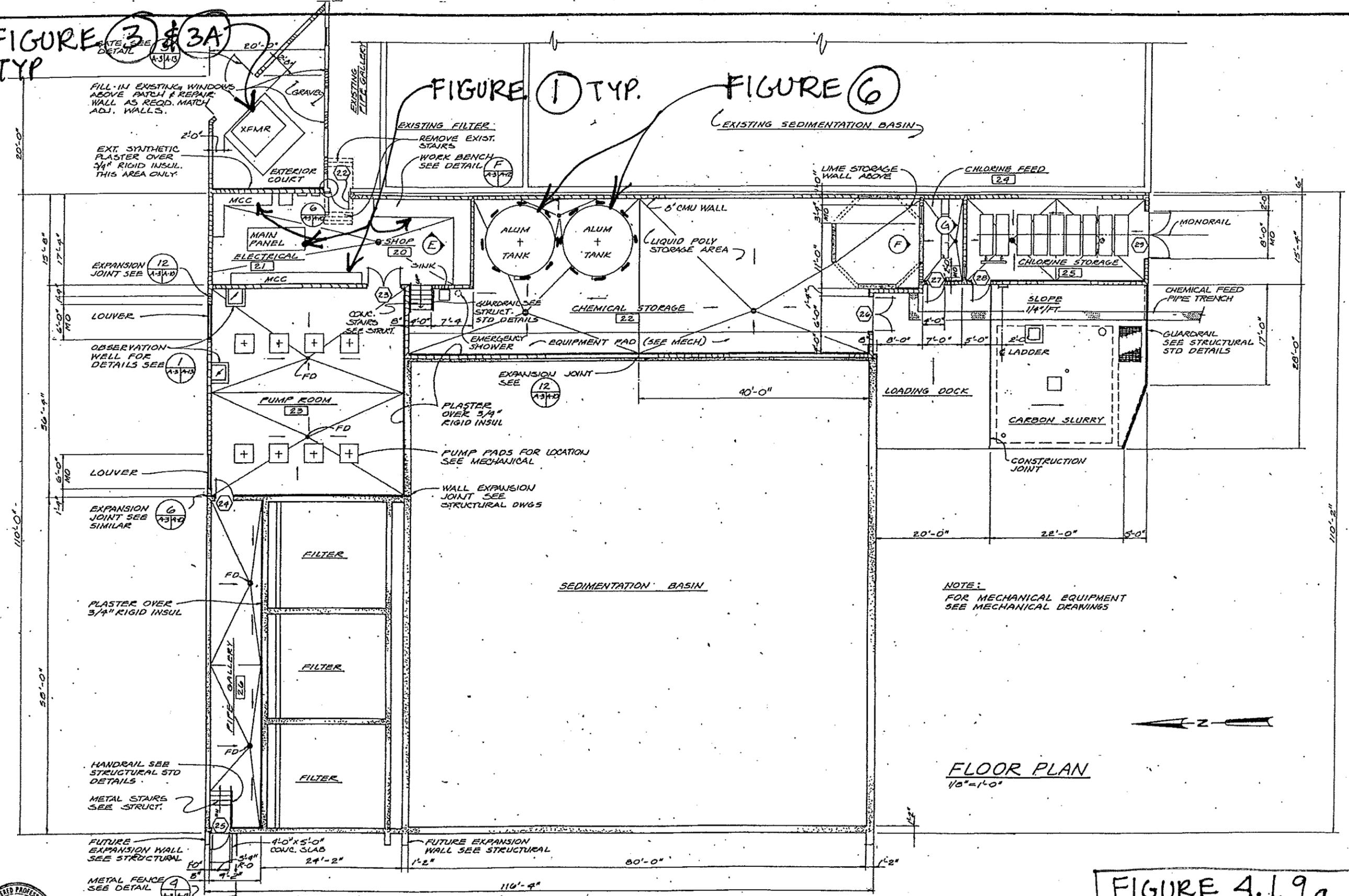
(4.1.8e)

FIGURE 4.1.8e

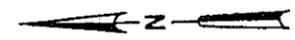
FIGURE 3 & 3A
TYP

FIGURE 1 TYP.

FIGURE 6



NOTE:
FOR MECHANICAL EQUIPMENT
SEE MECHANICAL DRAWINGS



FLOOR PLAN
1/8" = 1'-0"



CH2M HILL	DES. JWR				
	DR. TWM				
	CHK. JWR				
	APPD. AEM	NO.	DATE	REVISION	BY

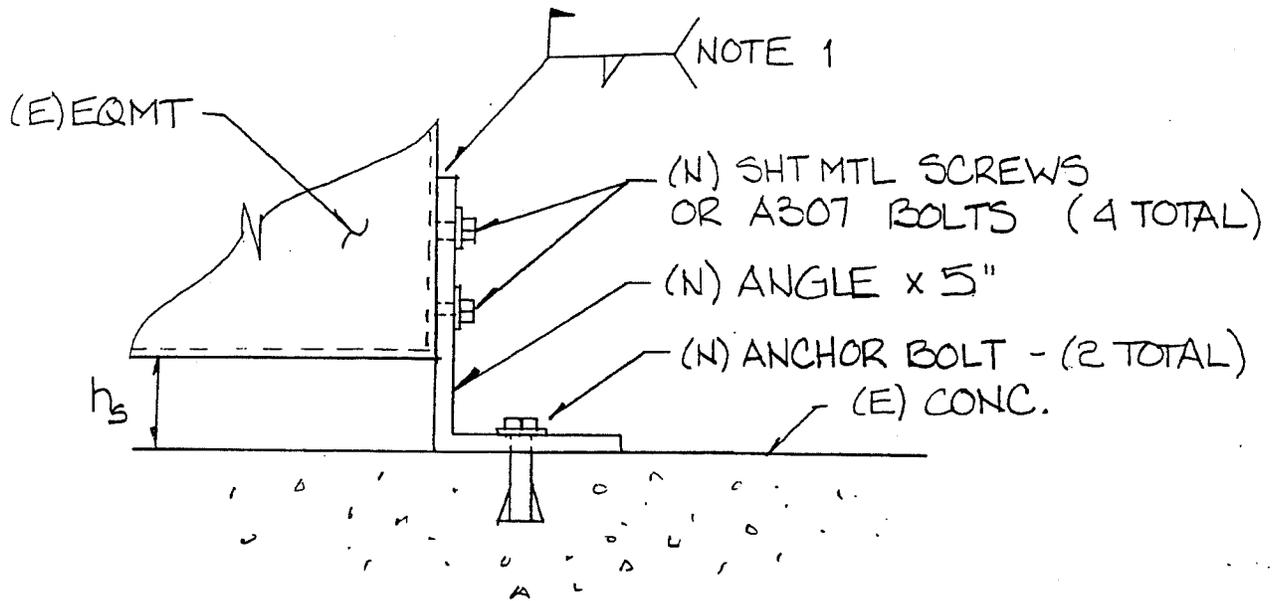
THIS PRINT IS REDUCED TO ONE-HALF
OF THE ORIGINAL SCALE
IF THE SCALE READS: 1" = 4'-0" USE 1/2" = 1'-0" OR 1" = 10' USE 1" = 20'

CITY OF GRANTS PASS, OREGON
WATER SYSTEM IMPROVEMENTS

FIGURE 4.1.9a

ARCHITECTURAL
NEW BUILDING ADDITION
GENERAL FLOOR PLAN

SH.	14
DWG. NO.	A-3
DATE	APR 1980
FILE NO.	C1307&T1

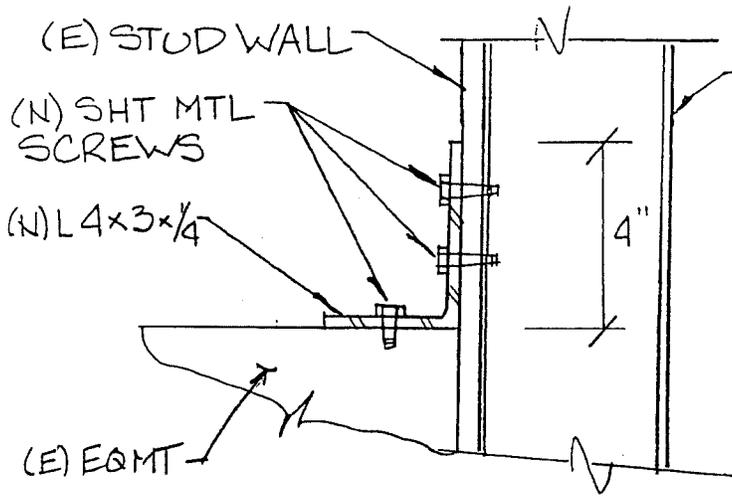


NOTES:

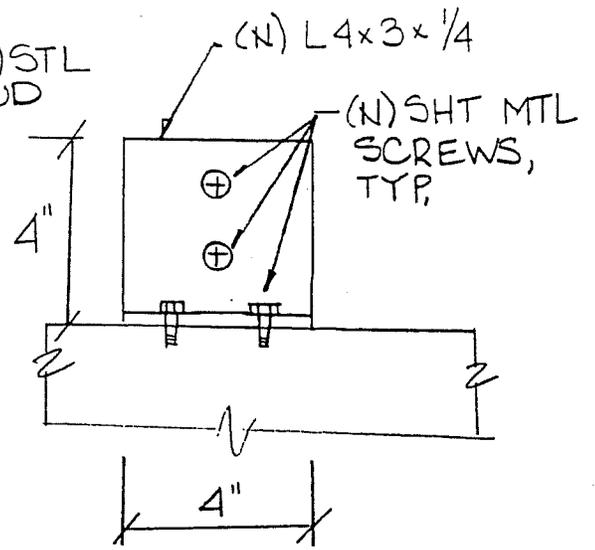
1. FIELD WELDING MAY BE USED AS AN ALTERNATE TO BOLTING. USE CONT. FILLET WELD ON ALL THREE SIDES.
2. h_s = HEIGHT OF EQMT BASE ABOVE CONCRETE.
3. CLIP MAY BE ROTATED FOR INSTALLATION INTO A CONC. WALL

① ANGLE RESTRAINT FOR EQMT CABINETS, ELECT CAB. & POLY TANK STAND FEET/COLUMNS

Figure 4.1.9b

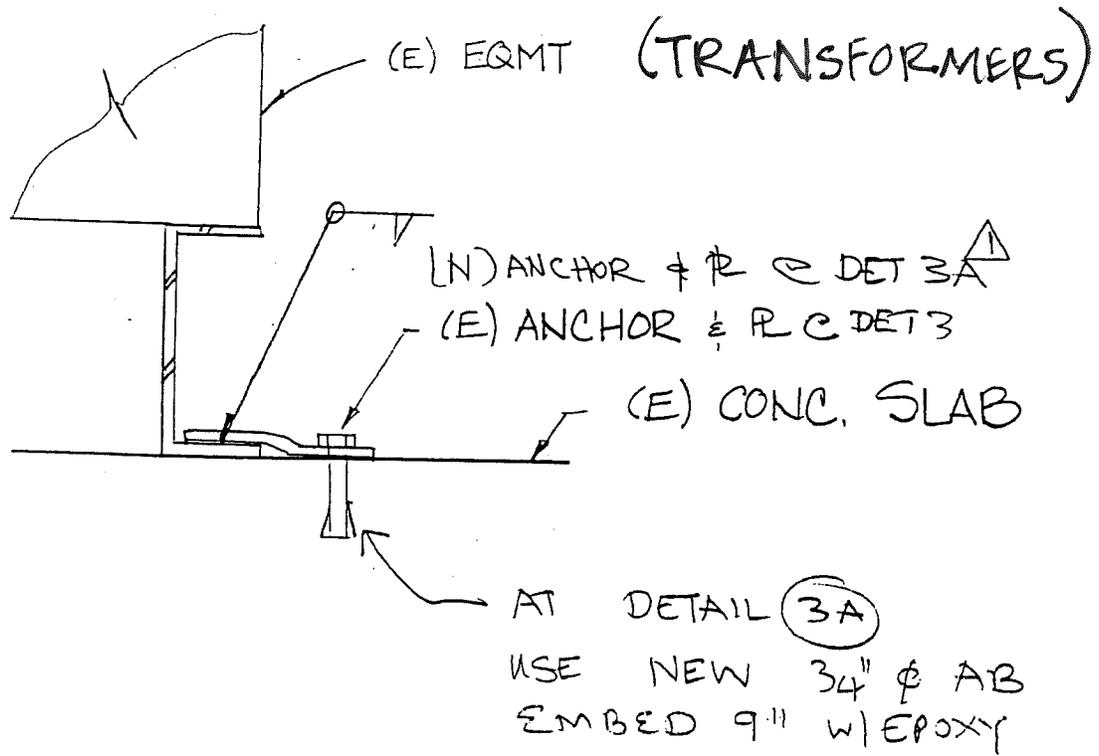


SECTION



ELEVATION

② ANGLE RESTRAINT FOR EQMT CABINETS



(3) CLIP PLATE RETROFIT DETAIL

(3A) NEW CLIP PLATE & NEW BOLT

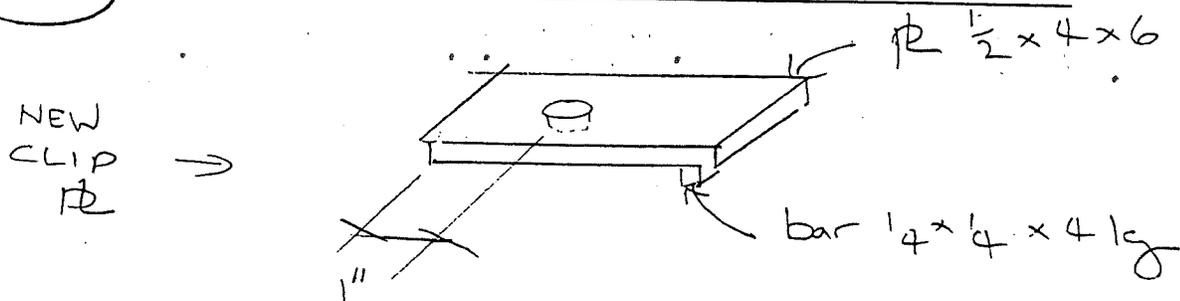
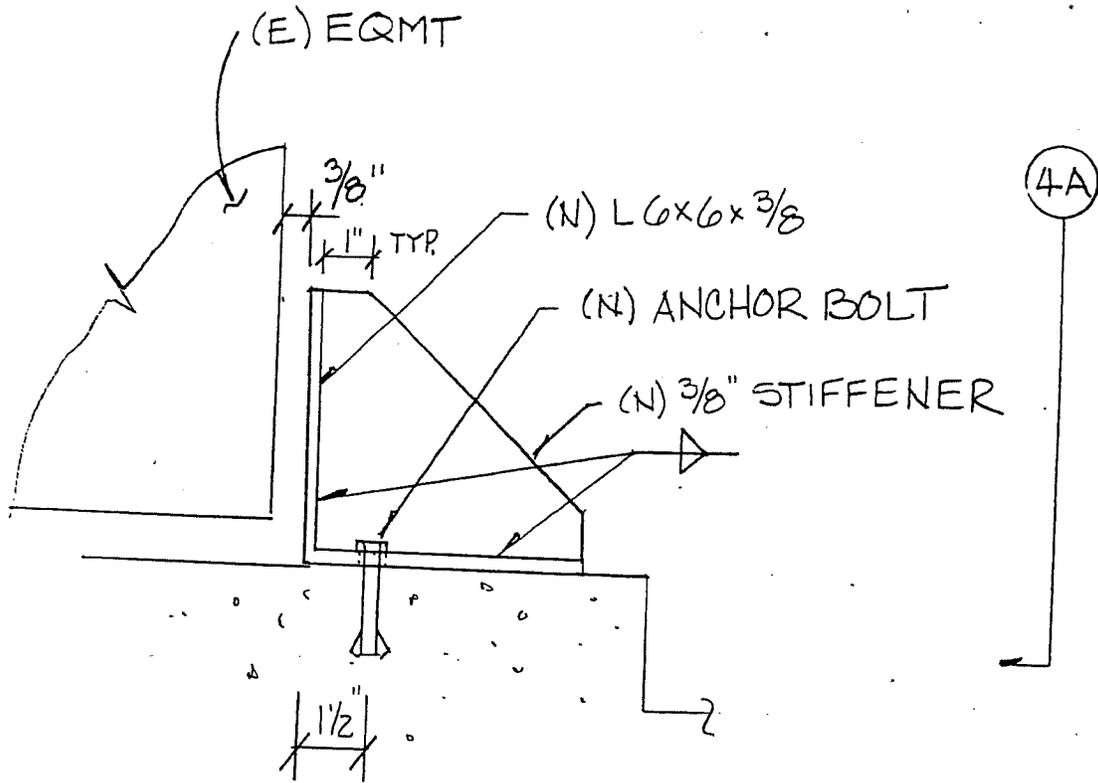


Figure 4.1.9d

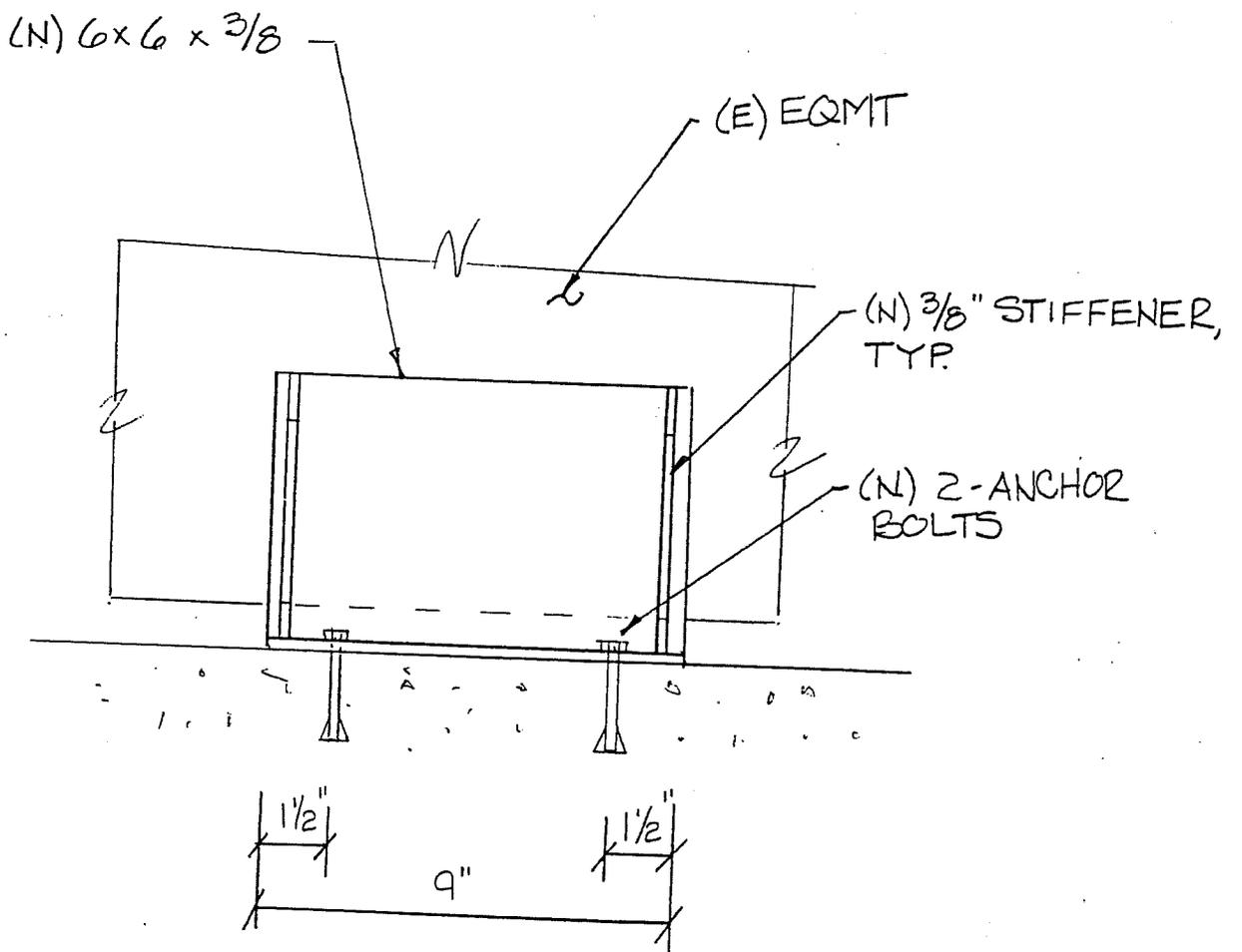


NOTE: STIFFENER PL'S NOT REQ'D FOR
EQMT WT. LESS THAN 5000#

④ SNUBBER DETAIL
3" = 1'-0"

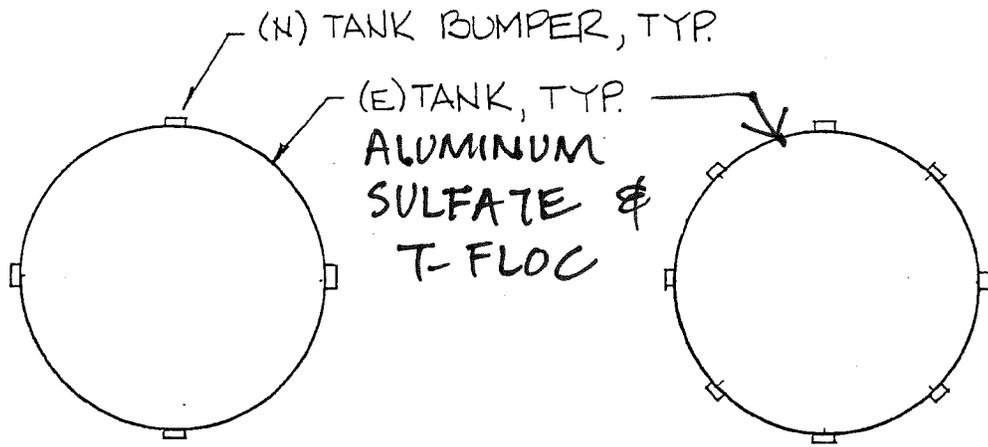
LIGHT-MEDIUM DUTY

Figure 4.1.9e



4A DETAIL
 3" = 1'-0"

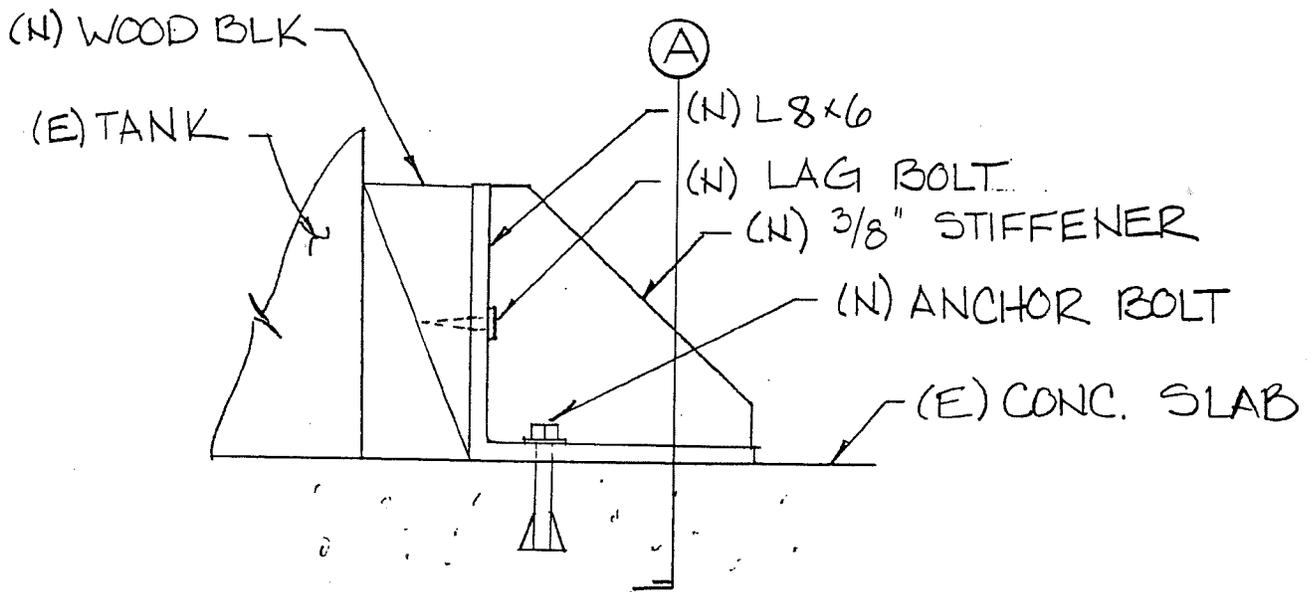
Figure 4.1.9f



5 4 BUMPER CONFIG.

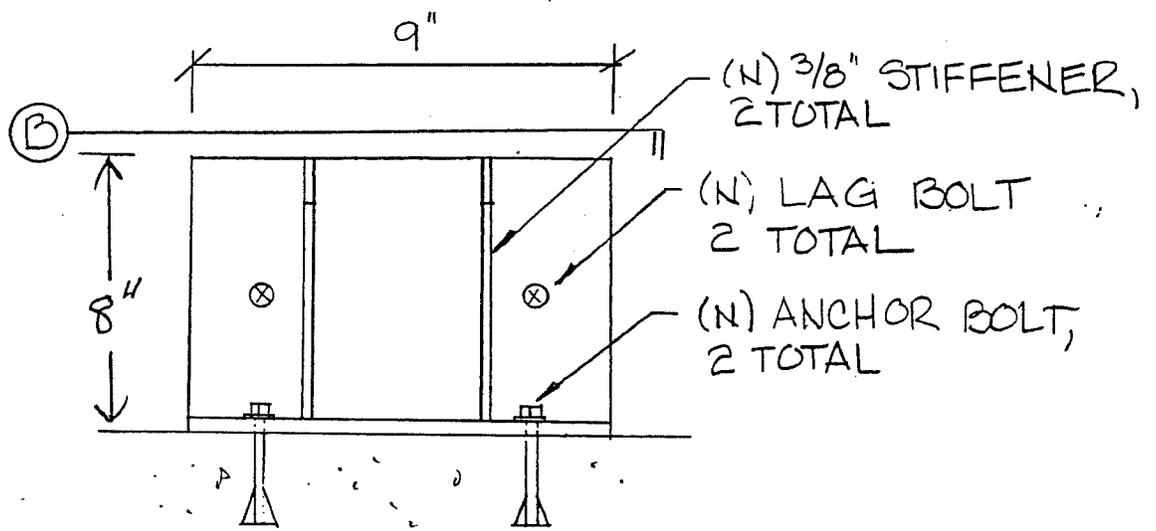
6 8 BUMPER CONFIG.

PLAN

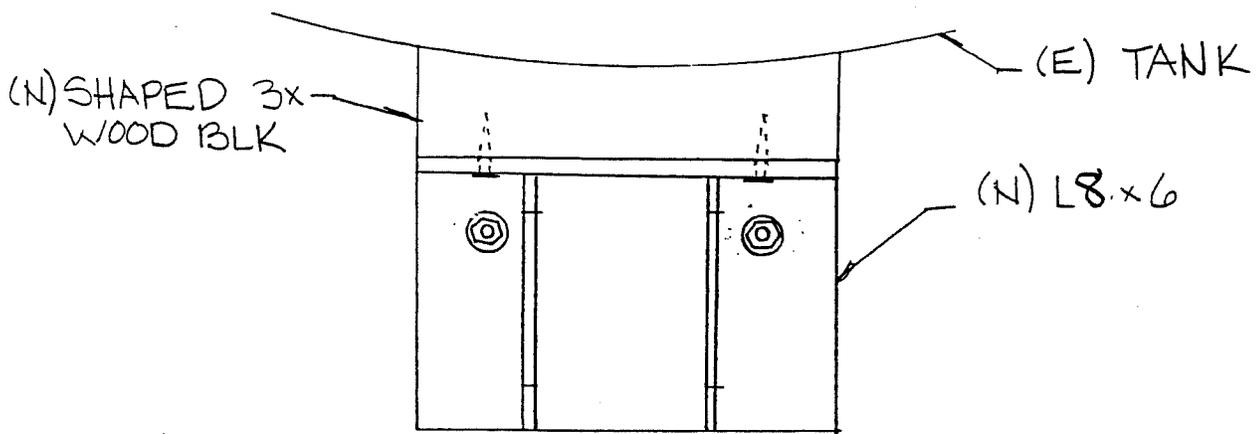


7 TANK BUMPER DETAIL

Figure 4.1.9g

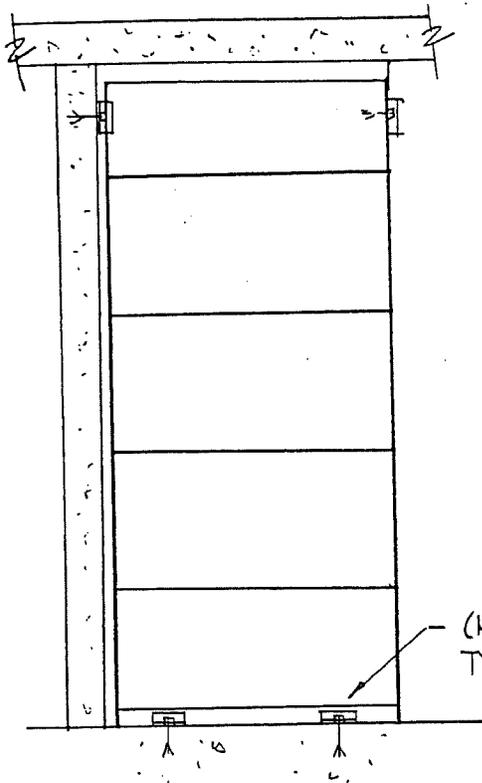


(7A) TANK BUMPER SECTION

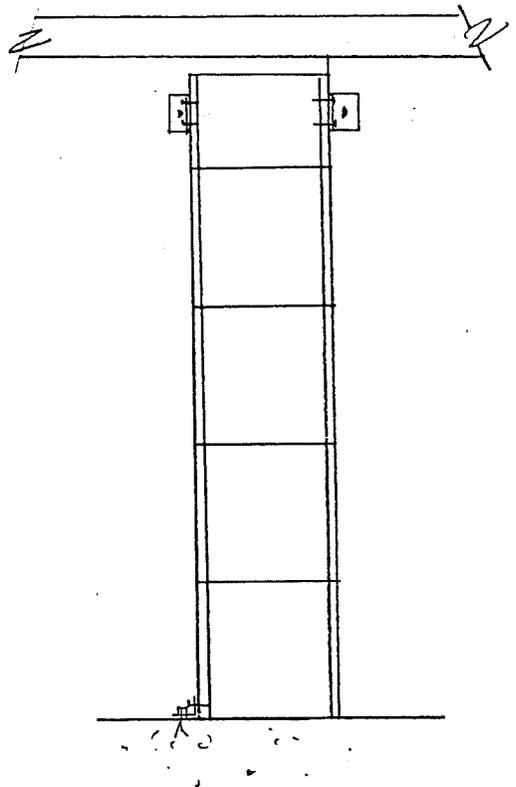


(7B) TANK BUMPER PLAN

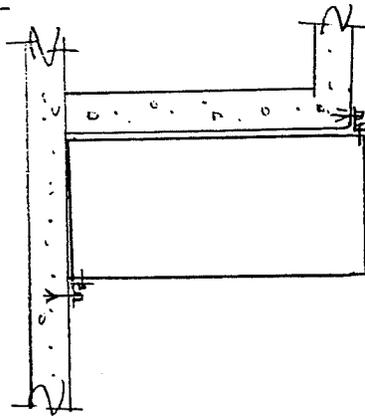
Figure 4.1.9h



ELEVATION



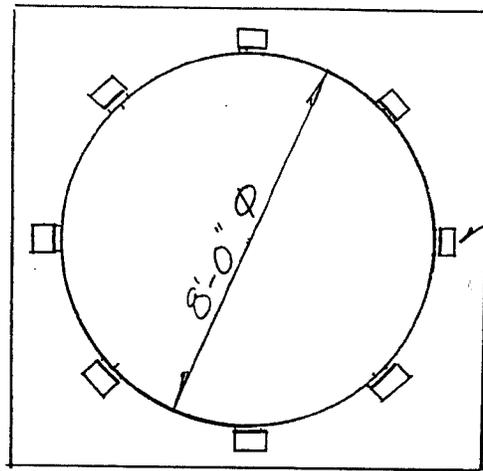
SECTION



PLAN

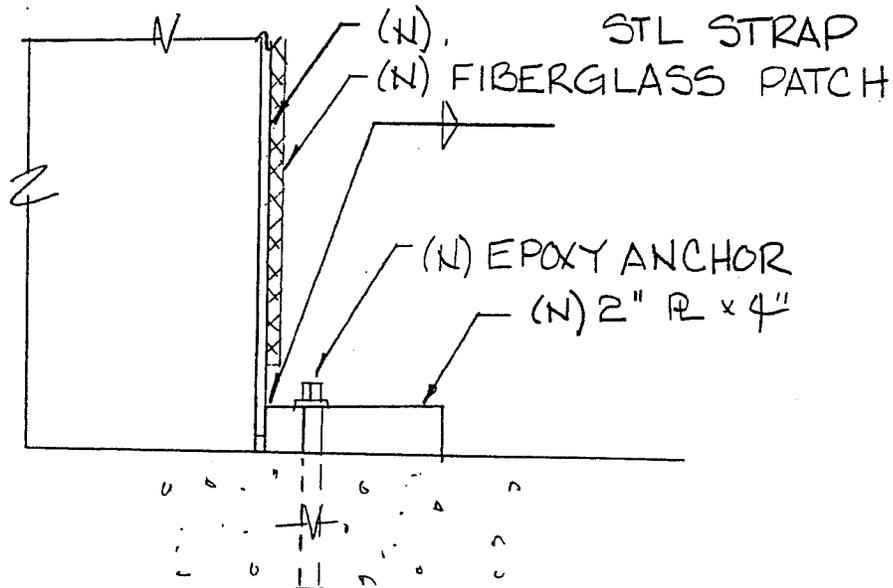
⑧ STEEL SHELF BRACING DETAIL & EQUIP
 $\frac{1}{2}'' = 1'-0''$

Figure 4.1.9j



(N) 1/4" x 4" STL STRAP & BASE R ANGLE

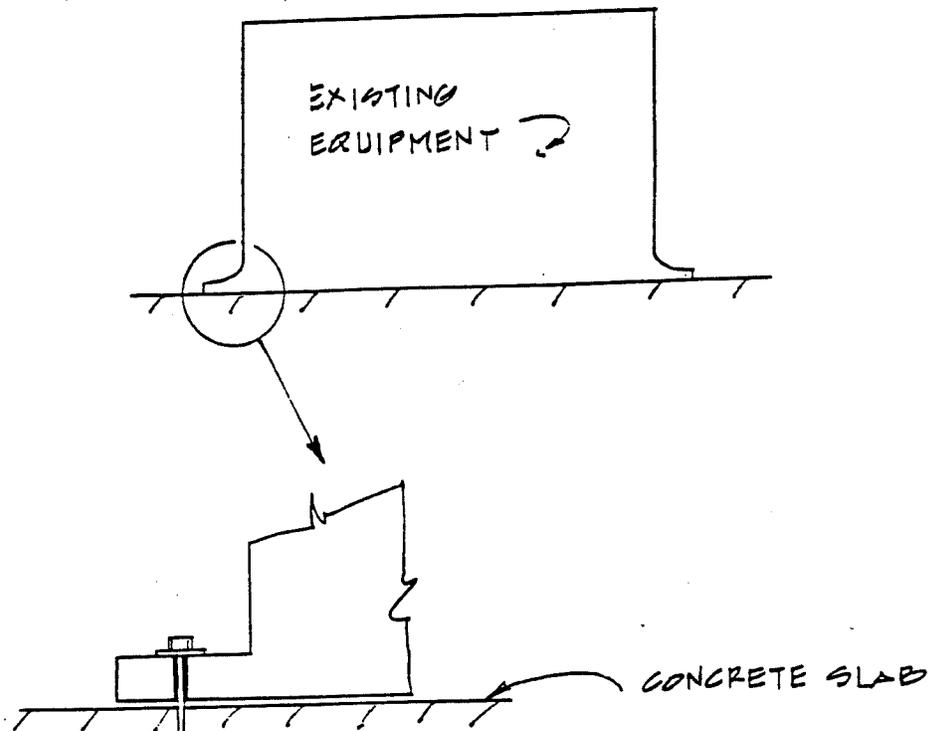
9 A TANK PLAN
1/4" = 1'-0"



RK

9 B BASE R ANGLE DETAIL
1/2" = 1'-0"

Figure 4.1.9k



CHECK WORKMANSHIP IN EXISTING ANCHORS.
 INSTALL MATCHING KWIK-BOLTS WHERE
 ANCHORS ARE MISSING.
 FOUR BOLTS MINIMUM (1 AT EACH
 CORNER) ARE REQUIRED FOR EACH
 PIECE OF EQUIPMENT. ADDITIONAL
 ANCHORS MAY BE REQUIRED AS
 SHOWN IN STRENGTHENING DETAILS

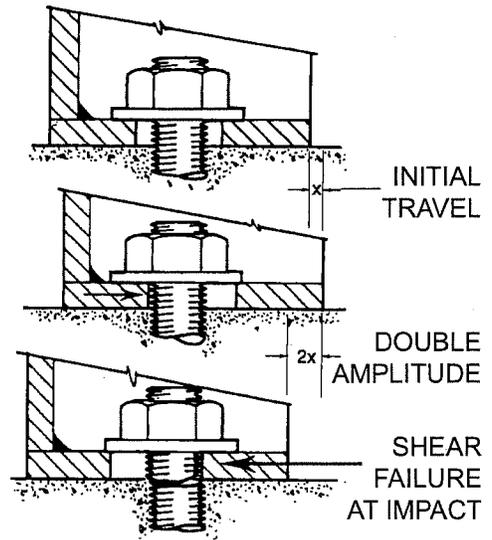
FIGURE 10

Figure 4.1: 9.2

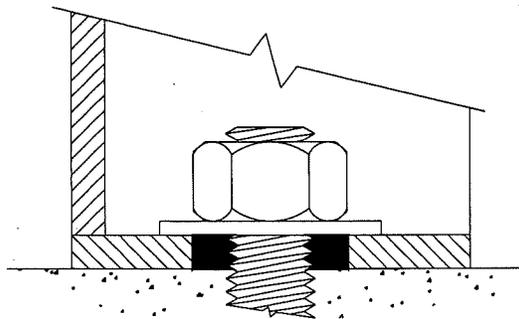
FLOOR MOUNTED EQUIPMENT

When there is excessive clearance between anchor bolts and equipment holes, the equipment has a tendency to shear off the anchor bolts during earthquakes or bomb blasts at accelerations as low as 0.2 G. The reason as explained in the figure is a velocity buildup because of sliding. What was initially analyzed as a static system becomes dynamic.

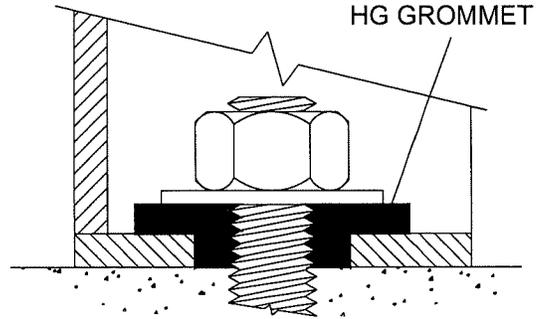
Type HG [FM49] neoprene hole grommets and Type HCF hole clearance filler provide quick solutions as they fill this clearance created by practical tolerances, off center bolts or the extreme situation where holes are enlarged on the jobsite by drilling or burning. HG grommets surround the bolt shaft and help deaccelerate the system.



IMPACT SHEAR FAILURE



**HCF REMEDY
(Hole Clearance Filler)**



**HG REMEDY [FM49]
(Neoprene Hole Grommet)**

Figure 4.1.9.m



MASON INDUSTRIES, INC.

NY Mailing Address - P.O. Box 410 • Smithtown, NY 11787

350 Rabro Drive • Hauppauge, NY 11788 • 631/348-0282 • FAX 631/348-0279
2101 W. Crescent Ave., Suite D • Anaheim, CA 92801 • 714/535-2727 • FAX 714/535-5738
Email Addresses - Info@Mason-Ind.com • Info@MasonAnaheim.com

Page

10a

TRANSVERSE SEISMIC SOLID BRACE GUIDELINES FOR ROUND PIPE

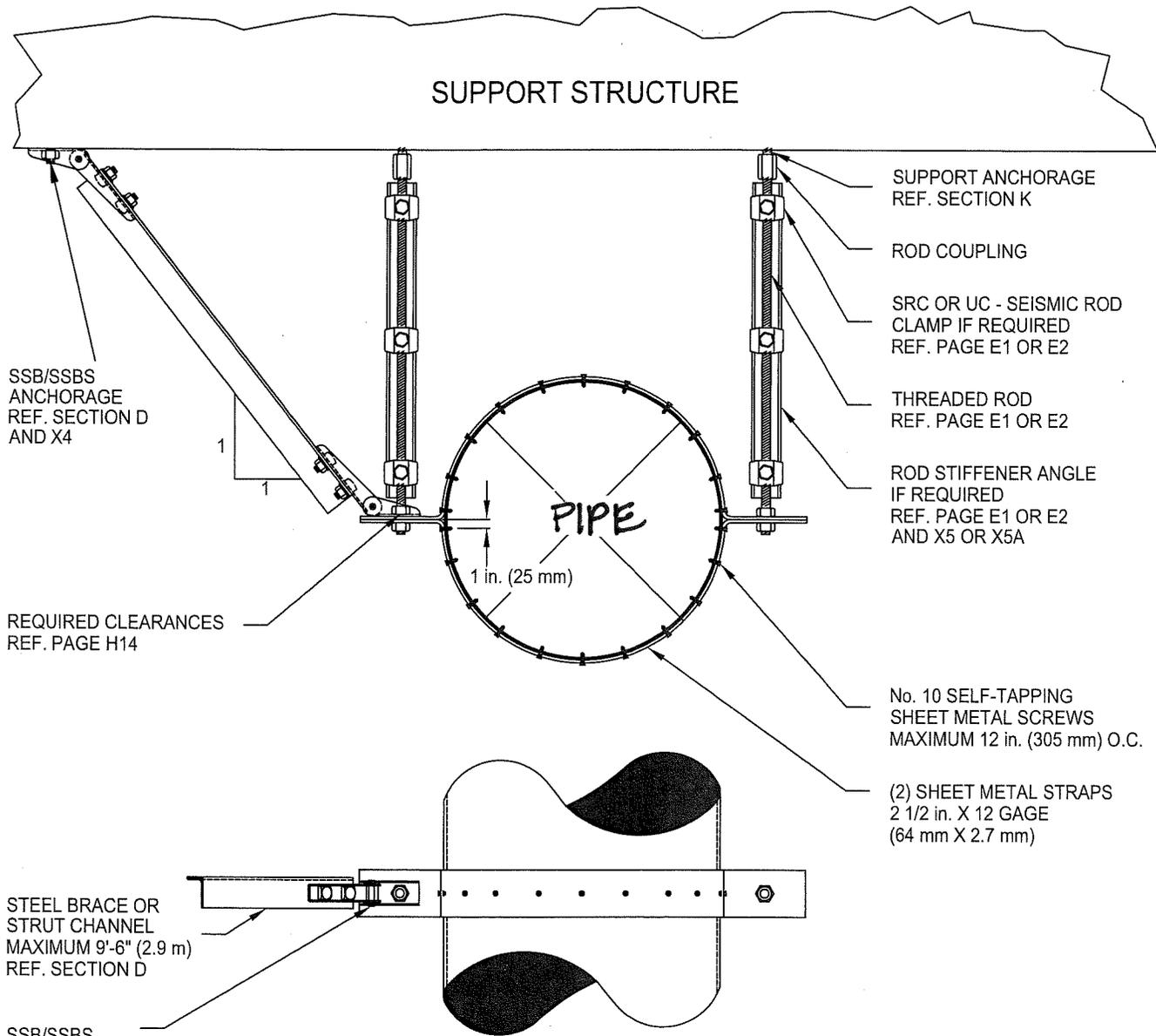


FIGURE 4.11a

NOTE 1: A ROD STIFFENER ANGLE MAY BE REQUIRED AS SHOWN. FOR ADDITIONAL INFORMATION, REF. PAGE E1 OR E2. BRACE ANGLE RATIO MAY BE INCREASED TO 2(VERT.): 1(HORIZ.). REFER TO SECTION D FOR LIMITATIONS.
 NOTE 2: FOR TIGHTENING REQUIREMENTS OF BOLTS, NUTS AND STRUT NUTS REFERENCE H15.

MASON INDUSTRIES, Inc.
 Manufacturers of Vibration Control Products
 NY Mailing Address: PO Box 410, Smithtown, NY 11787

350 Rabro Drive Hauppauge, NY 11788 631/348-0282 FAX 631/348-0279 Info@Mason-Ind.com	2101 W. Crescent Ave., Suite D Anaheim, CA 92801 714/535-2727 FAX 714/535-5738 Info@MasonAnaheim.com
--	--

APPROVED

California Office of Statewide
 Health Planning and Development

FIXED EQUIPMENT ANCHORAGE

OPA-0349 August 5, 2002

Dhiru Mali
 Dhiru Mali
 Structural Engineer
 California SE No. 2811

Bill Staehlin
 Bill Staehlin (916) 654-3362

Page

LONGITUDINAL SEISMIC SOLID BRACE GUIDELINES FOR ROUND PIPE

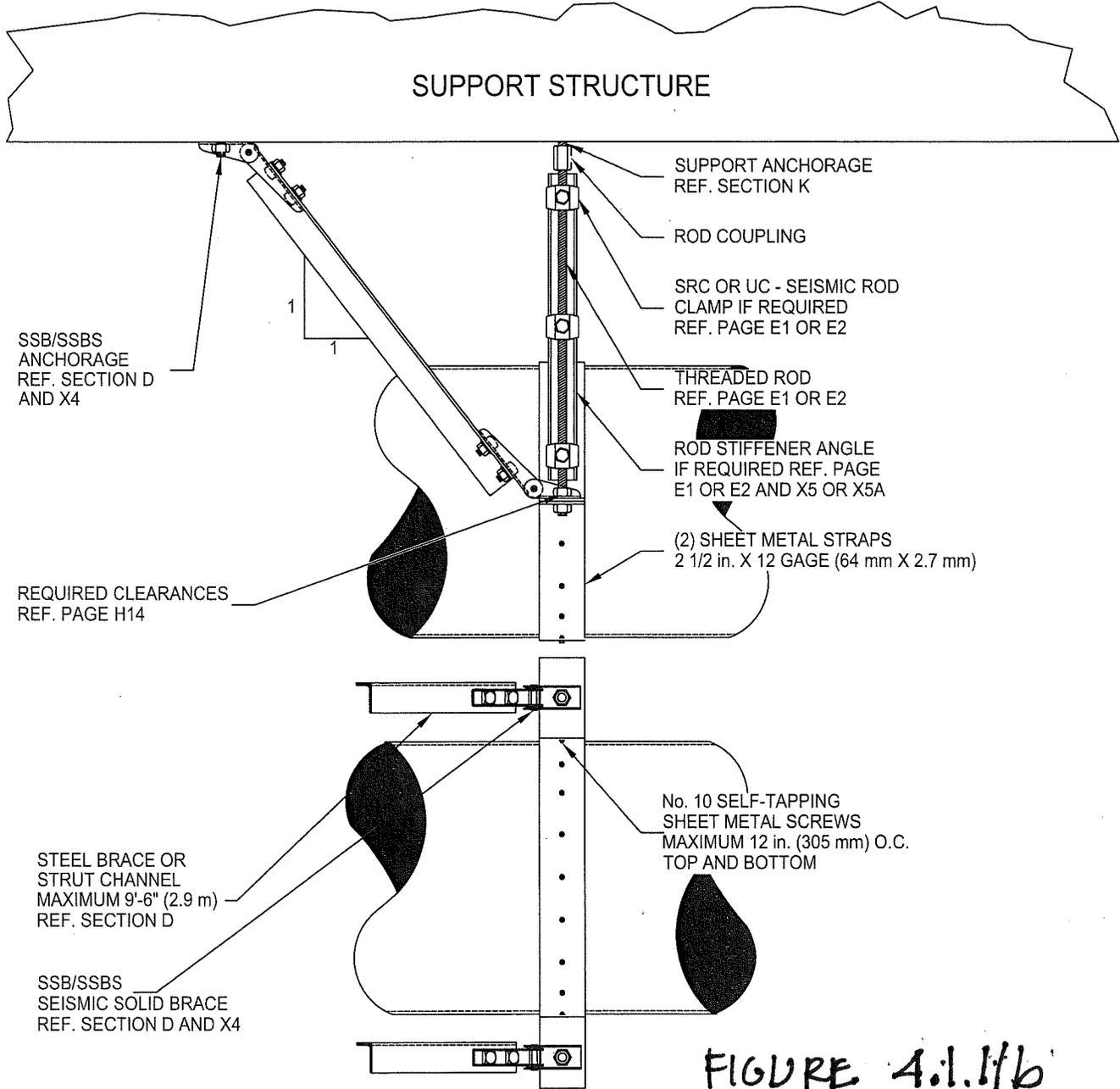


FIGURE 4.1.17b

NOTE 1: A ROD STIFFENER ANGLE MAY BE REQUIRED AS SHOWN. FOR ADDITIONAL INFORMATION, REF. PAGE E1 OR E2.
BRACE ANGLE RATIO MAY BE INCREASED TO 2(VERT.) : 1(HORIZ.). REFER TO SECTION D FOR LIMITATIONS.
NOTE 2: FOR TIGHTENING REQUIREMENTS OF BOLTS, NUTS AND STRUT NUTS REFERENCE H15.

APPROVED
California Office of Statewide
Health Planning and Development
FIXED EQUIPMENT ANCHORAGE
OPA-0349 August 5, 2002



Bill Staehlin
Bill Staehlin (916) 654-3362

MASON INDUSTRIES, Inc.
Manufacturers of Vibration Control Products
NY Mailing Address: PO Box 410, Smithtown, NY 11787

350 Rabro Drive Hauppauge, NY 11788 631/348-0282 FAX 631/348-0279 Info@Mason-Ind.com	2101 W. Crescent Ave., Suite D Anaheim, CA 92801 714/535-2727 FAX 714/535-5738 Info@MasonAnaheim.com
--	--

<i>Dhiru Mali</i> Dhiru Mali Structural Engineer California SE No. 2811	Page
--	------

Attachment A

Building Performance Data

A. Building Performance Data

A.1 Building Codes

Most building codes in use in the United States are based on one of three model building codes: the *National Building Code (NBC)*, *Standard Building Code (SBC)* and *Uniform Building Code (UBC)* which has been more recently combined into the *American Society of Engineers (ASCE) 7* and the *International Building Code (IBC)*. These codes are developed and published by industry associations representing building officials. The wind and seismic design provisions contained in these model codes are all similar. In the most wind active regions of the United States, including Florida, the Gulf Coast and Atlantic Coast states design has been based on the ASCE 7, for many years. In other regions of the nation, the other model building codes are commonly used. In many localities in these other regions, the wind and seismic provisions of these codes are not enforced.

The ASCE and IBC have adopted wind and seismic design philosophy intended to protect life safety, but allow for some structural and potentially significant non-structural damage for wind and seismic levels as severe as can be expected at some sites in the most active wind regions of the nation.

The lateral force design regulations of the ASCE and IBC primarily address requirements for building components and connections subjected to strong wind and seismic loading. The design regulations do not address the effects of large wind borne debris at a site. Projectiles can result in excessive damage to a building, regardless of how well the building is constructed.

The intent of the wind and seismic design provisions of the ASCE and IBC is to prevent injuries and loss of life under the conditions discussed above, not necessarily to minimize property damage. Over the years, a few buildings have been designed to provide for better performance. Such buildings are designed to substantially exceed the detailing, strength and stiffness requirements specified by the building code as a minimum design basis. In some jurisdictions, schools, hospitals, fire and police stations

and communications centers are required to be designed to such enhanced criteria. However, most buildings are designed for the code minimums.

A.2 Building Lateral Load Resisting Systems

Buildings are designed to resist wind and earthquake loads through the provision of a lateral-load-resisting system. The lateral-load-resisting system of a building typically consists of a combination of vertical and horizontal elements and their connections. Typical vertical elements are frames (beams and columns), braces, and walls. Typical horizontal elements are roofs, floors, and braces. Horizontal elements are usually termed diaphragms. Details of how these elements are constructed and interconnected are critical to the seismic performance of the building.

Beams and columns with connections designed to resist bending and shear forces are termed moment-resisting frames. Typically such frames are constructed of either monolithically placed reinforced concrete or structural steel. Recent codes require special detailing for these structures to ensure that they can behave in a ductile manner (able to absorb substantial amounts of energy and sustain extensive distress before collapse).

Walls designed to resist lateral loads are referred to as shear walls. Shear walls in buildings are composed either of reinforced concrete, reinforced or unreinforced masonry (concrete block or brick), or wood stud walls sheathed with plywood or gypsum board, or covered with stucco or plaster. In general, buildings with moment-resisting frames are more flexible than shear wall buildings and experience larger deformations in wind events or earthquakes. This commonly results in additional damage to architectural finishes.

When a building moves back and forth, in response to wind or ground motion, inertial lateral forces are generated at each point in the structure. Lateral loads generated within the roof and floors must be transferred to the vertical lateral-load-resisting elements. Typically these loads are transferred to the vertical elements either by horizontal braces or by roof or floor diaphragms. (Diaphragms can be viewed as deep beams turned on their sides that resist horizontal loads by spanning between the vertical

lateral-load-resisting elements.) Diaphragms can consist of plywood nailed to the floor framing, a reinforced concrete slab, or reinforced concrete over a metal deck.

A.3 Seismic Performance Data for Structures and Buildings

Most buildings can be classified as conforming to one of several model building types. Model building types are categorized by the materials of construction and type of lateral-load-resisting system employed. Observation of the performance of hundreds of buildings in past wind and earthquake events demonstrates that buildings conforming to certain model building types often share common vulnerabilities and are subject to similar damage. Knowledge of this common performance is a key factor in the evaluation of a building's lateral load risk. This section provides summary information on braced steel frame, concrete shear wall and concrete frame buildings.

A.3.1 Braced Steel Frame Building Discussion:

Braced steel frames are commonly used in mid- and low-rise steel construction. These structures derive their lateral strength through the presence of cross-bracing between their beams and columns. These braces resist lateral forces primarily through tension and compression. Often these buildings are recognizable by the presence of the braces, which may be visible across windows or within interior spaces of the building. Braced frame structures tend to be much stiffer than moment-resisting frames, another common form of structural steel construction, and therefore, are often used in combination with buildings of that type to reduce earthquake and wind induced swaying. The rigidity inherent in this system tends to minimize the amount of damage experienced by architectural elements.

There are three common systems of braced frames. These are concentric braced frames, special concentric braced frames, and eccentric braced frames. Concentric braced frames are the oldest form of this system. A number of patterns of concentric braced frames are common. One pattern is diagonal "X" bracing in which the braces extend directly between opposite beam-column joints in the frame. This system is often considered architecturally undesirable because the braces prevent access through the braced bay and obscure window space. A more common system is the so called chevron bracing, in which braces are arranged in a "V" or inverted "V" pattern extending

from adjacent beam-column connections to the middle of the beam above, or below. A third pattern is called knee or “K” bracing, in which the braces extend to the middle of a column.

Of these systems, “X” bracing has performed best in past earthquakes. These braces will commonly yield and buckle in strong ground shaking, and may even fracture, however such damage is easily repaired. If the braces are not adequately connected to the beams and columns, the connections themselves may fail, at premature load levels. Modern building code provisions require that connections be designed stronger than the braces to prevent such failures. Chevron braced frames have not performed as well as “X” braced frames. In chevron frames, the beams that support the apex of the “V” are commonly damaged, resulting in more difficult repair. “K” braced frames have performed poorly in past earthquakes because they induce large stresses on the critical column elements. “K” braced systems are not permitted for earthquake load resistance in modern codes.

A.3.2 Moment-Resisting Concrete-Frame Buildings Discussion:

Concrete frames consist of reinforced concrete beams and columns and their connections. Unreinforced concrete is weak in tension and is brittle. Consequently, reinforcing steel is typically provided along the axis of the beams and columns to resist tension originating from bending of the members. Lateral reinforcing(ties) is typically wrapped around this longitudinal steel to help hold the longitudinal steel and the concrete encased within it together, and to help resist loads applied perpendicular to the axis of the member.

The performance of concrete-frame structures in earthquakes is strongly dependent on the detailing of the reinforcing steel in the beams, columns, and connections. Prior to about 1970, most buildings were not adequately designed to dissipate energy from major earthquakes without failure. These frames are termed non-ductile. Extensive modifications of code requirements for detailing were made in the 1967 and subsequent editions of the *Uniform Building Code*. Consequently, it is expected that concrete-frame

structures designed after adoption of these criteria will perform substantially better than those designed prior to the more stringent codes.

Some non-ductile concrete-frame structures have been severely damaged and/or have collapsed in past earthquakes or wind events. Columns with insufficient ties to prevent cracking and spalling of the column concrete and/or buckling of their longitudinal reinforcing steel frequently have been the source of major damage. Adequately sized and detailed ties and relatively close spacing of the ties are required in more recent building codes.

Longitudinal reinforcing steel is typically of finite length and must be spliced in columns in multiple-story buildings. Some of the past damage to concrete-frame buildings in earthquakes and wind events can be attributed to the concentration of splices for longitudinal column reinforcing steel at one elevation (instead of staggering splices at different locations) or inadequate splice length, both of which create locations of weakness in the columns. During strong ground loading, actual forces and deformations experienced can be significantly larger and different in nature than assumed in a simplified code analysis, and inevitably these locations of weakness are "found" by the earthquake.

Configuration irregularities have also contributed to significant damage and/or collapse of concrete-frame structures in past earthquakes and wind events. Weak or flexible first stories, and lateral-load-resisting framing that is discontinuous at lower levels for architectural reasons, are main sources of past damage. The 1988 *Uniform Building Code* (UBC) restricts the use of such configurations and requires special details or design considerations when they are present.

Recent building codes have permitted designers to designate only certain portions of a concrete frame as lateral load-resisting. Under these provisions, the stringent detailing required for ductile performance was provided only in these elements while other portions of the building were constructed using less ductile detailing. This practice has been identified as a contributing cause for the collapse of several structures in the 1994 Northridge Earthquake. Code changes are currently pending to prevent the continuance

of this practice; however, it will likely be several years before these new provisions are implemented on a wide scale.

A.3.3 Concrete Shear Wall Building Discussion:

The performance of reinforced concrete shear wall buildings is highly dependent on the number of walls, their location within the building, their configuration, the size and number of openings in the walls, and steel reinforcing details. Well-designed concrete walls have adequate reinforcing throughout (both horizontally and vertically) as well as special reinforcing around openings and at edges. For walls greater in height than width, vertical reinforcing steel at edges should be provided with horizontal steel ties that wrap around this steel and the concrete within to hold it together.

Marginally reinforced shear walls can be severely damaged. Cracking and spalling are commonly observed around wall openings as a result of major earthquakes or wind loading. Openings added after the building is constructed, without providing additional edge reinforcement, are particularly common locations for damage. Beams spanning over an opening between two portions of shear walls (spandrel beams) can spall and experience significant cracking if not provided with sufficient reinforcing for shear resistance.

Shear wall buildings with abrupt changes in lateral resistance have often performed poorly in typhoons and earthquakes. Damage concentrates in weak or flexible stories, or at locations where shear walls at upper levels do not continue to the foundation level. Buildings with walls distributed primarily around only two or three sides are subject to large torsional displacements (twisting) and have been severely damaged in past earthquakes and wind events.

Because concrete shear wall buildings are relatively stiff, in comparison to other building types, smaller displacements and less subsequent damage to non-structural elements are anticipated for these buildings. Actual collapse of concrete shear wall buildings is fairly rare.

A.3.4 Reinforced Masonry Building Discussion:

Many modern buildings are constructed with reinforced masonry bearing walls. The walls are most typically constructed of hollow concrete blocks, with reinforcing steel inserted within the cavities in the block, which are then grouted solid. However some reinforced clay masonry walls are also constructed. In these walls, two wythes (layers) of brick are layed-up with a cavity space in between. Reinforcing steel is placed in the cavity between the wythes, which is then grouted solid. In either case, the walls serve as architectural elements, vertical load bearing structural elements, and lateral-load-resisting shear walls.

The floors and roofs of masonry buildings can be comprised of a number of different systems. Many one and two story structures are provided with floors and roofs of timber construction; however, metal deck and concrete filled metal deck supported by steel framing are also common in such structures. Many apartment buildings and hotels are constructed with a system of precast, prestressed concrete plank floors, bearing directly on the masonry walls.

The performance of reinforced concrete masonry buildings with timber floors and roofs is strongly related to the capacity of the wall anchorage to the roof and floor diaphragms. When provided with good wall anchorage details, these buildings often perform well. However, proper wall anchorage was not required by codes until after the 1971 San Fernando Earthquake, when many masonry walls separated from their roof diaphragms and failed due to out-of-plane forces on the walls. A series of code changes requiring improved anchorage of masonry walls to diaphragms were enacted following that earthquake to avoid these failures. Over the years, these provisions have been modified several times, as additional earthquakes and research demonstrated that previous requirements were inadequate. Most recently modifications have again been proposed for the 1997 building code, based on damage observed in the 1994 Northridge Earthquake.

Buildings incorporating precast concrete floor systems may have inadequate diaphragms for wind and earthquake resistance. Modern practice for such construction incorporates a thin reinforced concrete topping slab to bond the precast units together

and provide for monolithic behavior in earthquake ground shaking. Older structures often do not have such topping slabs and can be subject to extensive damage to the floor systems, leading to collapse.

The configuration and detailing of the walls themselves is extremely important to the building's wind and earthquake performance. Walls with extensive openings are often subject to large cracking and spalling of the masonry units around the openings. The 1994 Uniform Building Code adopted special detailing requirements for such masonry wall-frames, intended to provide better performance. However, the use of these improved requirements is optional. The pattern in which the masonry is laid-up is also important. Most concrete masonry is laid-up in a running bond pattern, in which the joints of the masonry units in each layer are staggered relative to the layers above and below. This is a preferred form of construction. Some buildings incorporate a masonry pattern known as stack bond in which the joints between units align vertically from the top of the wall to the bottom.

Another important factor in the performance of masonry buildings is the degree of quality control exercised during construction. Some collapses of masonry buildings in past typhoons and earthquakes have been attributed to failure of ungrouted reinforced cells of masonry, poor quality mortar, and similar construction deficiencies. Continuous inspection during the construction process is an effective method of avoiding such problems, however, such inspection is not required in all cases. Instead, the building codes require greatly reduced design stresses in masonry constructed without special inspection. Many believe that continuous special inspection should be provided, regardless of the design stresses.

As with other types of construction, masonry buildings with substantial plan irregularities have experienced damage in past typhoons and earthquakes. Storefront or corner buildings with walls on only two or three sides have proven very susceptible to damage.

Attachment B

Original Drawing Details & Field Photographs

THIS PAGE IS INTENTIONALLY LEFT BLANK



Figure 1 – Lack of wall to roof anchorage at Trusses. Gap between straight sheathing and truss top chord



Figure 2 – Split Blocking needing replacement and anchorage to wall.



Figure 3 – Split bottom chord bracing at bolt line.



Figure 4 – Split bottom chord of truss



Figure 5 – Lack of wall to roof anchorage at Trusses. Gap between straight sheathing and truss top chord



Figure 6 – Straight wood sheathing, re-entrant corner, lack of collector connection.



Figure 7 – Lack of anchorage at top of column and lack of chord/collector connection.



Figure 8 – Sedimentation Basin Reaction Well lacking lateral support.



Figure 9 – Thin Reinforced Concrete Sedimentation Basin Walls.

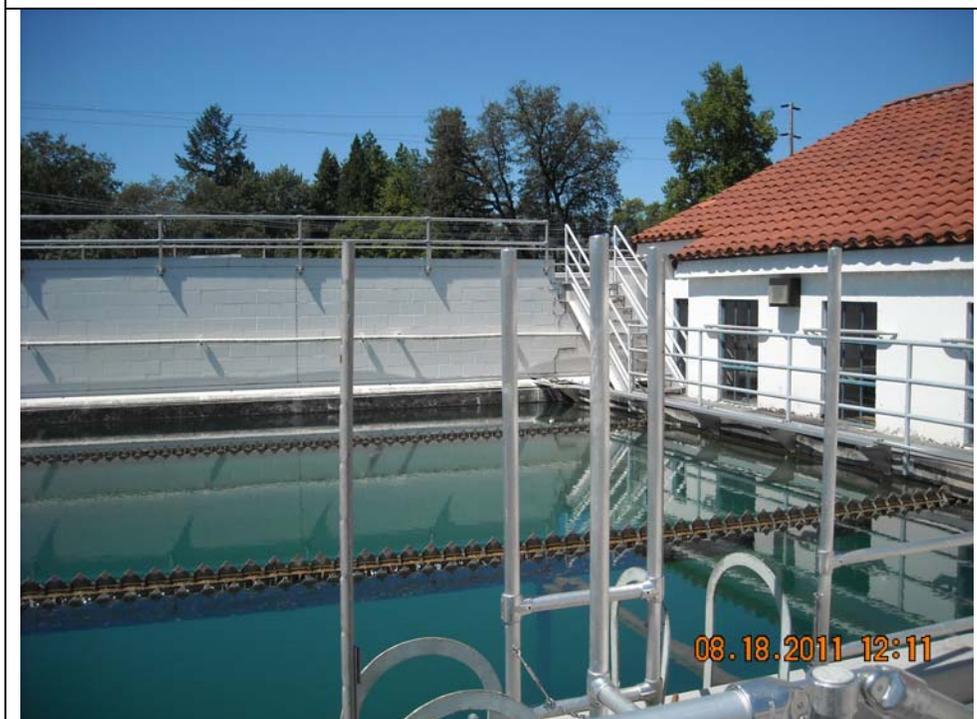


Figure 10 – Cracked CMU wall at corner interconnection from movement.



Figure 11 – Chemical Silo on roof susceptible to collapse.



Figure 12 – Redwood Baffle Wall susceptible to hydrodynamic lateral load failure.



Figure 13 – Deteriorated/Corroded Reinforced concrete around pipe penetration susceptible to brittle failure.



Figure 14 – Filter Gallery exterior wall cracking seepage



Figure 15 – Filter Gallery piping passing through wall that interrupts the jamb vertical steel



Figure 16 - Filter Gallery piping passing through wall that interrupts the jamb vertical steel and head horizontal steel



Figure 17 – Ledger angle missing anchor bolts



Figure 18 – Wall to roof anchorage not apparent (just gravity connection)



Figure 19 – Diagonal bracing for trusses missing anchor bolt to wall through the ledger



Figure 20 - Diagonal bracing for trusses with narrow plate to wall ledger connection



Figure 21 – Steel Collector at shear wall connection is light



Figure 22 - Honeycombed truss grout pockets



Figure 23 – Steel ledger light attachment to walls



Figure 24 – Steel wide flange to wall connections



Figure 25 – River Intake Pump Station that will have torsional seismic response



Figure 26 - River Intake Pump Station with light wall to wood roof anchorage



Figure 27 – Rigid piping leading away from Basins that are susceptible to shearing during seismic events



Figure 28 – Cracked concrete water channel between adjacent sedimentation basins



Figure 29 – Vertically cracked sedimentation basin wall

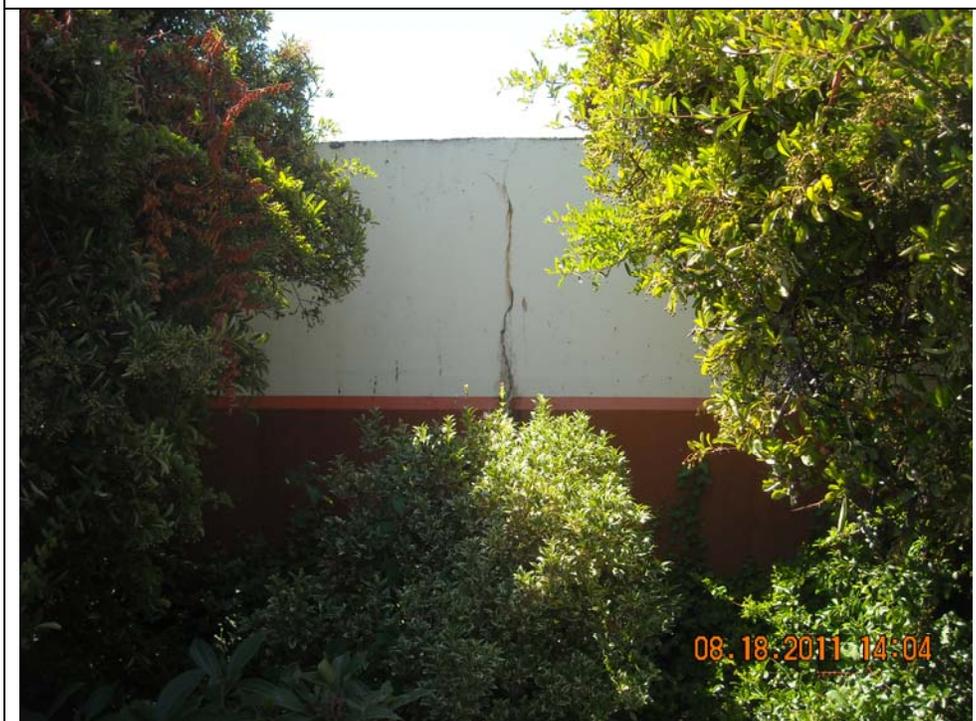


Figure 30 - Vertically cracked sedimentation basin wall



Figure 31 - Vertically cracked sedimentation basin wall at interconnection with filter building



Figure 32 – Horizontally cracked concrete filter building wall



Figure 33 – Rogue River proximity to plant and intake with intake apparently out-of-plumb, slope stability



Figure 34 - Rogue River proximity to plant and intake, slope stabilization/remediation present



Figure 35 – Transformer Anchorage not visible



Figure 36 – Gas meter piping narrow pipe sections lack of capacity for ductile movement



Figure 37 – Poly tank stands lack of anchorage for lateral sliding and overturning



Figure 38 - Polymer tanks lack of anchorage for lateral sliding and overturning



Figure 39 – Aluminum Sulfate and T-Floc tanks lack of anchorage for lateral sliding



Figure 40 - Electrical Equipment Narrow Pads have limited lateral sliding resistance availability



Figure 41 – T-bar ceiling, lighting and HVAC ducts missing lateral splayed wires and compression struts



Figure 42 - Lighting missing lateral splayed wires and compression struts