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1/10/14

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File: Pse/13-168-01

Re: College Hill 607 Reservoir Evaluation

Introduction

The purpose of this document is to include a summary of the following as they pertain to the College Hill 607 Reservoir evaluation:

- Review of original construction documents.
- Review of previous structural/seismic evaluations.
- Existing condition assessment.
- Structural/seismic analysis.
- Identified deficiencies.
- Generation of conceptual upgrades.
- Conceptual upgrades with an engineer's opinion of probable costs.

Codes and Standards

Looking ahead to the State of Oregon's impending adoption of the 2012 International Building Code (IBC) the following codes and standards were referenced for the structural/seismic analysis.

- International Code Council – *2012 International Building Code* (IBC)
- American Society of Civil Engineers – *Minimum Design Loads for Buildings and other Structures* (ASCE 7-10)
- American Concrete Institute – *Building Code Requirements for Structural Concrete and Commentary* (ACI 318-11)
- American Concrete Institute – *Code Requirements for Environmental Engineering Concrete Structures and Commentary* (ACI 350-06)
- American Concrete Institute – *Seismic Design of Liquid-Containing Concrete Structures and Commentary* (ACI 350.3-06)
- American Institute of Steel Construction – *Steel Construction Manual, 14th Ed.* (includes AISC 360-10)

Oregon Resiliency Plan

The Oregon Resilience Plan (ORP) was requested by the 77th Legislative Assembly of the Oregon State Legislature with the expressed purpose to:

1. "Determine the likely impacts of a magnitude 9.0 Cascadia earthquake and tsunami … and estimate the time required to restore functions if the earthquake were to strike under present conditions"

2. “Define acceptable timeframes to restore functions after a future Cascadia earthquake to fulfill expected resilient performance”
3. “Recommend changes in practice and policies that, if implemented during the next 50 years, will allow Oregon to reach the desired resilience targets”

The ORP concluded with a set of recommendations calling for comprehensive assessments of critical facilities, to launch programs for capital investment in Oregon infrastructure, create incentives for rehabilitating private infrastructure, and updating Oregon’s policies in regard to disaster preparedness. The ORP correlates with the design provisions set forth in ASCE 7-10, ASCE 31-03, and ASCE 41-06 which take a probabilistic approach to seismic design, incorporating all fault lines – including the Cascadia subduction zones – in the surrounding areas, with special considerations for essential facilities. It is our understanding that the current ORP does not provide any specific design spectral response accelerations for the proposed Cascadia design earthquake; therefore, the above outlined codes and standards, as well as the USGS values for the site, have been utilized for the analysis which includes the referenced subduction zone earthquake in generation of site specific seismic design values.

Essential Facility Requirements

The College Hill 607 Reservoir is considered an essential facility. According to ASCE 7-10, essential facilities are: “Buildings and other structures that are intended to remain operational in the event of extreme environmental loading from flood, wind, snow, or earthquakes.” As such the Risk Category for this reservoir is IV according to ASCE 7-10, Table 1.5-1. Therefore, analysis of this structure must use the Importance Factors as summarized in ASCE 7-10, Table 1.5-2. Most significantly, the Seismic Importance Factor, I_e, is set at 1.5.

Background

The College Hill 607 Reservoir was constructed in 1939 as part of the FDR Public Works Administration. The reservoir is located in Eugene, Oregon and is bounded by Lawrence St. and Lincoln St., between West 23rd Ave. and West 25th Ave. This rectangular standard reinforced concrete reservoir has a 15 MG capacity proportioned equally between two symmetrically square chambers which are divided by a shared wall. The 607 in the reservoir’s name indicates the location of the overflow pipe in terms of feet above sea level. In conjunction with other reservoirs in the area, the College Hill 607 Reservoir is an active drinking water source for the EWEB Service Area. It is our understanding that this reservoir is also part of the required fire suppression water supply system. Therefore, the reservoir is classified as an Occupancy Category IV essential facility for the basis of this evaluation.

Figure 1: College Hill 607 Reservoir During Construction
(Photo Credit: EWEB)

Review of Original Construction Documents

The College Hill 607 Reservoir is a rectangular standard reinforced concrete structure divided into two symmetric square tanks by a shared central wall. Each tank is approximately 240’ by 240’ and they are situated next to each other along a north-south axis with the short side perpendicular to the axis. Both cells have a hopper shaped bottom with a flat center area surrounded on all sides by a sloping floor. The sloped floor slabs connect to the vertical walls around the perimeter of the tanks.

For most of the perimeter of the tanks the sloped floor slabs are constructed on grade. However, for the entire eastern edge and short end returns of the north and south edges of both tanks, the sloped floor slabs are supported by a series of 8” and 12” thick bearing walls. Appendix Figure 2 shows the extent and location of these support walls. Open space exists between these walls and there is a hallway that allows access underneath these portions of the sloped floor slabs. Appendix Figure 3 is a reproduction of an original construction drawing showing a section through the eastern edge of the tanks including this hallway.

The roof is an 8-1/2” thick slab. The slab is supported by 18” Ø spirally reinforced concrete columns. The columns are on a regular grid supporting sixteen cast-in-place roof segments per tank. The segments have expansion joints between them and are not structurally tied together. Each segment is supported by nine columns which have 7’-9” square, 4-1/2” thick, drop panels and 5’0” Ø, 1’-9” tall, conical capitals at the top. The column bases rest upon square footing pedestals atop the floor slab. The forty-four perimeter columns rest on the sloped floor and thus are shorter than the one-hundred interior columns. The roof has no connection to the walls and is free to translate atop the perimeter walls.

The tanks can be accessed by stairs inside an enclosure structure atop the east side of the roof located over the shared central wall. The perimeter of the roof has a protective railing constructed of pipe rails supported by concrete pedestals.

Review of Previous Evaluations

EWEB provided PSE with an evaluation report of the College Hill 607 Reservoir written by OBEC Consulting Engineers on June 18, 1999. Using the building codes and standards current at that time, OBEC structurally analyzed the reservoir and provided a matrix of deficiencies based on their analysis.

From this report it is our understanding that, although there were minor deficiencies noted in the foundation walls and the elevated sloped floor slabs, two deficiencies stand out:

- The columns are structurally inadequate under seismic loads, especially the short columns.
- The exterior corners of the tanks are structurally inadequate under both seismic and serviceability loads.

OBEC based their analysis on loads generated according to the provisions found in the 1998 Oregon Structural Specialty Code (OSSC) which was primarily based on the 1997 Uniform Building Code (UBC). Since then significant changes have been incorporated into the current codes and standards. Whereas seismic loads were previously determined according to maps that sectioned large land areas into generalized seismic zones, seismic loads are now determined based on probabilistic models that are site specific. It is important to note that this increased granularity of data can result in site specific loads that are either greater or less as compared to those generated by earlier codes and standards which relied on generalized seismic zones. Additionally, the detailing requirements for structures in seismic regions are now much more stringent in order to guarantee ductility in the key structural elements comprising the lateral force resisting system and ensure the ability of the structures to redistribute loads in a seismic event. Therefore it is possible for seismic loads to decrease or remain similar but the expected seismic performance to have gone down due to the increased knowledge of ductility requirements in a seismic event.

Existing Condition Assessment

In 2013 PSE conducted three site visits with the help and support of EWEB personnel. The first took place on July 24th, the second on October 15th, and the third on October 29th. The first visit was just an introductory look at the outside of the structure and the surrounding environment. The second and third visits included investigations inside the water tanks as well as inside the hallway underneath the sloped floor slab along the eastern side of the reservoir. During the second visit the north tank was drained while the south tank was full, and during the third visit this arrangement was reversed. This allowed us to investigate the structural walls supporting the sloped floor slab in both the north and south tanks while they were filled with water.

While investigating the roof slab we observed several locations where the flexible mastic at the expansion joints had pulled away from the concrete, presumably due to thermal expansion and contraction of the adjacent roof slab panels. This mastic is part of a waterstop system which was installed to replace the original copper-strip waterstop system. The new waterstop system consists of backer rod with a flexible mastic coating. This new system was observed to have failed in several locations and EWEB continues to spot repair this system as required with varying results (see Appendix Figure 4). We observed several locations where the vertical exposure of the expansion joint was not watertight. It appears that water can penetrate directly underneath the backer rod at these locations (see Appendix Figures 5 and 6).

EWEB also previously replaced the water-stop system around the perimeter of the reservoir between the underside of the roof slab and the top of the walls with an application of flexible mastic. At several locations we observed that mastic pulling away from concrete and daylight could be seen between the roof slab and wall interface at several locations (see Appendix Figure 7)

OBEC, in their evaluation report, noted many locations where the roof slab had cracked on the topside. We did not observe any cracks on the topside of the roof slab as EWEB has applied a protective coating. However, cracks were observed on the underside of the roof slab (see Appendix Figure 8). Secondary efflorescence was visible along these cracks however indications – such as no evidence of stalactite formations – suggest that these cracks are hairline, self-healed, and/or that the coating topside is halting water from penetrating through the cracks, except as noted at the roof joints.

From the above observations it is assumed that penetration of rainwater and other materials into the storage volume occurs. We also observed that all of the appurtenances inside of the tanks showed signs of rust and corrosion. Appendix Figure 8 shows the washdown pipe hanging from the ceiling.

Concrete spalling and cracking was observed at several locations on the exterior wall surface (both can be seen in Appendix Figure 9). Many of these cracks appear to have self-healed over time and the spalling is not extensive relative to the overall surface area of the walls. However, at several locations, the cracks and spalling are, however, in need of repair and maintenance. Spalling was also observed at the top of the interior wall at the wall to roof slab interface. This appears likely to be occurring due to thermal expansion of the roof panels against this portion of the wall.

During our investigation of the support walls underneath the sloped floor on the eastern side of the reservoir we observed several active leaks. The locations of these leaks are noted by blue circles in Appendix Figure 10. The Appendix Figures 11 through 14 give a sense of the amount of water leaking at these locations. It was also observed that all of the leaks correspond to locations where stiffened angles on both sides of the support walls have been post-installed using 1” Ø A325 thru-bolts. The location of these angles are noted by solid red lines in Appendix Figure 10. The design and location of these angles follow, in part, the remedial recommendations and instructions found within OBEC’s evaluation report. It was also observed that these leaks correspond to the closing strips of the sloped reservoir floor slabs and walls located directly above. It was observed that a remedial waterstop system consisting of flexible mastic with backer rod has been applied to all of these closing strips (see Appendix Figures 15 and 16). The consistent location of the leaks corresponding with these closing strips suggests a failure of the waterstop system at these joints.

We observed that the reservoir is actively leaking through the exterior wall at the south-eastern corner (see Appendix Figure 17). This location corresponds directly with one of the leaks observed in the support wall underneath the sloped floor and discussed above. It also corresponds with a wetted area within the access hallway where dampness and droplets were observed on the wall and ceiling. The extents of this dripping damp area extend in both directions from the south-east corner and are noted by a solid blue line in Appendix Figure 10. The leaking in this area appears likely attributable to the failing watersop system at the closing strips as noted above. However, considering the volume of water leaking at this location and the fact that the leak is penetrating through the exterior wall, this leak may be the result of an additional failure of the original 16 GA galvanized iron waterstop (see Appendix Figure 18).

Structural / Seismic Analysis

Consistent with the current provisions of ACI, ultimate strength design methods (LRFD) were employed to evaluate the adequacy of reservoir's structural elements. Where appropriate, in addition to ultimate load cases, structural elements were analyzed for serviceability requirements. Additionally, for load cases that do not include earthquake load effects the environmental durability factor (S_d) was applied.

EWEB provided PSE with a geotechnical report previously written for the Willamette 800 Reservoirs which are located just under 3 miles from the College Hill 607 Reservoir. For the purposes of this evaluation, we have assumed that the geological conditions are similar between these two sites. We have therefore used the following geotechnical parameters consistent with the Willamette 800 Reservoirs site as the basis for our analysis of the College Hill 607 Reservoir:

- Site Soil Class: B
- At-Rest Earth Equivalent Fluid Density: 120 pcf
- Active Earth Equivalent Fluid Density: 40 pcf
- Seismic Thrust Equivalent Fluid Density: 13 pcf
- Height of all earth resultant forces: 0.33H

Note that no geotechnical evaluation has been performed for the College Hill 607 Reservoir at this time and the above values have been assumed based upon the close proximity of sites. It is our assumption that the sites are similar and the values are a reasonable assumption for the initial evaluation, however, a geotechnical evaluation should be performed prior to undertaking structural upgrades.

EWEB provided PSE with the original specifications and contract documents for the construction of this reservoir. From these references, the concrete strength was assumed to be 3,000 psi and the reinforcement was specified to meet ASTM A15-35, Intermediate Grade, which has a yield strength of 40 ksi.

The United States Geological Survey (USGS) provides tools for determining seismic parameters based on site location. The College Hill 607 Reservoir is located at 44.033°N, 123.098°W. Using the provisions of ASCE 7-10, the USGS seismic design maps tool calculates the following design spectral response acceleration parameters:

- S_{DS} = 0.516 g
- S_{D1} = 0.271 g

S_{DS} and S_{D1} are the design spectral response acceleration parameters for short periods and a 1-second period respectively.

Load generation took into account both static and dynamic load cases and additionally considered the tank in both the full and empty state, as well as alternating one tank full while the other tank is empty. All load cases were considered for both static as well as dynamic conditions.

In what follows we shall describe the structural analysis for the primary members of the reservoir. Note that the demand-capacity ratio (DCR) is a method of quickly expressing the adequacy of a structure (or member/s of a structure) for performing the intended function under consideration. Essentially one takes the demand that is placed on the structure and divides that by the calculated capacity. Per the requirements of current Codes and Standards, if the resulting ratio is greater than 1.00 the structure is considered inadequate as the demand is requiring more than 100% of the structures capacity. If the resulting ratio is less than 1.00 the structure is considered adequate for performing the intended function under consideration.

Critical Wall

Several load cases were considered in order to determine which wall is critical for structural analysis. The load cases were as follows:

- Tank Empty, Static Load, West Wall
- Tank Empty, Dynamic Load, West Wall
- Tank Full, Dynamic Load, Trailing Wall
- Tank Full, Dynamic Load, Leading Wall
- Tank Full, Dynamic Load, Center Wall

Given the symmetry of the overall reservoir the above load cases consider each tank independently with one exception. The Center Wall load case considers the entire reservoir as 1/2 empty (one tank full) as this is the critical loading condition for that wall. When both tanks are full the opposing hydrostatic forces on the center wall cancel each other thus reducing the overall loads. Additionally, Static and Dynamic Loads consist of all the forces generated by the contained fluid (if tank is full), retained earth and the structural mass. Static loads consist of the standing pressures of both the contained fluid (if tank is full), the retained soil at locations where the walls are below grade, and the self-weight of the structure. Dynamic loads consists of the seismic effects (i.e. lateral forces) of the contained fluid (if tank is full), the retained soil and the inertial forces related to the structures' mass.

In this instance, the first load case (tank empty with static load) controlled the analysis as it generated the greatest internal forces within the wall. The load was factored by the following load combination: S_d(1.6H).

The typical wall has two rows of 5/8" Ø vertical bars spaced at 13-1/2" on center and, at the base, has one row of 5/8" Ø vertical dowels spaced at 4-1/2" on center. Based on this reinforcement the moment capacity of the wall exceeded the required design load. Thus it was determined that the critical wall is adequate with a DCR of 0.87.

Sloped Floor Slab

In order to analyze the adequacy of the sloped floor slab a computer model of the slab was developed using SAFE v.12 modeling software. Two models were developed, a single simple-span of the slab between support walls and another continuous two-span model. Lateral, hydrostatic and gravity loads were generated and distributed across the sloped floor panel after being factored using the critical load combination: 1.2(D+F)+1.0E. The models were then analyzed to determine the required area of steel reinforcement. This was then compared to the provided reinforcement in the existing slab. The critical DCR was 0.48 for the bottom reinforcement in the single span slab. Therefore the sloped floor slab is adequate.

The sloped floor slab can also be considered as an extension to, or lower portion of, the tank's perimeter walls. From this perspective, and using the same load cases as those described above in the critical wall section, the sloped floors were analyzed as angled walls resisting lateral forces. Once again the tank empty with static load controlled the analysis. The factored – S_d(1.6H) – overturning moment and resisting moments were calculated and the resulting induced moment on the sloped floor slab was determined to be below the capacity of the slab. Therefore the sloped floor slab is adequate when considering the sloped floor slabs as angled walls.

Support Walls Supporting Sloped Floor Slab

The support walls underneath the sloped floor slab were analyzed for gravity and lateral loads. The critical load combination was 1.2(D+F)+1.0E. Our analysis showed that these walls are structurally adequate with a DCR of 0.50. However, the provided reinforcement in the existing structure is 0.2%. This is less than the minimum required, 0.3%, according to ACI 350-06.

Columns

Each tank has 144 columns spaced across a regular grid supporting the roof slab. The interior 100 columns are supported by square footing pedestals on top of the flat floor slab. The 44 perimeter columns are shorter with their base pedestals resting atop the sloped floor slab. Because the roof slab is free to translate by sliding atop the tank walls, the columns, by way of cantilever action, constitute the lateral force resisting system (LFRS). The gravity and lateral loads were factored by the critical load combination: 1.2D+1.0(E+L). Our analysis determined that both the long and short columns were inadequate with DCR's of 1.27 and 3.71 respectively.

Roof Slab

A computer model of the roof slab was developed using SAFE v.12 modeling software. This model was verified by hand calculations according to the equivalent frame method. The roof slab was determined to be adequate with a DCR of 0.85 after factoring the loads with the critical load combination: S_d(1.2D+1.6L).

The roof slab was analyzed with regard to the cracking that was observed on the underside during our site visit. Our analysis determined that these cracks are not due to a deficiency in strength. The determined maximum stresses inside the concrete slab are less than the calculated cracking stresses. Additionally, we did not observe any noticeable deflections corresponding to the areas of cracking, nor signs of ponding on the roof. Given the age of this concrete slab, 74 years, we assume that the cracks were either due to an untimely removal of the shoring during the construction, shrinkage cracks, and/or the result of years of repeated thermal cycles.

When considering the slosh wave, however, the roof slab was determined to be inadequate. The roof slab was constructed without any bottom reinforcement along the perimeter edges. This reinforcement is necessary to resist the bending forces that would be caused by the slosh wave if sufficient freeboard for a slosh wave is not provided. For this reservoir, the calculated slosh wave height was more than four times greater than the provided freeboard. The code required freeboard for this reservoir would be a minimum of 3'-3" and currently only 9" is provided.

Summary of Analysis Results and Matrix of Deficiencies

What follows is a brief summary, in matrix form, of the results of the structural analysis described above:

Table 1: Summary of Demand vs. Capacity

Structural Element	DCR	Load Case	Notes
Typical Wall	0.87	S_d(1.6H)	
Roof Slab	0.85	S_d(1.2D+1.6L)	
Roof Slab: Slosh	NG*	S_d(1.4F)	*No bottom reinforcement and insufficient freeboard.
Sloped Floor Slab	0.48	1.2(D+F)+1.0E	
Sloped Floor Support Walls	0.50*	1.2(D+F)+1.0E	*Provided reinforcement < required per ACI 350-06
Long Column	1.27	1.2D+1.0(E+L)	
Short Column	3.71	1.2D+1.0(E+L)	

Summary of Conceptual Upgrades

Table 1 shows that the columns are the greatest deficiency in this structure with a demand capacity ratio (DCR) for the short columns of 3.71. Thus there is a high probability that the columns, and subsequently the roof slab, will fail in a code level seismic event. It is assumed that extensive damage to the reservoir, likely rendering it unusable, will occur in a code level seismic event. Therefore retrofit lateral force resisting systems were conceptually evaluated in order to mitigate this deficiency and maintain the desired level of service of the structure during and after a seismic event.

We considered a LFRS that used the existing exterior walls and the existing tank divider wall as shear walls. This requires tying the roof slab panels together with a pour strip at the current expansion joint locations in order to transform the slab into a functional diaphragm. Shear forces are transferred to the wall by way of shear cans, a system PSE has designed and implemented on several other concrete reservoirs. In terms of strength this system would be adequate. However, in order to accommodate thermal expansion of the slab, the shear can receivers would require a minimum 3/4” clear space. Limiting the clear space more would cause the slab to be overstressed by the forces generated due to restricting thermal expansion. Therefore, the columns will deflect 3/4” when transferring the seismic force from the diaphragm to the walls. Although the long columns will not be overstressed by this amount of deflection, the short columns will be overstressed. Therefore, it appears using the existing walls as shear walls is not a viable LFRS.

According to our analysis the maximum deflection that the existing short columns can sustain is 1/2”. Thus any LFRS must limit overall deflections to equal or less than 1/2”. One option would be to construct new shear walls inside the existing tanks. An example layout plan of this type of LFRS can be seen in Appendix Figure 19. The cruciform shape and location of these shear walls would take advantage of the existing expansion joint locations in the roof slab. As described above, these joints are in need of repair. One option is to cut out these expansion joints and replace them with retrofitted cold joints, effectively making the slab monolithic. The shear walls could be incorporated directly into these joints as noted in Appendix Figure 19.

Another possible LFRS that limits the maximum deflection to 1/2” would be a cross-bracing system whereby the top of one column would be connected by bracing rods to the bottom of the adjacent column. Appendix Figure 20 depicts the bracing pattern for a typical roof panel. These braces can be constructed using stainless steel, galvanized steel, or aluminum with the latter two being similar in costs and significantly less expensive than stainless steel. This system was also included in OBEC’s evaluation report as a viable LFRS upgrade for the College Hill 607 Reservoir. A preliminary analysis of this LFRS has been completed and it assumed that a full analysis will require upgrades to the footings and columns for this system to be adequate.

In addition to providing a functional LFRS the reservoir is in need of repairs and maintenance. The spalling and cracks along the exterior walls of the reservoir should be repaired to halt any further deterioration. The waterstop system at the expansion joints between the roof panels also appears inadequate and needs to be replaced. Two options were considered:

- Remove the existing waterstop system and replace with a new backer rod and flexible mastic.
- Cut out the joints and surrounding concrete and replace with cold joints, effectively making the roof slab monolithic.

The latter option would work well should the new interior shear wall LFRS be implemented. The waterstop system at the roof slab to wall interface also appears that it needs to be replaced by removing the existing system and replacing with a new backer rod and flexible mastic is required. If no LFRS upgrades are performed it is likely that any new caulking repairs will eventually fail due to repeated thermal cycles. The closing strips inside the tank at the floor slab, sloped floor slab, and walls also need repair and maintenance.

Due to the existing freeboard being less than what is required by current Codes and Standards, there are three options:

- Operate the reservoir at less than capacity, thereby lowering the water level.
- Accept the likely damage to the cantilevered portions of the roof slab that will be most impacted by the slosh wave in a code level seismic event.
- Remove the cantilevered portions around the edges of the roof slab and replace with new slab with sufficient bottom reinforcement to resist the forces induce by a code level seismic event.

Lastly, the portions of the structure that do not have minimum required reinforcement according to current Codes and Standards are not easily upgraded. For example, to increase the reinforcement ratio in the bearing walls under the sloped floor slabs would require, in our opinion, a prohibitive amount of effort and expense. Thus it is likely that portions of the structure such at these bearing walls will not meet, in entirety, the requirements of the current Codes and Standards even after upgrades are performed.

Engineer's Opinion of Probable Costs

In order to provide a range of probable costs, we estimated the probable costs for three upgrade scenarios summarized in the following matrix:

Upgrade Scenario	Cost
Contruct internal shear walls. Replace existing expansion joints with concrete pour strips.	$ 2,790,000
Construct cross bracing. Replace existing expansion joints with concrete pour strips.	$ 3,100,000
Construct cross bracing. Repair existing expansion joints with new backer rod and flexible mastic.	$ 1,950,000

The above probable cost estimates are based on the preliminary analysis performed for this evaluation report and the conceptual upgrades described herein. Note that these costs do not include all of the repair and maintenance recommendations described in this report. For example, repairing the cracks and spalling concrete on the exterior walls is not included. The Engineers opinion of probable cost are based upon the experience and opinion of an Oregon licensed Structural Engineer based upon experience with similar projects. Appropriate ranges should be included with the cost estimates given the conceptual level of design (such as -30%/+50%) and additional O&P and design fees should be included as well (such as 10% for each).

Thank you for requesting our evaluation services. It has been our pleasure to assist you in this process. Please don't hesitate to call our office should you have any further questions or comments.

Sincerely,

Travis McFeron, P.E, S.E

Appendix:

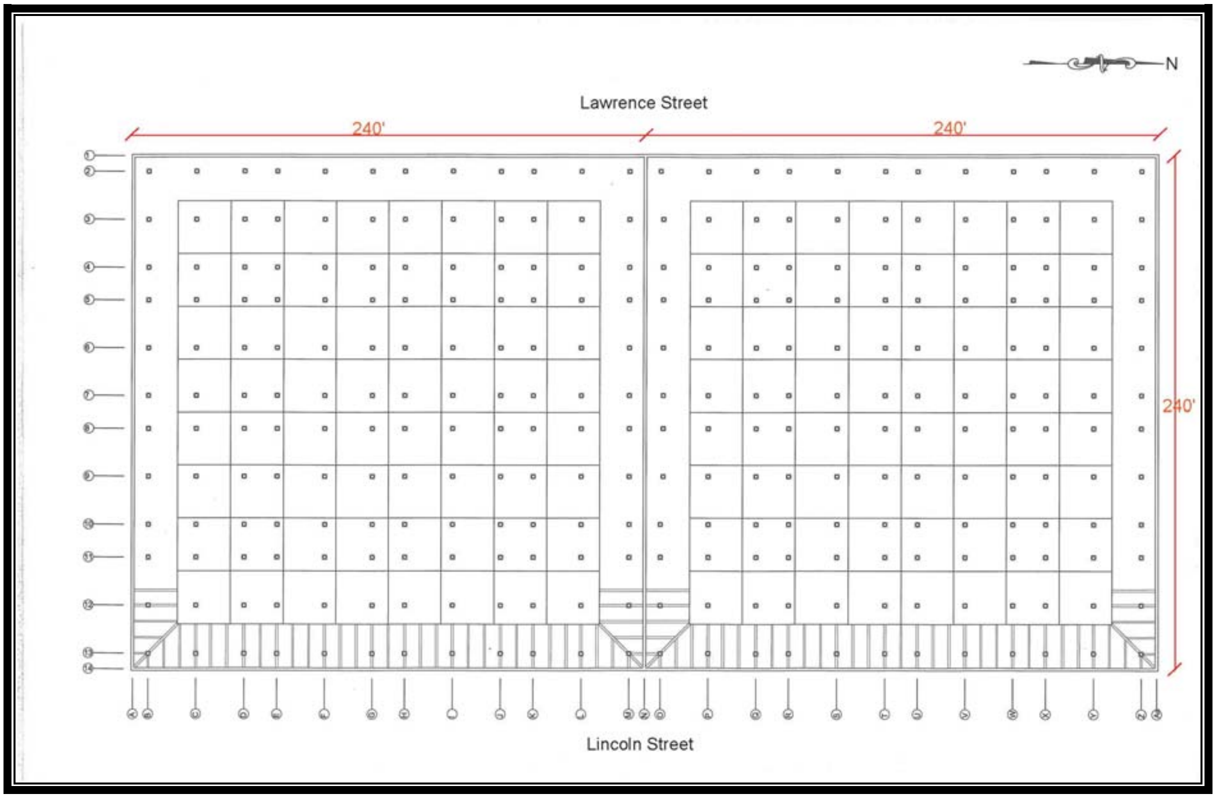

Figure 2: Floor Plan Showing Both Tanks and Extent of Walls Supporting the Sloped Floor Slab Along Eastern Edge and Portions Each Tank's North and South Edges

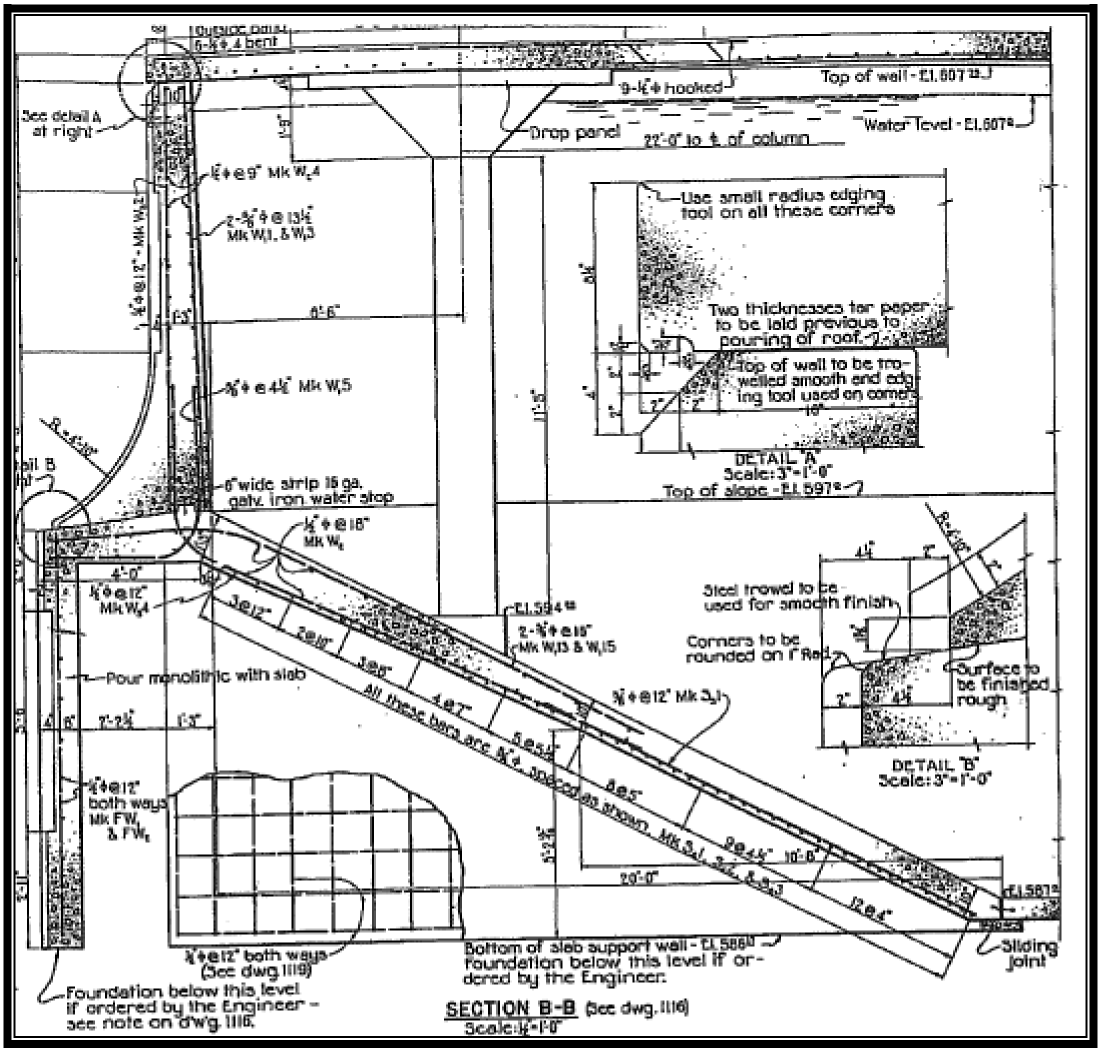

Figure 3: Original Construction Drawing Showing Section Through the Sloped Floor Slab, Bearing Wall, and Hallway Underneath the Eastern Side of the Tanks

Figure 4: Roof Slab Expansion Joint Showing Spot Repairs to Failing Mastic

Figure 5: Non-Watertight Expansion Joint in Roof Slab Perimeter

Figure 6: Non-Watertight Expansion Joint in Roof Slab Perimeter

Figure 7: Mastic Pulling Away from Roof Slab and Wall Interface

Figure 8: Cracks in Underside of Roof Slab

Figure 9: Concrete Spalling and Crack on Exterior Wall

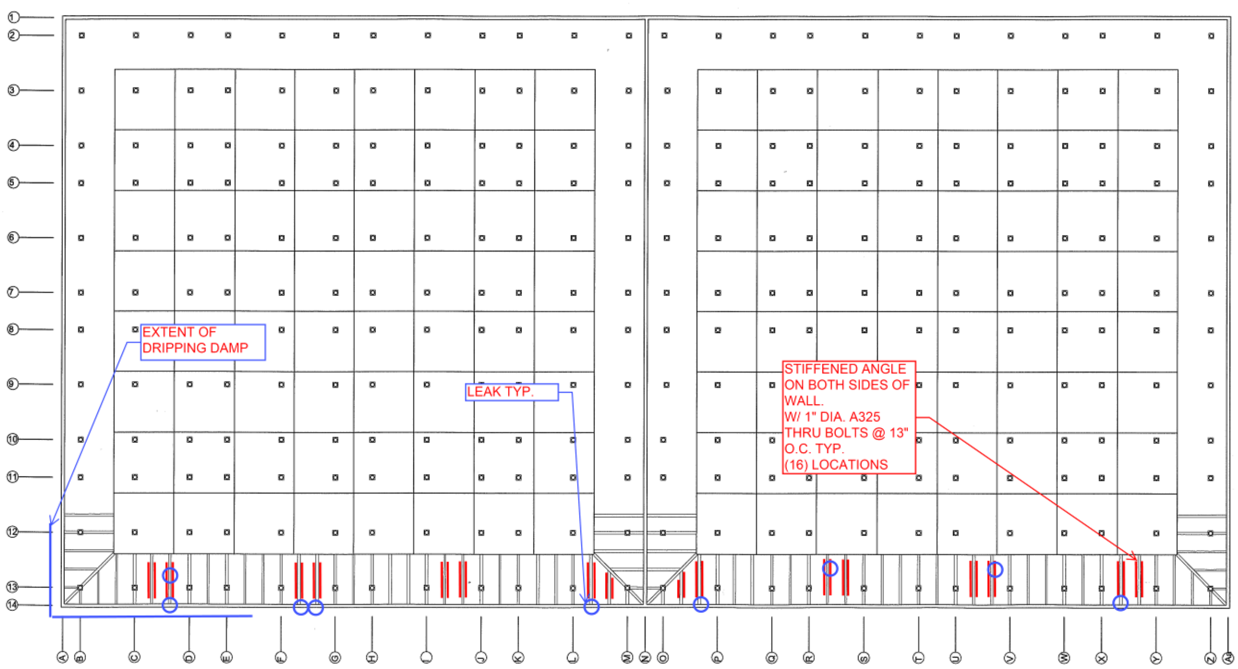

Figure 10: Location of Observed Leaks and Prior Remediation Attempts

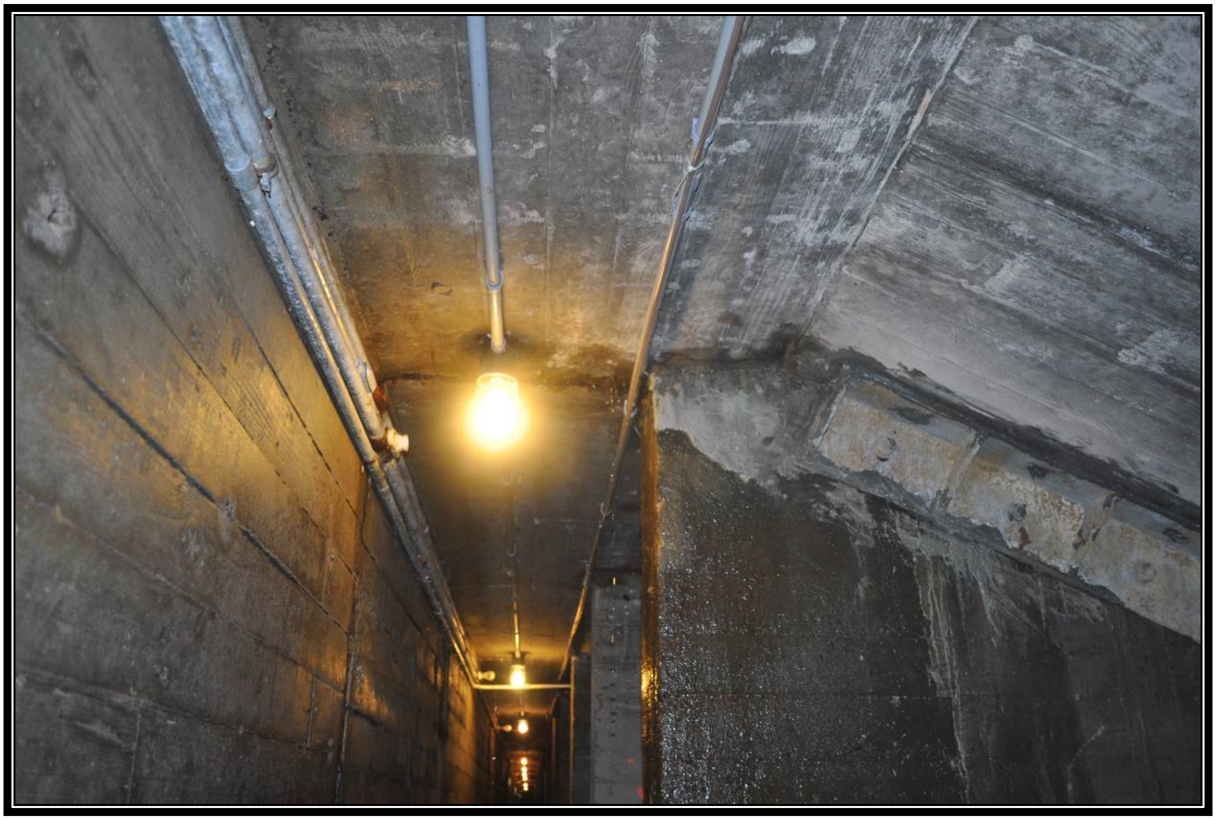

Figure 11: Leak Under Eastern Sloped Slab

Figure 12: Leak Under Eastern Sloped Slab

Figure 13: Leak Under Eastern Sloped Slab

Figure 14: Leak Under Eastern Sloped Slab

Figure 15: Closing Strips of Sloped Slab and Wall

Figure 16: Closing Strips of Floor Slab, Sloped Slab and Wall

Figure 17: Leak Through Exterior East Wall At South-East Corner

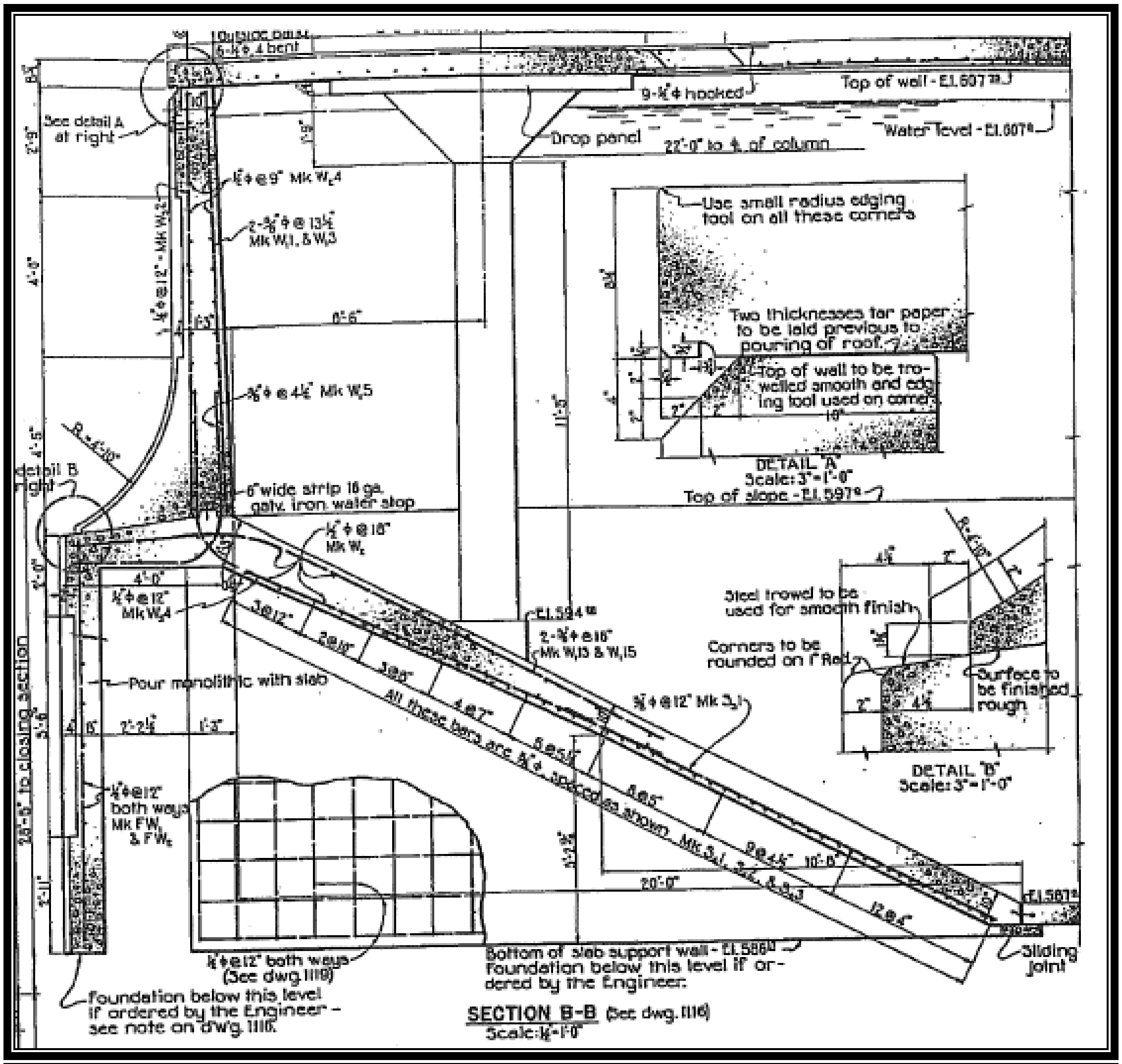

Figure 18: Original Construction Detail of Existing Exterior Wall Along Eastern Side

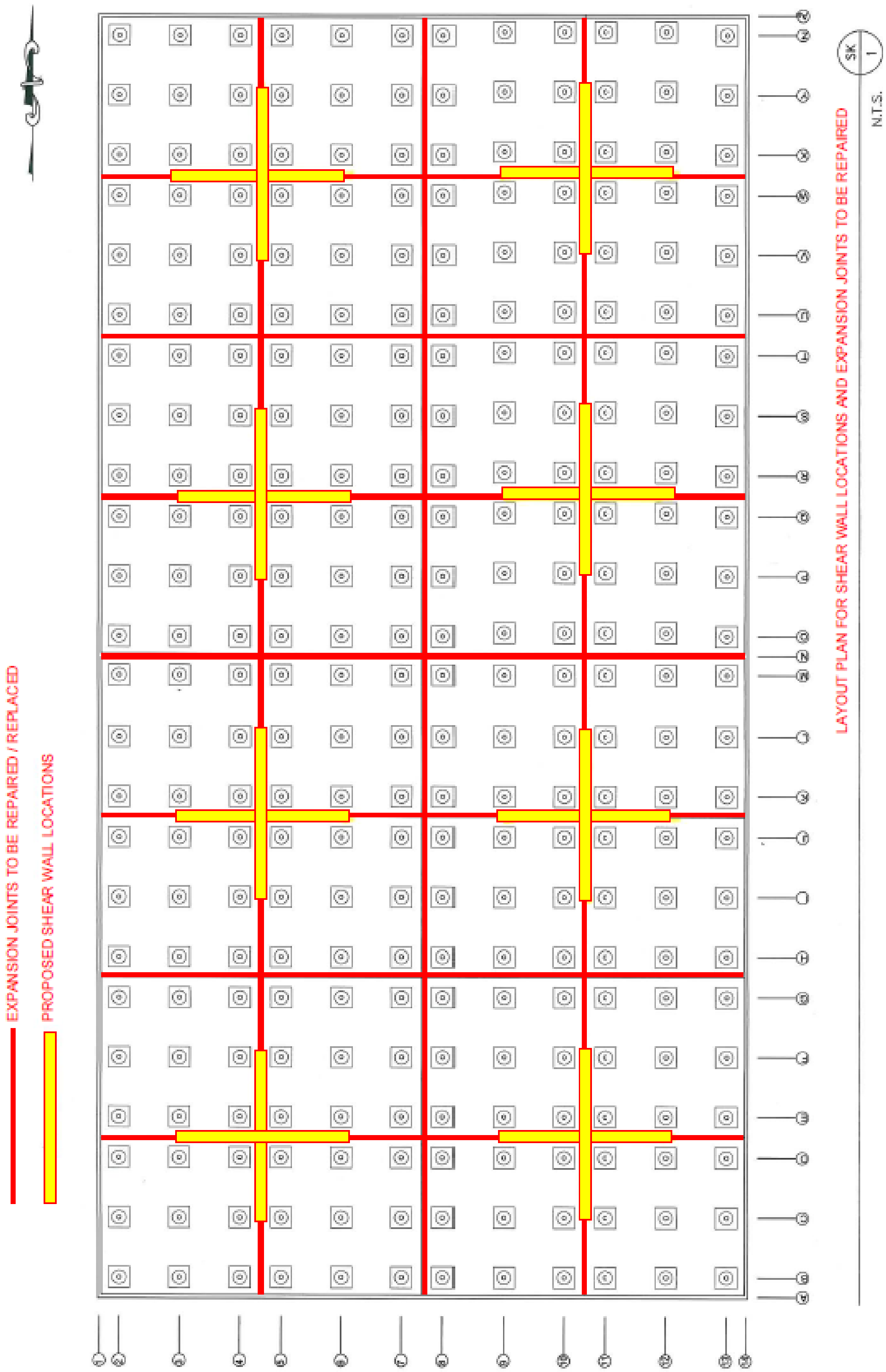

Figure 19: Example of Shear Wall Layout

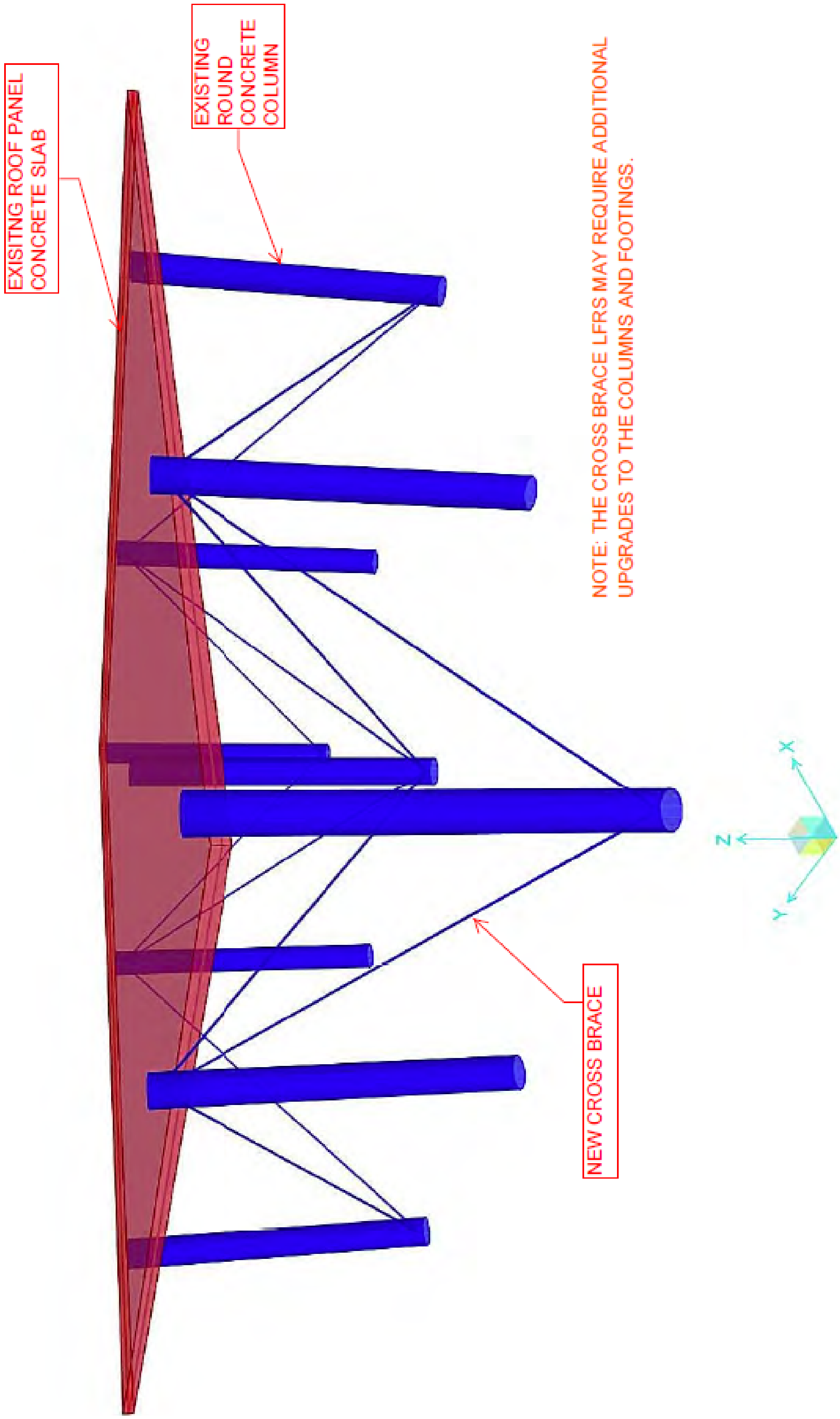

Figure 20: Cross Bracing LFRS for Typical Roof Panel