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MOSER TOWER

CORROSION AND MATERIAL DURABILITY ANALYSIS

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FOREWORD

This document is the material and corrosion assessment performed at Moser Tower, Naperville, IL. In-depth corrosion and material testing was performed on the tower to establish the condition and provide modelling for expected performance in the future.

Acknowledgement

Echem Consultants would like to thank the following firms for their contribution, assistance and general help on the project: Brian Dusak from ERA, Zack Banasik from Golf Construction, and Mary Brush and Barnaby Wauters from Brush Architects.

QUALITY ASSURANCE

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DISCLAIMER

EXECUTIVE SUMMARY

The material assessment and testing carried out at Moser Tower focused on the assessment of corrosion to the reinforcing steel embedded in the concrete. As the concrete provides a corrosion resistant environment to the embedded steel it is essential to understand if this environment is compromised around the structure. To enable this to be carried out a combination of test methods are adopted providing scientific data which is used to ascertain existing conditions and to provide an indication on how the structure will perform in the future.

As the structure consisted of many individual components, Echem's test program only tested a select few. These were chosen based on discussions with the team and previous areas where damage existed. This is not an uncommon approach as it is generally impractical to test all areas due to access and cost.

Our findings are consistent with a structure exposed to the elements where high moisture conditions prevail. All of the testing carried out supported the embedded temperature and humidity sensors findings as corrosion levels were low most probably due to high moisture and low oxygen conditions. When combining areas of the test together the structure would be said to be at medium risk for the precast concrete [Condition State 3 – 2/3 of its service life for materials] and low risk for the cast in place [Condition State 4 – 1/3 of its service life for materials]. This means that it is an optimum time to address future issues which we would be certain will reach deterioration thresholds moving forward. This would be regarded as a proactive repair program where works would be designed to slow down or prevent issued from occurring. In contrast if it takes too long of a time period to implement this repair program, the degradation of the structure will need to be addressed on a reactive basis where a much more significant cost would be incurred.

Overall the structure has a number of minor material issues but, generally speaking, all of these can be addressed reasonably easily. Most of these issues require water management to be the driver for decision making. If water ingress is managed the long-term durability of the materials will be improved.

We have included our Life 52® Reliability process defined as the probability of survival in a given time period. The context of survival is usually associated with some kind of failure avoidance.

Two main deterioration processes were considered as our objective as follows:

- degradation of the concrete surface (e.g. spalling of concrete)
- reduction in structural integrity caused by the reduction of reinforcing bar area

Both of these processes of deterioration are related to the corrosion of reinforcing steel. The products of corrosion are greater in volume [typically six times] than the original steel, and stresses are therefore gradually imposed on the concrete, eventually leading to delamination and spalling of the concrete. Corrosion is accelerated by the presence of chloride ions deposited on the concrete.

The time at which corrosion and subsequent deterioration commences and the rate at which it proceeds is related to several factors, including depth of concrete cover, reinforcing steel, water-cement ratio of the concrete, concentration of chloride ions, crack size and temperature/humidity.

All of these parameters were taken into consideration in our testing and final assessment. This work should be coordinated with the structural engineers report to complete the assessment.

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SECTION 1 PROJECT OVERVIEW AND OBJECTIVES

1.1. Description of The Structure

The Moser Tower, commonly known as the Bell Tower, was constructed between 1999 to 2000. Located in Naperville, Illinois the Tower was sponsored by The Millennium Carillion Foundation as a commemorative gift to the beginning of the third millennium. The tower is a 160 feet tall open-air structure created to contain carillons and bells.

FIGURE 1 MOSER TOWER AERIAL VIEW

The bells and carillons are located at the belfry [Top four levels of the tower], which includes the largest of the carillon bell and the Carillonneur's Cabin from where musicians offer their concerts. Finally, at elevation +138.1 feet, there are the smaller, higher-pitched bells encircling the rafters. An observation deck is located at the top of the belfry from where to enjoy views of the park and the city.

The tower was originally designed to be an enclosed space, but due to lack of funding, it was never completed and remains today as an-open-structure. The structure comprises of reinforced concrete foundations, columns at the basement and at the entry level of the building. After the entry wall, pre-cast post-tensioned panels are designed as load-bearing components for the remaining elevations of the tower. The pre-cast panels are located at the four corners of the tower as shown in Figure 2. In addition, pre-cast fins run the full height of the structure above the entry wall designed as a decorative element.

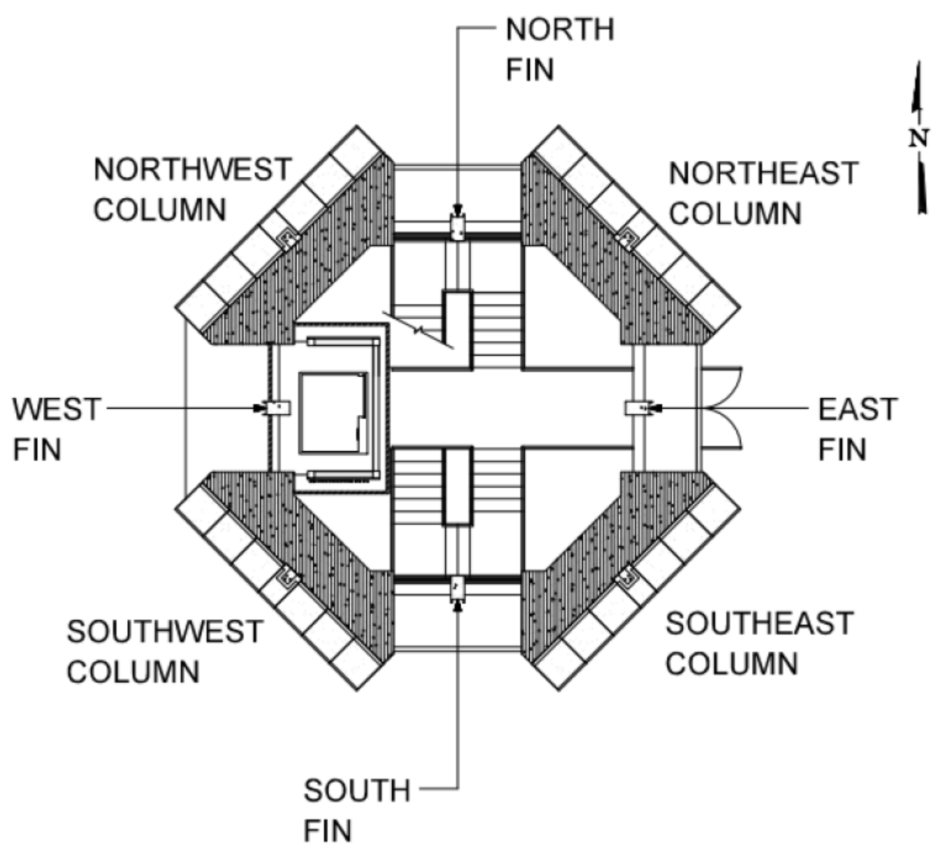


FIGURE 2 TYPICAL PLAN VIEW FOR ENTRY LEVEL 1 TO CABIN LEVEL

Seven Compressive rings at different heights of the structure are connected to the pre-cast panels as shown in Figure 3 and Figure 4. Each of the compressive rings have a critical role in the structural stability of the tower.

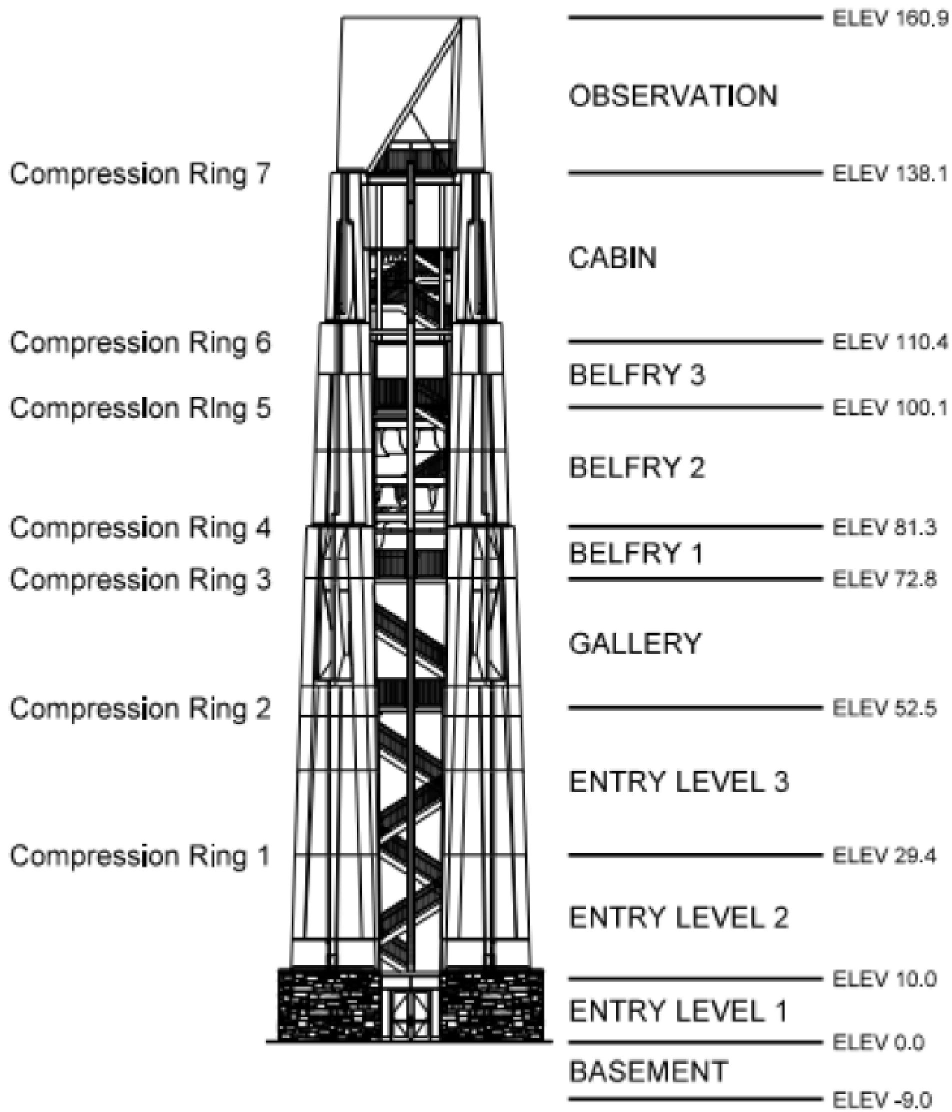


FIGURE 3 MOSER TOWER ELEVATION DIAGRAM LOOKING WEST

The tower includes an elevator which provides access from the basement to the gallery level at Elev. +52.5. Above this level, a series of steel columns are added as an internal support for the operation room and for the bells located within the belfry.

FIGURE 4 COMPRESSIVE RING AT ELEV. +81.3

1.1.1. Location of Surveyed Area and References

The corrosion investigation focuses on the reinforced concrete which form part of the overall framework of the structure. The framework for durability is based on the protocol set out in our Life 52™ inspection document which is divided into six (6) levels. A good basis for the condition assessment protocol is obtained with such a framework, because the framework divides the complete structure into modules, components and sub-components in a logical way.

Level 1: Objects

Classification of the complete structure, e.g. building, bridge, tunnel, etc.

Level 2: Module

The classification criterion for these modules is the logical set up of the structure, which yields the order of production. These main modules divide a structure into the largest units. A main module can have several, varying functions and is made up of different materials. A main module is a generic term comprising certain components.

Level 3: Component

A component fulfils a certain function as a unit but can consist of different materials. For each component the function within the whole object must be identified:

- load bearing, e.g. column;
- functional safety, e.g. hand rail;
- physical performance, e.g. expansion joints, noise barrier;
- protective measure, e.g. coating, roofing.

Level 4: Sub-component/ resistance

The sub-division has to be carried out with the target to identify sections with equal resistance at the surface and of the inner layers. The sub-components consist of the same materials (terracotta, brick, etc.) and are produced with the same design.

Level 5: Surface/ Environmental influences

Sub-components are divided into sections and areas, which are exposed to the same environmental influence and/ or stresses. This can be defined as a specific microclimate on or surrounding a structure which may lead to material degradation.

Level 6: Detail/ Material

Inner layers of a surface are regarded in depth. An identification of details perpendicular to the surface is made and a distinction is made for details which consist of the same material. With some structures, it is possible to take this as guidance and redefine the coding system taking into account the real needs of identification and the level of accuracy. For example, in some structures the Level 2 module level may not be necessary and thus it may be reasonable to combine Levels 2 and 3. In some other cases it may be possible to merge the Levels 4 and 5, etc.

The following coding system for the assessment is utlized:

S.008.100.250.300

S	Building (Module)
008	Wall (Component)
100	Concrete (Sub-Component)
250	Environment & Exposure Stress
300	Material Type (Concrete, Steel Reinforcement, Steel)

The following Life 52® references are used to identify the individual components.

Life 52™ Reference	Component	Sub Component Name
S.008.100.250.300	Pre-cast Wall [Main Panel]	Concrete
S.008.100.250.110	Pre-cast Wall [Main Panel]	Reinforcing Steel
Z.008.100.250.300	Pre-cast Fin [Non-load bearing]	Concrete
Z.008.100.250.110	Pre-cast Fin [Non-load bearing]	Reinforcing Steel
S.004.300.250.120	Compressive Ring	Steel
E.005.100.250.300	Foundation Column	Concrete
E.005.100.250.110	Foundation Column	Reinforcing Steel
S.004.100.300.300	Beam	Concrete
S.004.100.300.110	Beam	Reinforcing Steel

TABLE 1 COMPONENT REFERENCES

FIGURE 5 LOCATION MAP OF MOSER TOWER, NAPERVILLE-ILLINOIS

1.2. Classification of The Structure

This structure is categorized as a reinforced concrete building.

The structure type classification is built up from the following tables:

Location	**Distance from Coast**		
	< 1 Mile	**1 – 10 Miles**	**> 10 Miles**
Naperville, IL	Yes	Yes	Yes
	Temperature/Percentage		
	> 70 F (75% of the Year)	**> 70 F (50-70% of the Year)**	**> 70 F (less than 50% of the Year)**
			Yes
	Humidity Mean (Yearly)		
	> 65%	**40-65%**	**< 40%**
	Yes	Yes	

TABLE 2 BASIC EXPOSURE OF THE STRUCTURE

Function	**Industrial**	**Infrastructure**	**Commercial**
			Yes
Material	**Concrete**	**Masonry**	**Steel**
	Yes		
Technological	**Monolithic**	**Prefabrication**	**Masonry**
		Yes	
Size	**Super Structures**	**Single Component**	**Long Span/Single Story**
		Yes	
Important factors	**Critical Plant**	**Marine**	**Contaminants**
			Yes

TABLE 3 PRINCIPLES OF TYPES OF STRUCTURES

The previous two tables allow Echem to define the type, location, importance factors and exposure which enable a classification to be formed as follows:

Definition	Description	Rating	
Class I	High Risk Is defined as a structure with high importance in operations. A structure where high contaminants are present and located in a tropical type environment.	≥23	
Class II	Medium Risk Typical structure where certain principles could be high and individual factors need to be assessed	10 to 22	Yes
Class III	Low Risk All conditions and factors indicate a structure of low importance and in a non-aggressive environment.	≤9	

TABLE 4 CONDITION STATE RATINGS

The above classification is intended to provide an overall means to address service life. This allows us to look at temperature, contaminants and the location for assessment purposes. As seen above, Moser Tower falls under a medium risk category.

1.3. Condition State Classification of The Structure – Service Life

In order to understand the long-term durability of the structure a condition state rating system is utilized which defines the current condition of the structure with regard to its degradation. This then provides a time frame for critical failures and the ability to demonstrate how repairs will provide a condition state extension, or when the structure components require replacement. Service life extension can be demonstrated based on what method of repair is utilized.

Three types of **service life** have been defined (Sommerville 1986).

Technical service life is the time in service until a defined unacceptable state is reached, such as corrosion, safety level below acceptable, or failure of components. In typical situations, unacceptable limit states for corrosion can be related to percentage of concrete/masonry deterioration, percentage of section loss of steel or levels of contamination extending beyond industry specific thresholds.

Functional service life is the time in service until the structure no longer fulfills the functional requirements.

Economic service life is the time in service until replacement of the structure (or part of it) is economically more advantageous than keeping it in service.

Condition State	Section Loss (%)	Comments
5	< 1	Very Low
4	1 - 9	Low to Medium
3	10 - 17	Medium
2	18 - 25	Medium to High
1	>25	Very High

TABLE 5 INTERPRETATION OF CONDITION STATES FOR STRUCTURAL INTEGRITY FROM CORROSION

Life 52® provides the basis of the corrosion assessment by utilizing a five (5) state system based around section loss of the steel sub-component. As more steel section is lost the condition state changes from one state to another. This method of approach allows us to develop models where we can determine the amount of section loss with time ultimately changing states. When condition state 2 is met the sub component is said to be at its limit and any additional section loss will ultimately lead to obsolescence. In this instance, obsolescence is not necessarily defined as structurally deficient, but so far progressed in the deterioration curve that repair and rehabilitation become financially impractical to implement. The limit states are also influenced by the risk factors affecting the concrete, and the financial implications of the cost of deterioration to the owner.

Although the best approach is to be able to measure section loss of the actual component this is often impossible to carry out due to the physical amount of work required to access the sub-component. As a result of this our approach is to use a combination of inspection methods where we are able to create a site-specific condition state table utilizing the test results obtained. On this project we put together an inspection plan that incorporates our suite of corrosion techniques which are intended to be used to classify condition states for deterioration. By knowing the condition state of each sub-component repair work can be planned accordingly with the ultimate aim of being more cost effective.

At Moser Tower, the condition state classification of the reinforcement steel sub-component is based around corrosion rates [i-cos] and potentials [i-vos]. Table 6 provides the test measurement classification.

Condition State	Corrosion Rate (µm/yr)	Potential (mV) w.r.t. Cu/CuSO4	Comments
5	<0.1	>- 200	Very Low
4	0.1 – 1.16	-200 to -250	Low to Medium
3	1.16 – 3.34	-250 to -350	Medium
2	3.34 – 11.6	-350 to -500,	Medium to High
1	>11.6	< - 500	Very High

TABLE 6 CORROSION CONDITION STATES

It should be noted that Table 6 is unique to each structure and created based on results, visual observations and experience.

1.4. Project Objectives

Echem Consultants LLC (Echem) was engaged by ERA Engineering Consultants to carry out a Material Durability Assessment of the Moser Tower with the following scope and objective:

Our teams objective was to provide a material and corrosion-based condition assessment of the critical components of the tower which included reinforced concrete foundations, pre-cast panels and pre-cast fins. The testing program included an architectural assessment by Brush Architects as-built arrangement of the reinforcement layout, corrosion testing, chloride level, carbonation depths and petrographic analysis. By understanding where the structural components are within its service life allows for an appropriate repairs to be determined in the future.

SECTION 2 LABORATORY TESTING

Laboratory testing of materials obtained from at risk structures is a key step in understanding degradation. L-spec is the laboratory services of Life 52® and is broken down into three (3) types of inspection as follows:

- **L-met** Laboratory methods for diagnosing and discerning metals
- **L-con** Laboratory methods for understanding the strength and durability of concrete
- **L-mas** Laboratory methods for understanding the strength and durability of masonry

2.1. LABORATORY TESTING OF CONCRETE (L-CON)

The Laboratory Testing consists of a number of critical tests used to understand the concrete properties and durability. The test included within the project are as follows:

- L-asc – Acid-Soluble Chloride Test
- L-car – Carbonation Test
- L-pet – Petrographic Analysis
- L-clp –Chloride Ion Penetration

The objective with the Laboratory testing is to identify the current concrete's condition to understand if the structure meets required standards and to help provide repair options to deal with the expected conditions in the future.

2.2. Acid Soluble Chloride Analysis [L-asc]

Acid-soluble chloride determinations were performed in accordance with ASTM C1152 "Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete". The acid-based test provides the total chloride content including cast-in and free chlorides.

Testing was performed at two (2) depth intervals on four (4) samples extracted from three (3) pre-cast panels, one (1) pre-cast fin and from a reinforced concrete beam located at the entry level. In addition, one sample at 1.5 inches depth was extracted and tested from one (1) of the reinforced concrete columns located on the north side.

A sample of the grout used to fill the post-tensioned cable channel present within the pre-cast panel was removed and tested also. The depth intervals represent the total depth of the reinforcement from the surface of the concrete. [With the exception of the grout sample] The chloride test results are evaluated against threshold levels as detailed within the following table.

Chloride Ion Limits (ACI 318- vs ACI 222)			
Units in % by weight of cement	ACI 318	ACI 222	ACI 222
	Water Soluble	Acid Soluble	Water Soluble
Pre-stressed Concrete	0.06	0.08	0.06
Reinforced Concrete Exposed to Chlorides in Service	0.15	Not mentioned	Not mentioned
Reinforced Concrete Dry in Service	1.00	0.20	0.15
Reinforced Concrete Wet Conditions in Service	Not mentioned	0.10	0.08
Other	0.30	Not mentioned	Not mentioned

TABLE 7 CHLORIDE ION LIMITS

By assessing the transgression of chloride ions within the concrete, it is possible to determine when chloride thresholds have reached the embedded steel elements.

Location	Depth – in	Chloride Content: % by Weight of Cement*	Drawing Reference
South Beam – C1	1”	0.0974	X502
South Beam – C1	2”	0.1127	
Precast Panel - C2	1”	0.1232	
Precast Panel - C2	2”	0.158	
Precast Fin – C3	1”	0.0926	
Precast Fin – C3	2”	0.1754	
Precast Panel – C4	1”	0.1134	
Precast Panel – C4	2”	0.1844	
Precast Fin – C5	1”	0.1823	
Precast Fin – C5	2”	0.3055	
Grout Pocket – Precast Fin – C6	1”	0.1775	
Grout Pocket – Precast Panel – C7	2”	0.1051	
North-East Entry Level Column 3	1.5”	0.1816	

TABLE 8 ACID SOLUBLE CHLORIDE TEST RESULTS

*For this calculation, the cement content is assumed to be 14.37%.

Full detailed test results are included in Appendix B of this document and actual locations are shown on drawing X502.

2.3. Concrete Carbonation

Carbonation testing was completed at five (5) locations where dust samples were removed for chloride testing.

Carbon dioxide penetrates through the surface of concrete reacting with the alkaline components in the cement paste, mainly calcium Hydroxide [Ca(OH)2]. This process (carbonation) leads to a reduction of the pH-of the concrete which generally ranges between 9 to 14. The alkalinity of the concrete provides a passive layer on the steel protecting it from corrosion.

The depth of the carbonated surface layer is refer to as "the depth of carbonation". A solution of phenolphthalein is used for determining the depth of carbonation, which is directly applied on freshly drilled [exposed] concrete. The reduction of the pH-value is made visible by observing the color change where the phenolphthalein turns pink in non-carbonated concrete and remains colorless in carbonated concrete as shown in Figure 6.

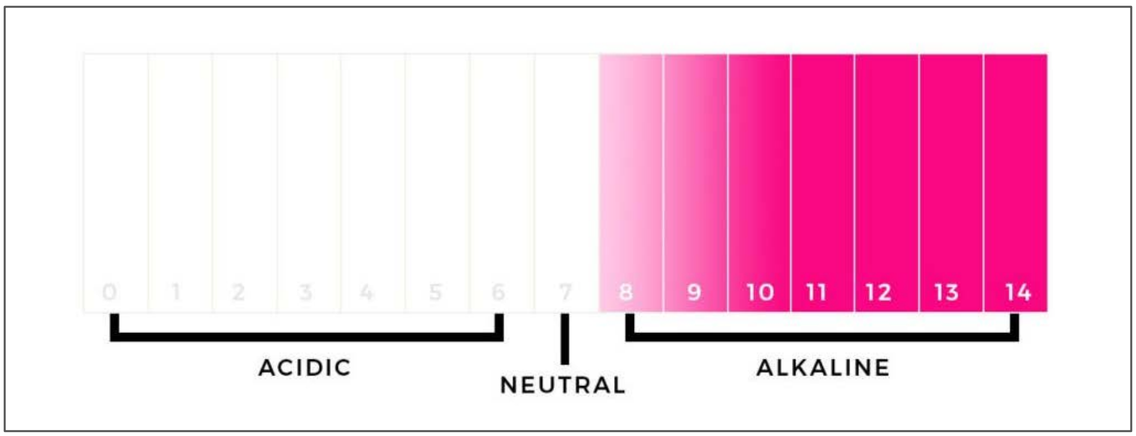


FIGURE 6 PHENOLPHTHALEIN INDICATOR

Location	Carbonation Depth [inches]	Picture after applying the Phenolphthalein Solution	Drawing Reference
South Beam C1	0”		X502
Precast Panel C2	0”		
Precast Fin C3	0”		
Precast Panel C4	0”		
Precast Fin C5	0”		

TABLE 9 CARBONATION TESTING RESULTS

The results showed that no carbonation of the concrete has occurred from the surface to two (2) inches in depth.

2.4. Petrographic Analysis

Petrographic analysis was carried out on four (4) samples in accordance with ASTM C856 - 17 Standard Practice for Petrographic Examination of Hardened Concrete.

Four samples were extracted as follow:

Sample ID	Location	Type/Layer	Drawing Reference
Sample 1	Pre-cast Fin [Elev. +81.3]	Pre-cast fin panel	X501
Sample 2	Pre-cast Panel	Pre-cast main panel	
Sample 3	Column [Entry Level]	Structural/ Reinforced concrete	
Sample 4	Entrance Wall	Reinforced concrete [sandblasted finish]	

TABLE 10 SAMPLE REFERENCE ID

The petrographic analysis of concrete is detailed under section 4.2 to 4.5 of the ASTM standard.

The following observations can be summarized from the analysis:

For the pre-cast concrete components, two cores were extracted for analysis as follow:

- **Sample 1:** Core extracted from a pre-cast fin panel located at Elev. +72.9 feet.
- **Sample 2:** Core extracted from a main pre-cast post-tensioned panel located at Elev. +52.2.

Both cores were extracted from the interior of the north elevation of the tower due to access, for actual location, refer to drawing x501.

Sample 1 and 2 are made of well-hydrated ordinary Portland cement with an estimated air content that ranges between 2% to 3% for sample 1 and 5% to 7% for sample 2. Both samples contain fine spherical air voids added in the mix as an additive to provide resistance for freeze-thaw. For sample 1, the air content [2%-3%] appears to be somewhat low to provide proper protection for freeze-thaw given the location of the structure [Naperville, Illinois].

According to the Köppen Climate Classification subtype, Naperville is defined as "Dfa". (Hot Summer Continental Climate) with an average temperature recorded in winter of 22.9° F. The following graph represents the annual variation in temperature and precipitation for Naperville, IL.[1]

[1] https://en.climate-data.org/location/17574/

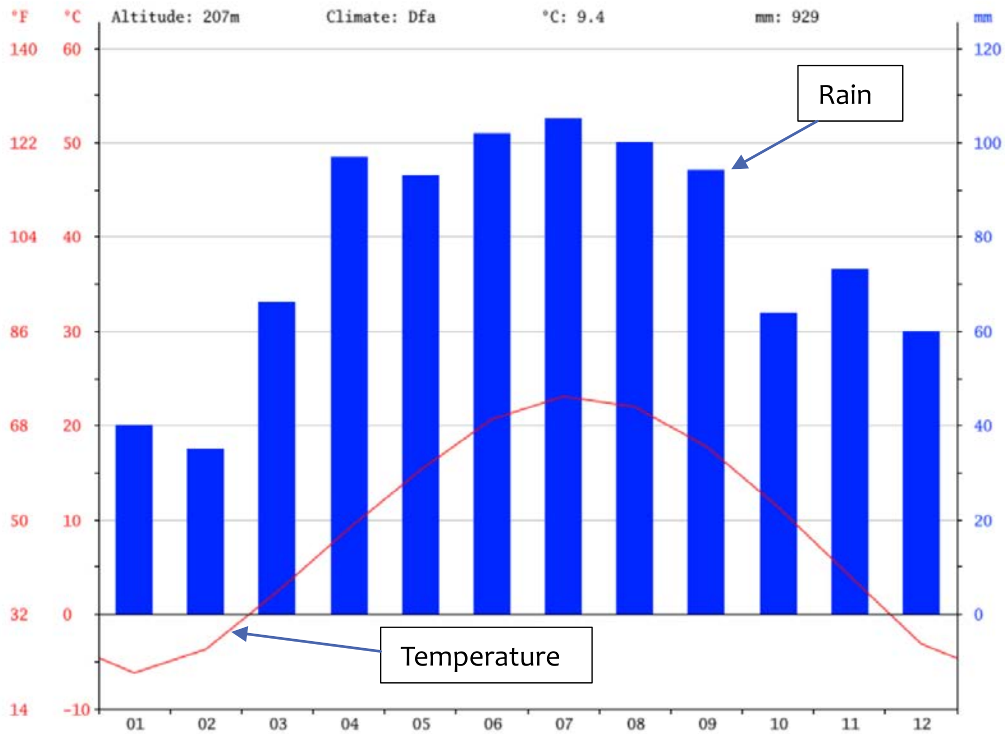


FIGURE 7 CLIMOGRAPH OF NAPERVILLE, IL.

The water/cement ratio for sample 1 and 2 is 0.50 and 0.55 with the maximum coarse aggregate size observed at 1/4" for Sample 1 and 5/16" inches for Sample 2.

Minimal carbonation depth is found on both samples indicating that the pH of the concrete is still in the alkaline region. This was also confirmed by in-situ carbonation testing described in Section 2.3 of this document.

For the cast-in place concrete components, two cores were extracted to conduct the petrographic analysis as follow:

- **Sample 3**: Core extracted from the reinforced concrete column located at the entry level. For detailed location, refer to Drawing X501.
- **Sample 4:** Core extracted from the north-side of the entry wall. This sample was extracted from the interior of the tower and presented a sand-blasted finish. For detailed location, refer to Drawing X501.

Sample 3 and 4 was comprised of well to moderately hydrated GGBF slag and blend with Portland cement with natural siliceous and calcareous sand with an estimated air content that ranges between 3% to 5%. Air-entrained voids are observed on both samples with the voids up to ½" Ø for sample 3 and 1/8" Ø for sample 4. Water/cement ratio is 0.40 for both samples.

Negligent depth of carbonation is detected on both samples. This is also confirmed from the in-situ carbonation testing described in Section 2.3 of this document.

A full detailed petrographic analysis is included in Appendix C of this document.

2.5. CHLORIDE ION PENETRATION [L-CIP]

Chloride Ion Penetration test was carried out on two (2) samples in accordance with ASTM 1202 – 17a Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration. The samples were extracted at the following locations:

Sample ID	Location	Type/Layer	Drawing Reference
Sample 1	Column Entrance	Structural / Reinforced Concrete	X501
Sample 2	Pre-cast Panel	Pre-cast main panel	

TABLE 11 SAMPLE REFERENCE ID

Originally developed by the Portland Cement Association, under a research program paid for by the Federal Highway Administration (FHWA), this test method is used to evaluate how rapidly chloride ions can penetrate through concrete by using electrical conductance. Electricity is used to accelerate the migration process, which is generally a slow process even in high water/cement ratio concrete due to the complexity of concrete. The total electrical charge passed (coulombs) has been found to be related to the resistance of the specimen to chloride ion penetration.

ASTM C1202 "Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration" is actually a test of electrical conductance, rather chloride permeability as is often stated. Electrical conductivity is related to the diffusion coefficient. In this test, a water-saturated concrete specimen, nominally 100mm diameter and 50mm thick, is positioned in a test cell (Figure 8) containing fluid reservoirs on both ends of the specimen. One reservoir is filled with a 3 % NaCl solution and the other with a 0.3N $NaOH^-$ solution. An electrical potential of 60 VDC is applied across the cell. The negative terminal of the potential source is connected to the electrode in the the NaCl solution and the positive terminal is connected to the electrode in the $NaOH^-$ solution. The negatively charged ions will migrate towards the positive terminal resulting in current flow through the specimen which is subsequently measured.

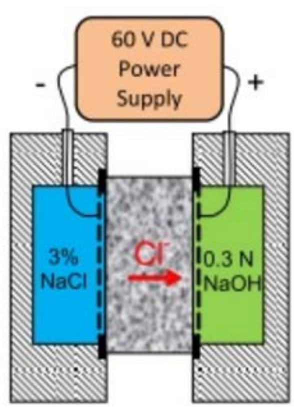


FIGURE 8 ASTM C1202 TEST CELL.

The more permeable the concrete, the more negative ions will migrate through the specimen, an a higher current will flow. The current is measured for a period of six [6] hours. The area under the curve of current versus time is determined, which represents the total charge or Coulombs passed across the specimen. This test method measures concrete resistivity. Resistance is calculated as volts divided by current. The Coulomb values are used for classifying the concrete as follows:

Charged Passed [Coulombs]	Chloride Permeability	Typical of
>4000	High	High W/C ratio (0.60)
2000 – 4000	Moderate	Moderate W/C ratio (0.40 – 0.50)
1000 – 2000	Low	Low W/C ratio (<0.40)
100 – 1000	Very Low	Latex-modified concrete or internally-sealed concrete
<100	Negligible	Polymer-impregnated concrete, Polymer Concrete

TABLE 12 ASTM 1202 CHLORIDE PERMEABILITY REFERENCE TABLE

The following results were obtained on the two test samples:

Sample ID	Location	Charge Passed [Coulombs]	Chloride Permeability
Sample 1	Column Entrance	1234	Low
Sample 2	Pre-cast Panel	1829	Low

TABLE 13 ASTM1202 TEST RESULTS

In accordance with Table 1, both samples belong to a low chloride permeability class. Test results for ASTM C1202 test are included within the Appendix of this document.

2.6. LABORATORY SUMMARY

Within the laboratory, the series of tests performed were designed to enable an evaluation of the concrete and its current levels of contamination and the risk for long term durability. Durability of concrete is primarily affected from corrosion of the embedded reinforcing steel based on chloride contamination and carbonation. In addition the concrete itself is subject typically to deterioration from its properties. To evaluate this an in depth petrographic is performed to understand the concrete itself.

The chloride levels in the cast-in place concrete components and in the pre-cast panels are negligible according to ACI222 thresholds [0.20 % by Weight of Cement]. Only one sample at the 2" pre-cast fin was over this threshold at a value of 0.30%. Further testing should be carried out at the fin pre-cast panels to establish if calcium chloride was used in the concrete-mix as a curing agent. During the field test, only one fin was tested due to limited access.

In addition, to the chloride levels, both cast-in place and pre-cast samples fit into a low chloride permeability class according to ASTM1202. This indicates a low permeability concrete that is less vulnerable to chloride ingress.

Insignificant carbonation depth is observed in all samples within the petrographic analysis and the in-situ carbonation test. This is expected due to the age of the structure and the speed at which carbon dioxide penetrates outdoor exposed concrete.

The petrographic analysis showed that both the pre-cast panels and the cast-in place samples appeared to be in good condition. Air-entrained voids are found in all samples as a protection for freeze-thaw. Water/cement ratio appears to be within expected design limit as well. [0.40 for the cast-in place concrete and 0.55 for the pre-cast components].

In addition to common issues the pre-cast samples showed no trace of delayed ettringite. Delayed ettringite is a common deterioration process regularly occurring in pre-cast components that have been cured at high temperature.

Section 3 IN-SITU INSPECTION (i-spec™)

Inspection of materials is one of the fundamental tools used in Echem's consultancy group. The in-situ inspection is referred to as i-spec™ which forms part of Life 52® network protocol to establish underlying conditions. Construction materials in the Life 52® network are broken down into four categories as follows:

- Metals
- Concrete
- Masonry
- Wood

As part of this project the in-situ inspection was to ascertain if the embedded reinforcing steel was corroding or if there was evidence of corrosion activity. To establish this Echem divided its testing program into the following methods.

- Measurement of Corrosion activity by half cell in accordance with ASTM C876 referred to as i-vos.
- Measurement of Corrosion rates by linear polarization resistance (LPR) in accordance with Rilem TC154 referred to as i-cos.
- Measurement of Concrete resistivity utilizing the four pin wenner method referred to as i-rom.

In addition to the corrosion testing Echem establish concrete cover and reinforcing steel distribution used to evaluate chlorides and carbonation depths allowing for a complete analysis of corrosion to be established.

3.1. CORROSION TESTING (i-COR)

The i-cor Non-Destructive Testing Investigation Life 52® protocol is identified as follows:

- **i-vos** Reinforcing Steel Voltage Measurements
- **i-cos** Reinforcing Steel Corrosion Rate Measurements
- **i-rom** Resistance of Concrete materials

I-cor testing groups together all of the corrosion test methods, allowing us to ascertain underlying conditions in a moderately semi-destructive manner. The ultimate objective is to categorize our sub-components into condition states where section loss can be predicted in the future.

3.1.1. CONTINUITY TESTING

Electrical continuity testing of the steel elements is carried out prior to any on site electrochemical testing procedure. Continuity is verified between the reinforcing steel across the structure and any internal electrical conduit or fittings and sub surface metal work.

It should be noted that electrical continuity testing does not confirm that all steel reinforcement is joined. Due to the many electrical parallel paths created within structures it is often found that electrical continuity exists even when by design it should not. The relevance of this is we cannot confirm instances where the reinforcement across a construction joint is still structurally sound or not.

The testing entails measuring the electrical resistance between the steel elements within the chosen areas of testing with an insulation resistance meter with a test current of 200 mA. Continuity is measured in Ohms (Ω) where acceptable readings less than 1 Ohm (<1Ω) are defined as electrically continuous. Should the readings be greater than 1 Ohm (>1Ω), it is generally accepted that the steel elements are discontinuous.

When testing for continuity all leads, connectors, and vise grips must be tested and their respective resistance values subtracted from the overall resistance between the disparate elements.

Continuity testing was performed at nine (9) locations as follows:

Location #	Location	Ohms (Ω)	Millivolts (mV)	Continuity: Yes/No
1 Cast-in Place Components	North-east Reinforced Concrete Beam Main bar to North-east Reinforced Concrete Beam stirrup	0.19	0.00	Yes
	North-east Beam Main bar to Column Main bar	0.22	0.00	Yes
	North-east Reinforced Concrete Beam Main bar to steel plate	0.28	0.00	Yes
	North-east Reinforced Concrete Beam Main bar to pre-cast panel at Elev. +29.4	128	>1	No
2 Pre-cast Elements	Pre-cast Panel 1 Horizontal Bar to Pre-cast Panel 1 Vertical Bar at North-east corner at Elev. +52.5	0.15	0.00	Yes
	Pre-cast Panel 1 to Pre-cast Panel 2 at North-east corner at Elev. +52.5	125	>1	No
3 Steel Compressive Ring	Steel Ring 1 at Elev. +29.4 to Steel Ring 2 at Elev. +52.5	0.10	0.00	Yes
	Steel Ring 1 at Elev. +29.4 to Steel Ring 3 at Elev. +72.8	0.15	0.00	Yes
	Steel Ring 1 at Elev. +29.4 to Steel Ring 4 at Elev. +81.3	0.15	0.00	Yes

TABLE 14 CONTINUITY TEST RESULTS

All reinforcement of the cast-in place components [reinforced concrete beams and columns] tested continuous within itself and within each cast-in place component tested.

The pre-cast panel tested continuous within itself and discontinuous from the cast-in place components and other pre-cast panels.

The seven [7] steel compressive rings are continuous within each other.

Discontinuity of the pre-cast panels is a common feature of this type of construction as each panel acts as a separate unit. The steel plate connected to the cast-in place component was also continuous.

3.1.2. CORROSION ACTIVITY (I-VOS)

The i-vos test program was carried out as a standalone test method and was also measured as part of measuring the corrosion rate of the steel which is measured at the start of the corrosion rate test. The unit of measurement for i-vos is voltage which is typically reported in millivolts (mV) and referred to as the half-cell potential. The absolute potential across a metal/solution cannot be determined by measurement and can only be measured with respect to a second electrode referred to as the reference electrode.

The Nernst equation indicates that a metal electrode potential is a function of the metal ion activity which is related to the metal ion concentration. As the metal ion concentration (Mn^+) increases the metal electrode potential becomes more electropositive.

$$E_m = E_{M^0} + \frac{RT}{nF} ln \frac{a^{(M^{n+})}}{a^{(M^0)}} \qquad \text{Nernst Equation}$$

In the i-cos corrosion rate test the measurement is referred to as the E_{corr} potential. E_{corr} measurements are made with a Silver/Silver chloride (Ag/AgCl) reference electrode where the standalone method use a $Cu/CuSo_4$ reference electrode. All potentials can be affected by temperature which is not generally taking into consideration when reviewing the readings on concrete structures when stable humidity readings are measured.

The corrosion rate of steel in concrete increases with increasing humidity and temperature, provided it is not immersed. The rate of corrosion may be expected to increase substantially with relatively small rises in temperature, but if the humidity remains constant then changes in temperature have little effect on the half-cell mapping values. The reason for this is most likely due to the anode and cathode reaction rates increase equally with temperature.

Half-cell mapping was undertaken within typical temperature and humidity conditions for the structure with a stable humidity and for this reason, no temperature adjustments were made for the E_{corr} values.

3.1.2.1. CORROSION ACTIVITY RESULTS

The i-vos test was performed at two (2) reinforced concrete beams located at the entry level as shown in Figure 9. A total of thirty-eight (38) E_{corr} readings were recorded as part of the i-cos measurements.

Measurements were recorded on a one (1) foot grid on the South and North internal face of the beams. Contoured maps of all data are included within the associated drawings X403.

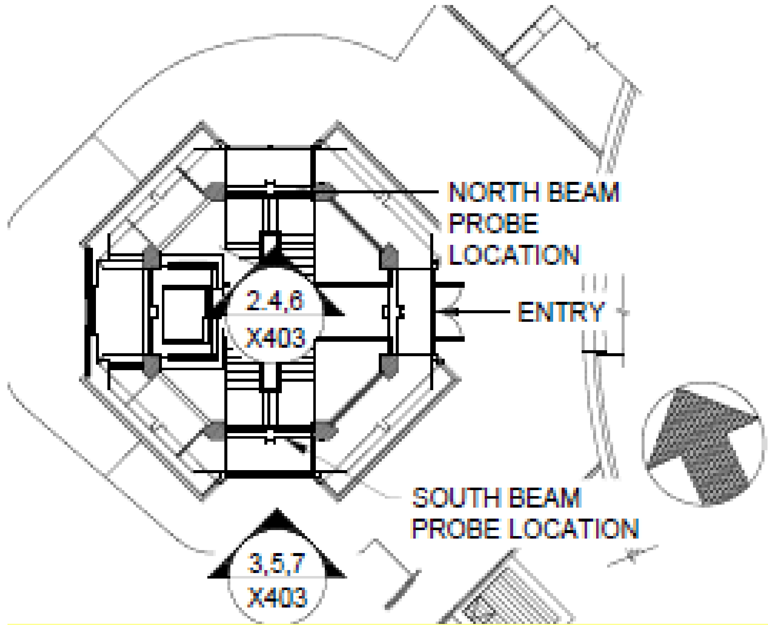


FIGURE 9 ENTRY LEVEL PLAN – TEST LOCATIONS

All data collected is evaluated in accordance with ASTM C876 "Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete" guidelines. In this standard, the following guidelines are given.

- X1.1.1 If potentials over an area are more positive than –200 V (w.r.t. $Cu/CuSo_4$), there is a greater than 90 % probability that no reinforcing steel corrosion is occurring in that area at the time of measurement.
- X1.1.2 If potentials over an area are in the range of –200 to –350 V (w.r.t. $Cu/CuSo_4$), corrosion activity of the reinforcing steel in that area is uncertain.
- X1.1.3 If potentials over an area are more negative than –350 V (w.r.t. $Cu/CuSo_4$), there is a greater than 90 % probability that reinforcing steel corrosion is occurring in that area at the time of measurement.

i-vos Potentials [$Cu/CuSo_4$]				
	>-200 mV	**-200 to -350 mV**	**<-350 mV**	
North Beam	20	0	0	**Count**
	100	0	0	%
	Maximum	**Minimum**	**Mean**	**Standard Deviation**
	-33.6 mV	126 mV	55.78 mV	45.28 mV
South Beam	**>-200 mV**	**-200 to -350 mV**	**<-350 mV**	
	18	0	0	**Count**
	100	0	0	%
	Maximum	**Minimum**	**Mean**	**Standard Deviation**
	-38 mV	41 mV	11.82 mV	17.74 mV

TABLE 15 I-VOS SUMMARY RESULTS

Potential (mV) w.r.t. Cu/CuSo4	Probability of Corrosion	No. of Readings (38)	(%)
>-200	90% Probability of No Corrosion	38	100
-200 to -350	Uncertain	0	0
<-350	90% Probability of Corrosion	0	0

TABLE 16 CORROSION ACTIVITY (E_{corr}) SUMMARY ALL LOCATIONS [ASTM C876]

In accordance with ASTM C876, all of the readings [100%] have a 90% probability that no corrosion exists on the structure as shown in the table above. When measuring half cell potentials there is always a risk of misinterpretation of results based on moisture levels. When carrying out a comprehensive assessment, it is always necessary to carry out corrosion rates as well to validate the potentials measured.

3.1.3. CORROSION RATES (I-COS)

The i-cos test program provides the corrosion rate measurement of current measured in an electrochemical cell. Embedded metallic sub-components corrode by means of electrochemical reactions at the interface between the metal and the electrolyte solution [concrete].

Corrosion normally occurs at a rate determined by an equilibrium between opposing electrochemical reactions. The harmful reaction is the anodic reaction (oxidation), in which a metal is oxidized, releasing electrons into the metal and the other is the cathodic reaction (reduction), in which a solution species (often Oxygen (O_2) or Hydrogen ions (H^+)) is reduced, removing electrons from the metal. When these two reactions are in equilibrium, the flow of electrons from each reaction is balanced, and no net electron flow (electrical current) occurs. The two reactions can take place on one metal or on two dissimilar metals (or metal sites) that are electrically connected.

The i-cos measurements are made by using a hand-held sensor on the surface of the structure and an electrical connection to the steel. Measurements are then made for a short period of time (< 1 minute) where at the end of this time period readings of E_{corr} and i_{corr} are logged.

The values of i_{corr}, are then used to assess the rate of degradation of the steel and the predictive condition state. This measurement cannot give information on the actual loss of steel cross section currently which, can only can be assessed by means of direct visual observation and manual measurement.

3.1.3.1. CORROSION RATE RESULTS

The i-cos test was performed at fifteen (15) locations across the structure as follows:

- A total of twelve (12) pre-cast main panels were tested. Measurements were recorded on a grid of one-foot centers for a total of three-hundred and sixty-eight (368) readings.
- A total of one (1) pre-cast fin panel was tested. Measurements were recorded on a grid of one-foot centers for a total of fourteen (14) readings.

- A total of two (2) reinforced concrete beams were tested. Measurements were record on the internal face of each beam based on a one (1) foot grid for a total of thirty-eight (38) readings [Only one face of the beam was accessible].

Contoured maps of all data are included within the associated drawings X401, X402, X403, and X404. All data collected is evaluated in accordance with RILEM TC 154-EMC "Test methods for on-site corrosion rate measurement of steel reinforcement in concrete by means of the polarization resistance method". In the Rilem standard, the following guidelines are given in the document as Table 17.

I_{corr} (µA/cm²)	V_{corr} (µm/yr)	Corrosion Level
≤ 0.1	≤ 1	Negligible
0.1 – 0.5	1 - 5	Low
0.5 – 1	5 - 10	Medium
> 1	>10	High

TABLE 17 CORROSION RATE ASSESSMENT – RILEM TC 154-EMC

Within the Life 52™ service life guidelines we have further developed this from field results and laboratory experiments to create the following condition state assessment.

I_{corr} (µA/cm²)	V_{corr} (µm/yr)	Condition State	Comment
≤ 0.01	≤ 0.1	5	Very Low
0.01 – 0.116	0.1 – 1.16	4	Low to Medium
0.116 – 0.579	1.16 – 5.79	3	Medium
0.579 – 1.158	5.79 - 11.6	2	Medium to High
> 1.158	>11.6	1	Very High

TABLE 18 CORROSION RATE CONDITIONS

The tables below provide the data collected at all locations tested and categorized in accordance with Table 18.

Location 1: Precast Panel, Elev. 110.4, Exterior			
Condition State	Corrosion Rate (µm/yr)	No. of Readings (12)	%
5	<0.1	1	8.33
4	0.1-1.16	7	58.33
3	1.16-3.34	4	33.33
2	3.34-11.6		
1	>11.6		

TABLE 19 I-COS RESULTS LOCATION 1 PRECAST PANEL, ELEV. 110.4, EXTERIOR

Location 2: Precast Panel, Elev. 81.3, Exterior			
Condition State	Corrosion Rate (µm/yr)	No. of Readings (12)	%
5	<0.1		
4	0.1-1.16	7	58.33
3	1.16-3.34	5	41.67
2	3.34-11.6		
1	>11.6		

TABLE 20 I-COS RESULTS LOCATION 2 PRECAST PANEL, ELEV. 81.3, EXTERIOR

Location 3: Precast Panel, Elev. 81.3, Exterior			
Condition State	Corrosion Rate (µm/yr)	No. of Readings (12)	%
5	<0.1		
4	0.1-1.16	12	100
3	1.16-3.34		
2	3.34-11.6		
1	>11.6		

TABLE 21 I-COS RESULTS LOCATION 3 PRECAST PANEL, ELEV. 81.3, EXTERIOR

Location 4: Precast Panel, Elev. 81.3, Exterior			
Condition State	Corrosion Rate (µm/yr)	No. of Readings (12)	%
5	<0.1		
4	0.1-1.16	11	91.67
3	1.16-3.34	1	8.33
2	3.34-11.6		
1	>11.6		

TABLE 22 I-COS RESULTS LOCATION 4 [LEFT]PRECAST PANEL, ELEV. 81.3, EXTERIOR

Location 5: Pre-Cast Panel, Elev. 52.6, Exterior			
Condition State	Corrosion Rate (µm/yr)	No. of Readings (16)	%
5	<0.1		
4	0.1-1.16	11	68.75
3	1.16-3.34	5	31.25
2	3.34-11.6		
1	>11.6		

TABLE 23 I-COS RESULTS LOCATION 5 [RIGHT]PRECAST PANEL, ELEV. 52.6, EXTERIOR

Location 6: Top Precast Panel, Elev. 52.6., Interior			
Condition State	Corrosion Rate (µm/yr)	No. of Readings (60)	%
5	<0.1	1	1.67
4	0.1-1.16	36	60
3	1.16-3.34	22	36.67
2	3.34-11.6	1	1.67
1	>11.6		

TABLE 24 I-COS RESULTS LOCATION 6 [TOP]PRECAST PANEL, ELEV. 52.6, INTERIOR

Location 7: Bottom Precast Panel, Elev. 52.6., Interior			
Condition State	Corrosion Rate (µm/yr)	No. of Readings (16)	%
5	<0.1		
4	0.1-1.16	17	42.5
3	1.16-3.34	23	57.5
2	3.34-11.6		
1	>11.6		

TABLE 25 I-COS RESULTS LOCATION 7 [BOTTOM]PRECAST PANEL, ELEV. 52.6, INTERIOR

Location 8: Top Pre-Cast Panel, Elev. 52.6., Interior			
Condition State	**Corrosion Rate (µm/yr)**	**No. of Readings (48)**	**%**
5	<0.1		
4	0.1-1.16	27	56.25
3	1.16-3.34	20	41.67
2	3.34-11.6	1	2.08
1	>11.6		

TABLE 26 I-COS RESULTS LOCATION 8 [TOP] PRECAST PANEL, ELEV. 52.6, INTERIOR

Location 9: Bottom Pre-Cast Panel, Elev. 52.6., Interior			
Condition State	**Corrosion Rate (µm/yr)**	**No. of Readings (72)**	**%**
5	<0.1	3	4.17
4	0.1-1.16	23	31.94
3	1.16-3.34	35	48.61
2	3.34-11.6	10	13.89
1	>11.6	1	1.39

TABLE 27 I-COS RESULTS LOCATION 9 [BOTTOM] PRECAST PANEL, ELEV. 52.6, INTERIOR

Location 10: Top Pre-Cast Panel, Elev. 72.9., Interior			
Condition State	**Corrosion Rate (µm/yr)**	**No. of Readings (28)**	**%**
5	<0.1	2	7.14
4	0.1-1.16	24	85.71
3	1.16-3.34	1	3.57
2	3.34-11.6	1	3.57
1	>11.6		

TABLE 28 I-COS RESULTS LOCATION 10 [TOP]PRECAST PANEL, ELEV. 72.9, INTERIOR

Location 11: Bottom Precast Panel, Elev. 72.9., Interior			
Condition State	**Corrosion Rate (µm/yr)**	**No. of Readings (28)**	**%**
5	<0.1	1	3.57
4	0.1-1.16	15	53.57
3	1.16-3.34	8	28.57
2	3.34-11.6	4	14.29
1	>11.6		

TABLE 29 I-COS RESULTS LOCATION 11 [BOTTOM]PRECAST PANEL, ELEV. 72.9, INTERIOR

Location 12: Bottom Precast Panel, Elev. 72.9., Interior			
Condition State	**Corrosion Rate (µm/yr)**	**No. of Readings (28)**	**%**
5	<0.1	1	3.57
4	0.1-1.16	26	92.86
3	1.16-3.34	1	3.57
2	3.34-11.6		
1	>11.6		

TABLE 30 I-COS RESULTS LOCATION 12 [BOTTOM] PRECAST PANEL, ELEV. 72.9 INTERIOR

Location 13: Precast Fin, Elev. 72.9			
Condition State	**Corrosion Rate (µm/yr)**	**No. of Readings (14)**	**%**
5	<0.1	2	14.29
4	0.1-1.16	10	71.43
3	1.16-3.34	2	14.29
2	3.34-11.6		
1	>11.6		

TABLE 31 I-COS RESULTS LOCATION 13 PRECAST FIN, ELEV. 72.9

Summary I-cos, All Locations				Time Period
Condition State	Corrosion Rate (µm/yr)	No. of Readings (392)	%	
5	<0.1	10	0.00	No Corrosion
4	0.1-1.16	237	62.04	15 - 25
3	1.16-3.34	127	33.25	10 – 15 Years
2	3.34-11.6	17	4.45	2 – 10 Years
1	>11.6	1	0.00	< 2 Years

TABLE 32 I-COS RESULTS ALL LOCATIONS

Summary I-cos All Locations Precast Elements					
Test Area	Max	Min	Mean	Standard Deviation	Condition State based on Mean
Precast Elements	15	0.0	1.2	1.3	3

TABLE 33 I-COS STATS SUMMARY RESULTS ALL LOCATIONS

Table 32, can be summarized by stating that 62% of the readings recorded on the pre-cast panels are at condition state 4 Low to medium corrosion activity. It should be noted that at the high end of this condition state a corrosion rate of 1.16µm/yr^{-1} would take 100 years to lose 0.1 mm section loss, however the corrosion product would produce a volume six times this which inevitably would crack the concrete increasing this corrosion rate to much higher levels. This has to be taken into consideration when reviewing the long-term performance model.

At condition state 3 we have a further 33% making a total of 95% at condition state three and four which has a deterioration prediction of between 10 to 25 years before corrosion distress is caused. The remaining 5% of the readings belongs to condition state 2 having a medium-high corrosion rate which is expected to show failure within 10 years. No visual distress was noticed during the field work which confirms the majority of the readings fall within condition state 4 as tested.

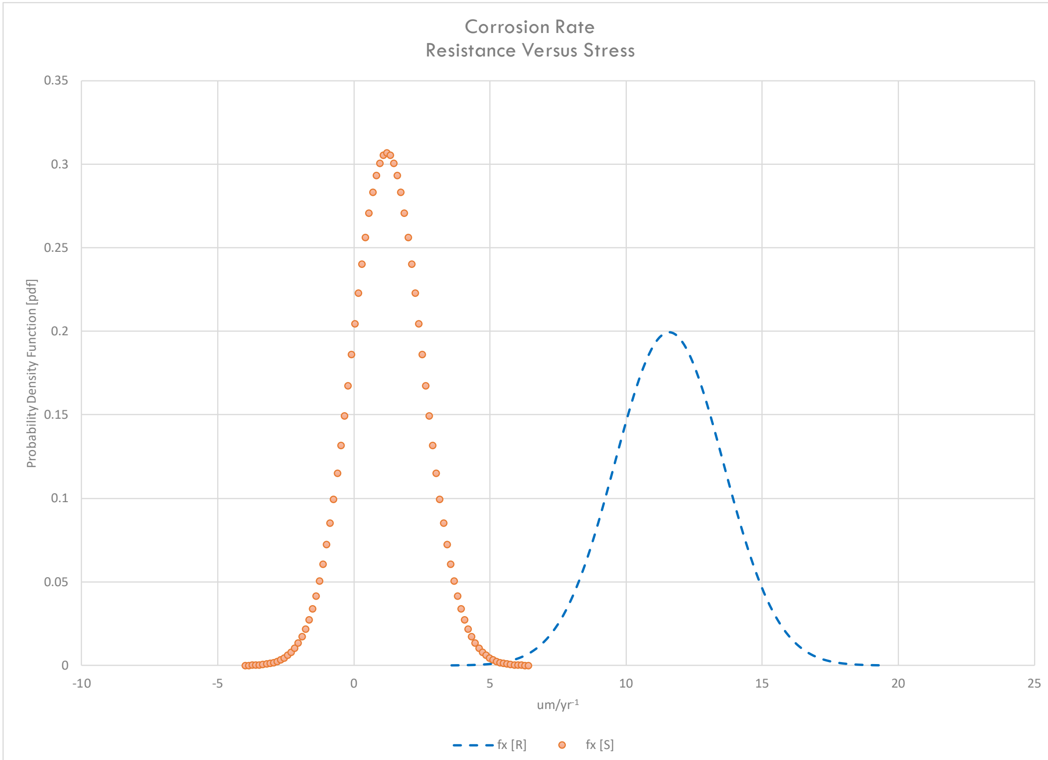


FIGURE 10 CORROSION LEVELS PRECAST RESISTANCE V STRESS

North Beam			
Condition State	Corrosion Rate (µm/yr)	No. of Readings (16)	%
5	<0.1		
4	0.1-1.16	20	100.00
3	1.16-3.34		
2	3.34-11.6		
1	>11.6		

TABLE 34 I-COS RESULTS LOCATION NORTH BEAM

South Beam			
Condition State	Corrosion Rate (µm/yr)	No. of Readings (16)	%
5	<0.1		
4	0.1-1.16	15	83.33
3	1.16-3.34	1	5.56
2	3.34-11.6	2	11.11
1	>11.6		

TABLE 35 I-COS RESULTS LOCATION WEST BEAM

Summary I-cos					
	Max	Min	Mean	Standard Deviation	Condition State based on Mean
Beam North & South	4.41	0.1	0.50	0.90	4

TABLE 36 I-COS STATISTICS ALL BEAM LOCATIONS

Table 34 and Table 35, can be summarized by stating that 100% of the readings recorded on the North beam and 83% on the South beam are at condition state 4 Low to medium corrosion activity.

At condition state 3 we have a further 5.5% on the South beam making a total of 88.5% at condition state three and four which has a deterioration prediction of between 10 to 25 years before corrosion distress is caused.

The remaining 11% of the readings belongs to condition state 2 having a medium-high corrosion rate which is expected to show failure within 10 years. No visual distress was noticed during the field work which confirms the majority of the readings fall within condition state 4 as tested.

It was visually noted by the team that water splashes onto these areas causing an excessive amount of moisture. This was confirmed by the temperature and humidity measurements taking from site.

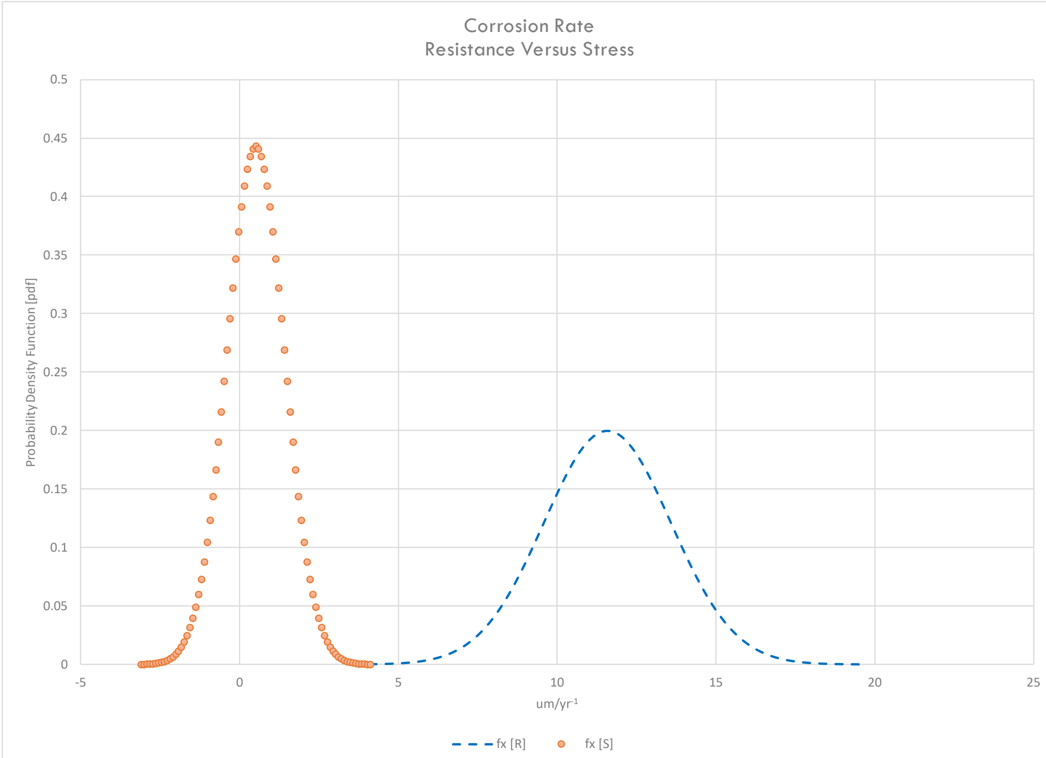


FIGURE 11 CORROSION LEVELS BEAM RESISTANCE V STRESS

3.1.4. CONCRETE RESISTIVITY (I-ROM)

Electrical resistivity is defined as the electrical resistance of a cube of unit size. In this standard definition, a voltage is applied between opposite faces of the cube and the resulting electrical current is measured. Resistance is then determined using Ohm's Law.

The electrical resistivity is an indirect measure of the porosity and the connectivity of the pore structure. Concrete resistivity in its basic definition is a material property, which is influenced by the environment but is independent of geometry.

High degrees of water saturation (wet material) and more frequent, larger pores in the concrete matrix cause lower resistivity. For a constant moisture content, the resistivity is increased by a lowering water/cement-ratio, allowing longer curing periods and additives such as slag, fly ash or silica fume.

The resistivity increases if the material dries out or when it carbonates. The sole effect of a decrease in resistivity due to an ingress of chlorides, leading to a more conductive pore solution, is relatively small. Nevertheless, due to the hygroscopic behavior of chlorides water is retained, which will enhance the effect. A temperature increase causes a decrease of resistivity. This is based on the influence on ion mobility, ion-ion and ion-solid interactions.

The density of a material affects resistivity, as does moisture content. Should a material be extremely dense, less current is likely to flow through the material to the underlying steel. If a material has a glazed or non-porous surface, this too can affect resistivity. Moisture content affects the rate of resistivity, and concrete generally has 2% moisture content in a dry environment. Unusually wet conditions will give higher resistivity readings which is counterintuitive. If the water is highly saline the test apparatus yields extremely conductive readings.

Resistivity can be affected by the following:

- Embedded reinforcement with cover depths less than the probing space
- Concrete admixtures containing conductive materials such as calcium chloride or calcium nitrite
- Variations in relative humidity in the concrete
- Cracks within the test area

Low resistivity values do not necessarily imply that active corrosion is occurring though low values indicate high rates of corrosion can occur if sufficient amounts of oxygen are present.

Resistivity measurements can be used to estimate the likelihood of corrosion, as seen in Table 37. When the electrical resistivity (ρ) of the concrete electrolyte is low, the likelihood of corrosion increases. Empirical tests have arrived at the following typical values for the measured resistivity from the four pin Wenner test which can be used to determine the likelihood of corrosion.

KΩ/cm²	Corrosion Rate Correlation
>20	Low
10-20	Moderate
5-10	High
<5	Very High

TABLE 37 – TYPICAL CORROSION RISKS ASSOCIATED WITH RESISTIVITY ACCORDING TO BROOMFIELD ET AL.

On the contrary, carbonated, aged concrete, concrete in internal spaces, and masonry will have higher resistivity values, even though corrosion may be occurring. Similar to voltage differences, gradients should be noted as electrochemical differences in the electrolyte will drive corrosion activity.

3.1.4.1. CONCRETE RESISTIVITY RESULTS

The i-rom test was performed at fifteen (15) locations across the structure as follows:

- Twelve (12) locations on the precast main panels were tested with measurements recorded on a one-foot grid for a total of three-hundred and sixty-eight (368) readings.
- One (1) precast fin was tested with measurements recorded on a one-foot grid for a total of fourteen (14) readings.
- Two (2) reinforced concrete beam were tested with measurements recorded on a one-foot grid for a total of thirty-eight (38) readings. NOTE: Only one face of the beam was accessible for testing.

Contoured maps of all data and exact locations are included within the associated drawings X401, X402, X403, and X404.

Resistivity Results kΩ/cm²				
Test Area	Max	Min	Mean	Standard Deviation
Location 1 Precast Main Panel	227	69	167	44
Location 2 Precast Main Panel	201	41	94	41
Location 3 Precast Main Panel	252	85	185	60
Location 4 Precast Main Panel	261	68	147	58
Location 5 Precast Main Panel	621	40	177	147
Location 6 Precast Main Panel	634	36	122	86
Location 7 Precast Main Panel	204	64	110	32
Location 8 Precast Main Panel	412	89	191	71
Location 9 Precast Main Panel	559	80	145	67
Location 10 Precast Main Panel	352	17	133	72
Location 11 Precast Main Panel	293	51	149	60
Location 12 Precast Main Panel	1354	120	474	280
Location 13 Precast fin	1200	218	461	242
North Beam First Floor	844	145	357	203
South Beam First Floor	232	66	158	46

TABLE 38 TEST AREA RESISTIVITY RESULTS

Resistivity Summary kΩ/cm²					
Test Area	Max	Min	Mean	Standard Deviation	Corrosion Rate Correlation
All	1354	17	188	156	Low

TABLE 39 RESISTIVITY SUMMARY RESULTS

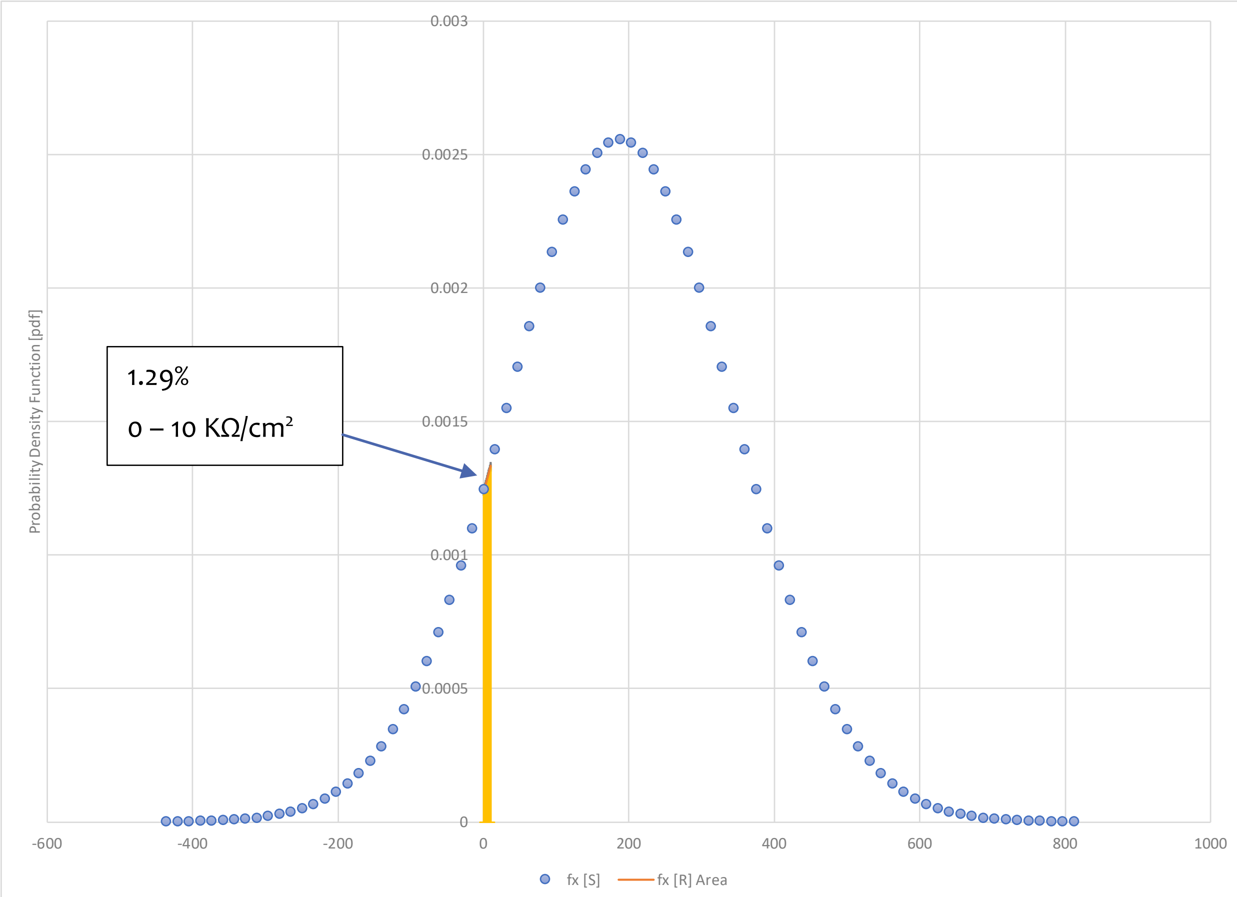


FIGURE 12 RESISTIVITY SUMMARY PROBABILITY DENSITY FUNCTION

When analyzing the resistivity data the amount of data we would expect to fall within a very high range [< 5 kΩ/cm^2] on the overall structure is less than 1%.

3.1.5. TEMPERATURE & HUMIDITY

Temperature and humidity readings were recorded over a period of forty-eight (48) hours at three (3) locations around the structure. Readings were recorded of the internal conditions of the concrete at two (2) locations by embedding the temperature and humidity sensor into a drilled hole approximately six (6) inches in the concrete and sealing it with silicone. Ambient condition were recorded at the entry area of the tower.

The following Table 40 and Table 41 provides the individual statistics of all three (3) probes. The results are shown graphically in Figure 14, Figure 15, Figure 16, and Figure 17.

		Temperature (F)			
Test Location	**Drawing Reference**	**Average**	**Min**	**Max**	**Standard Deviation**
Embedded TH1 [Pre-cast concrete panel]	X502	44.77	42.38	46.76	1.28
Embedded TH2 [Reinforced Concrete Beam Entry Level]		45.56	42.47	48.42	1.57
TH3 Ambient [Entry Level]		46.00	40.63	58.32	2.97

TABLE 40 SUMMARY OF STATISTICAL ANALYSIS OF TEMPERATURE SENSORS

Test Location	**Drawing Reference**	**Relative Humidity (%)**			
		Average	**Min**	**Max**	**Standard Deviation**
Embedded TH1 [Pre-cast concrete panel]	X502	98.04	92.40	98.95	0.81
Embedded TH2 [Reinforced Concrete Beam Entry Level]		100	100	100	0.00
TH3 Ambient [Entry Level]		79.39	59.03	92.80	8.46

TABLE 41 SUMMARY OF STATISTICAL ANALYSIS OF HUMIDITY SENSORS

In reviewing the statistics, a number of observations can be made with regard to the temperature and humidity as follows:

Location TH1 [Embedded at Precast panel]

This probe was installed in a pre-cast panel located on the entry level on the south elevation interior at +12.

The amount of temperature variation was four [4] degrees Fahrenheit indicating an area unaffected by atmospheric conditions which varied by eighteen [18] degrees Fahrenheit over the same time period.

The humidity data also showed no correlation to atmospheric conditions as the embedded humidity never fell under 92% while atmospherically the minimum humidity was recorded at 59% and varied considerably during the test period.

Location TH2 [Embedded at Beam]

This probe was installed in the south beam located on the entry level on the south elevation interior at +12.

The amount of temperature variation was six [6] degrees Fahrenheit indicating an area unaffected by atmospheric conditions which varied by eighteen [18] degrees Fahrenheit over the same time period.

The humidity data also showed no correlation to atmospheric conditions as the embedded humidity was constantly at 100% while atmospherically the minimum humidity was recorded at 59% and varied considerably during the test period.

During periods of rain recorded on Wednesday, March 28th, 2018 during the field work water run off was observed at this area.

Location TH3 [Atmospheric]

This probe was located on the entry level on the south elevation interior at +12 on the inside of the structure. This probe is used to evaluate embedded conditions during the test period.

The amount of temperature variation was eighteen [18] degrees Fahrenheit much larger than that recorded internally.

The humidity data also showed large differences than that recorded by the internal probes.

FIGURE 13 EMBEDDED TEMPERATURE HUMIDITY LOCATIONS

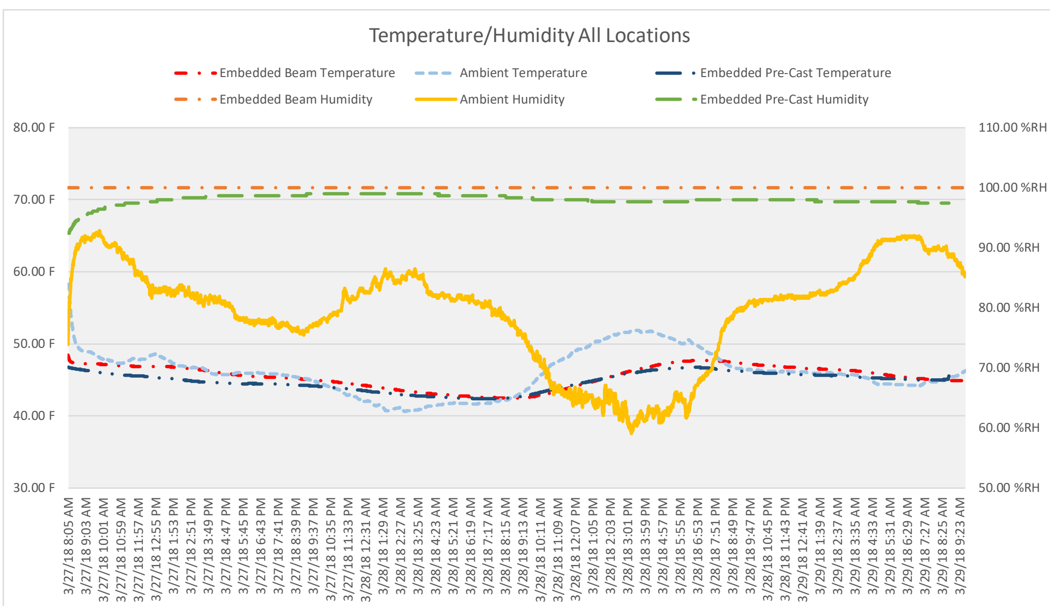


FIGURE 14 EMBEDDED TEMPERATURE HUMIDITY ALL LOCATIONS

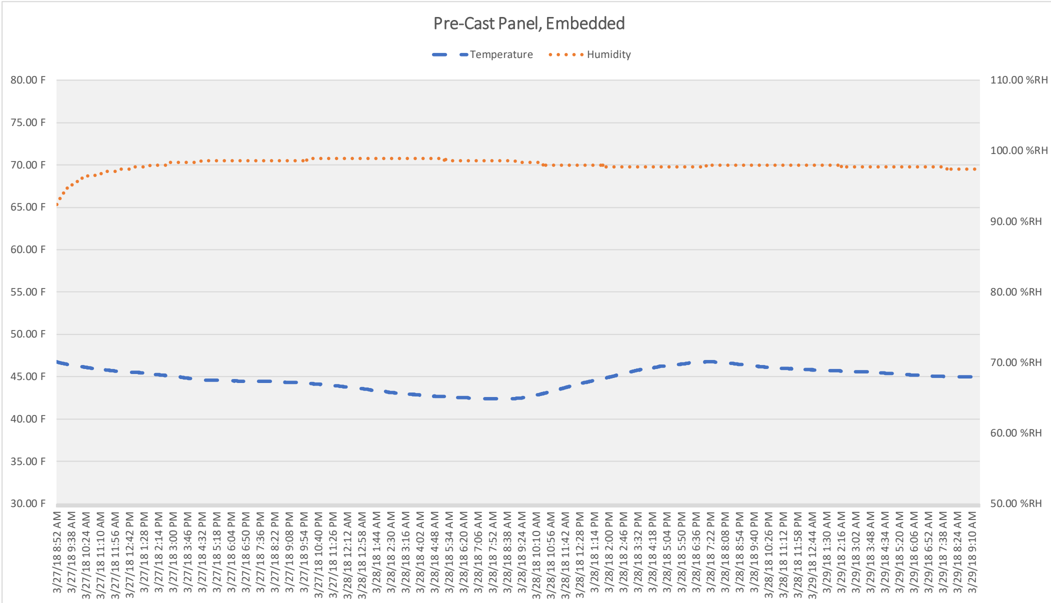


FIGURE 15 EMBEDDED TEMPERATURE HUMIDITY PROBE PRE-CAST PANEL [TH1]

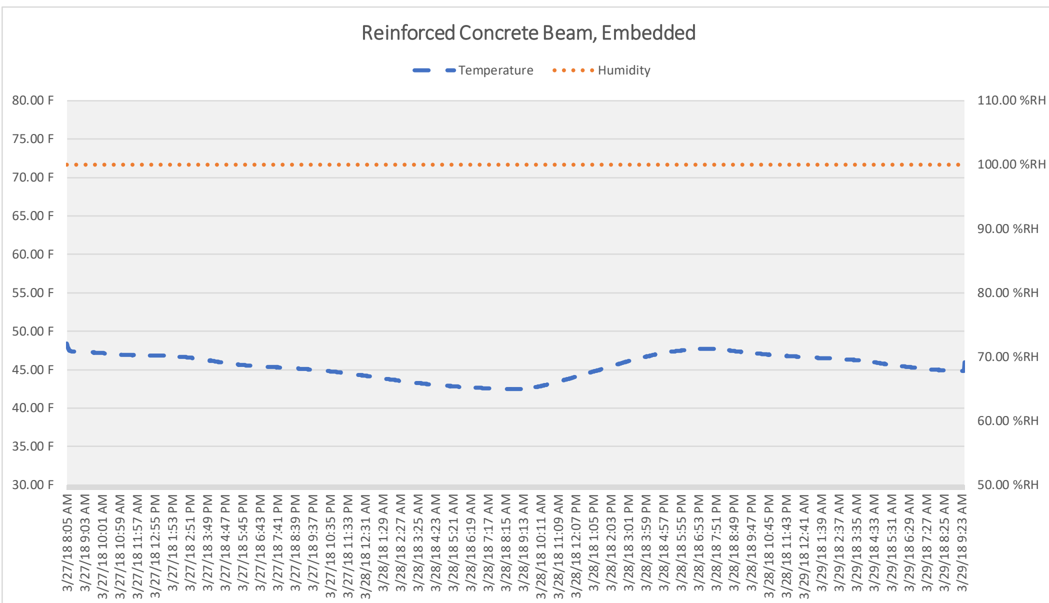


FIGURE 16 EMBEDDED TEMPERATURE HUMIDITY PROBE REINFORCED CONCRETE BEAM ENTRY LEVEL [TH2]

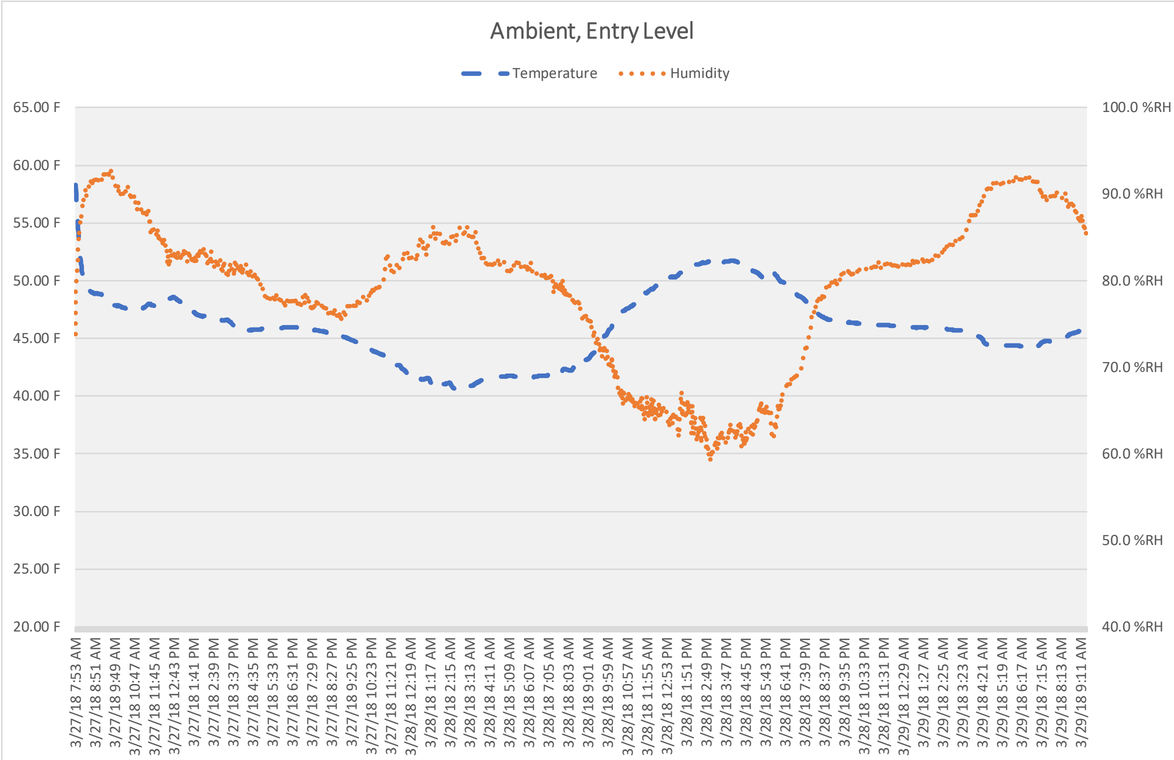


FIGURE 17 AMBIENT TEMPERATURE HUMIDITY [ENTRY LEVEL]

3.1.6. CORROSION TESTING SUMMARY

Sixty-two [62] percent of the corrosion rate measurements at the thirteen (13) precast panels were at condition state 4 indicating a time period of between 15 to 25 years for initiation. A further thirty-three [33] percent totaling ninety five [95] percent of the structure was at condition state 3 with a time period of 10 – 15 years before initiation. The remainder fell under condition state 2 where a time period of 2 to 10 years is expected.

This data basically is demonstrating a structure which is performing adequately for its age and the small amount of low performance data outside this is most likely due to cracks which ultimately removes any resistance the concrete has to corrosion and is not uncommon in all structures. Low corrosion activity was also confirmed by the electrical resistivity values collected at the same locations and supports this analysis.

Both the North and South beams tests showed a similar status, although the North Beam showed very low corrosion activity, typical of a saturated condition identified by the embedded temperature and humidity probe in the South beam

During the field work, it was observed that several of the pre-cast panels and the reinforced concrete beams are directly exposed to the environment with no presence of any flashing and minimal waterproofing membrane as shown in Figure 21 and Figure 22.

Both embedded probes show excessive level of moisture with an average humidity of 98.04% for TH1 and 100% for TH2. The probes were installed for a period of forty-eight (48) hours starting on Tuesday, 27th March 2018. During the time period of recording there were periods of rain recorded which would have contributed to the high moisture levels, however it does demonstrate the concretes remains high in humidity beyond the period of rain fall. This is not uncommon and understanding the availability of oxygen is the most important ingredient when assessing the corrosion risk when this condition exist.

The most overlooked factor in corrosion reactions is oxygen availability. This includes gaseous and dissolved oxygen supplied by the environment. When there is insufficient oxygen corrosion rates are drastically reduced even in the presence of excessive chloride levels and moisture. This is typically the case in a submerged marine pile which is exposed to a constant source of chlorides but an insufficient amount of oxygen.

In these instances, the rate of corrosion is limited by the slow rate of oxygen diffusion which is the limiting factor for the current density. Oxygen diffusion models are often based on the conservation law of oxygen in concrete.

Figure 18 provides the amount of oxygen available from water to air. As oxygen is consumed in the corrosion reaction it can be calculate by Faraday's law as shown below:

$$Q_{O2} = -\emptyset S \frac{M_{O_2} i_{corr}}{Z_{O_2} F} \cdot \frac{A_{bar}}{V_{elem}}$$

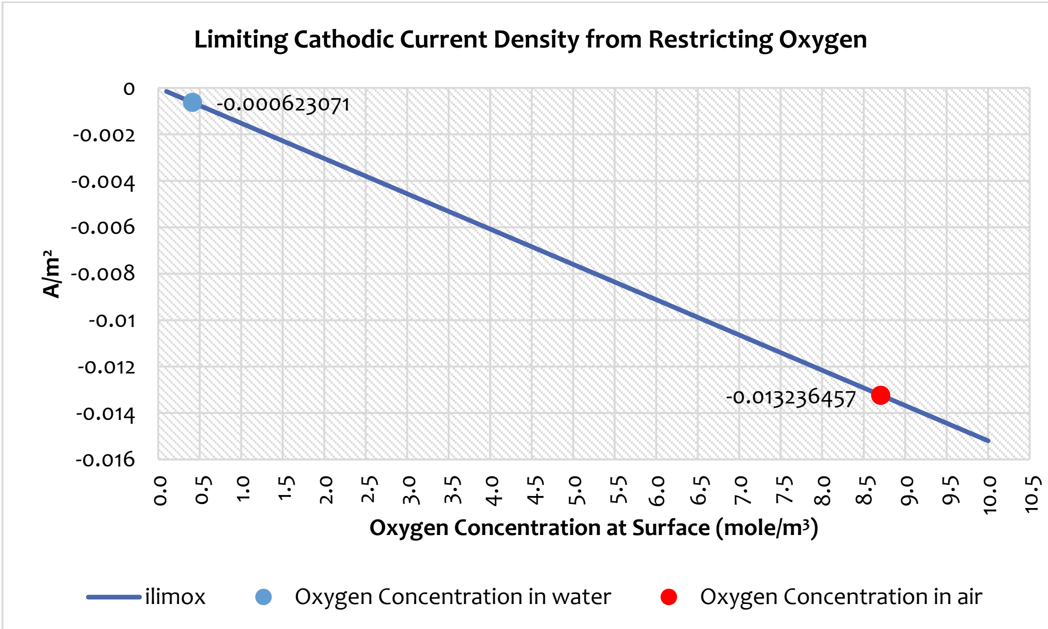


FIGURE 18 CATHODIC CURRENT DENSITY WITH OXYGEN AVAILABILITY IN CONCRETE

The figure above shows how large the reduction of current is when restricting oxygen availability in the corrosion reaction. The amount of current available when saturated is negligible and hence why reinforcing steel when saturated corrodes very slowly (0.72µm/yr^{-1}). At this corrosion rate in one hundred years you would have less than 0.1 mm section loss.

The time from steel corrosion initiation to cover cracking is mainly dependent on the corrosion rate, cover thickness, spacing between steel reinforcement, diameter of the reinforcement and the properties of the concrete. When the spacing is large enough, the cover concrete will expand, crack and spall along the longitudinal reinforcement. While the cover thickness, or the ratio of cover thickness to diameter c/d, is dominant, the delamination of cover concrete will develop at the surface layer of the steel.

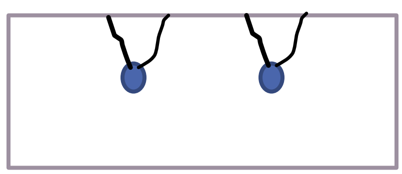


FIGURE 19 LONGITUDINAL CRACKING

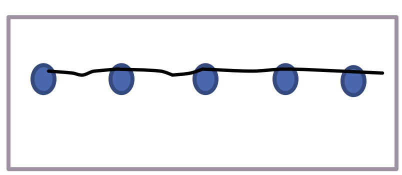


FIGURE 20 REINFORCING STEEL SURFACE LAYER CRACKING

In both of the above two cases, it is certain that the corrosion cracking will reach the exterior of thinnest concrete cover before spalling and delamination occur. When calculating for corrosion induced cracking two methods are typically adopted as follows:

1. The amount of corrosion products which can result in entire cracking of cover without taking account of the ingress of corrosion products into corrosion cracks.
2. The amount of corrosion products which accumulates in the open radial cracks during the progress of the crack front.

In conclusion it can be determined that the corrosion risk is of paramount importance to the long-term durability of the overall structure and currently is isolated to a number of previously repaired areas. We can also conclude that currently corrosion is not the underlying issue on the structure based on our test results.

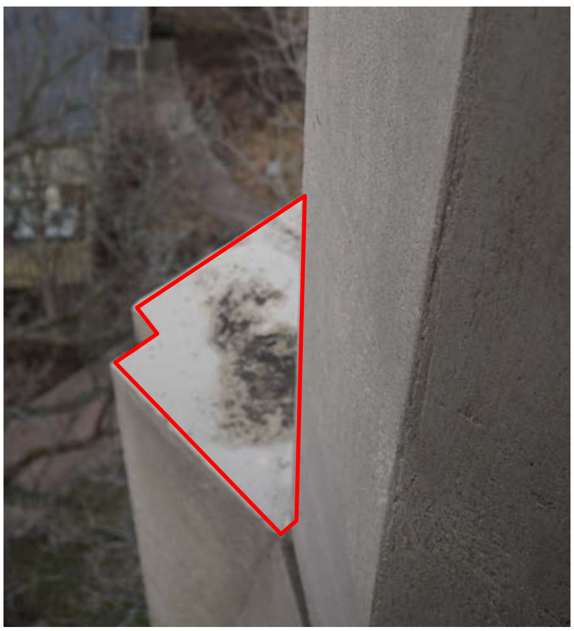


FIGURE 21 STANDING WATER ON TOP OF A PRE-CAST PANEL [NO PRESENCE OF ANY FLASHING]

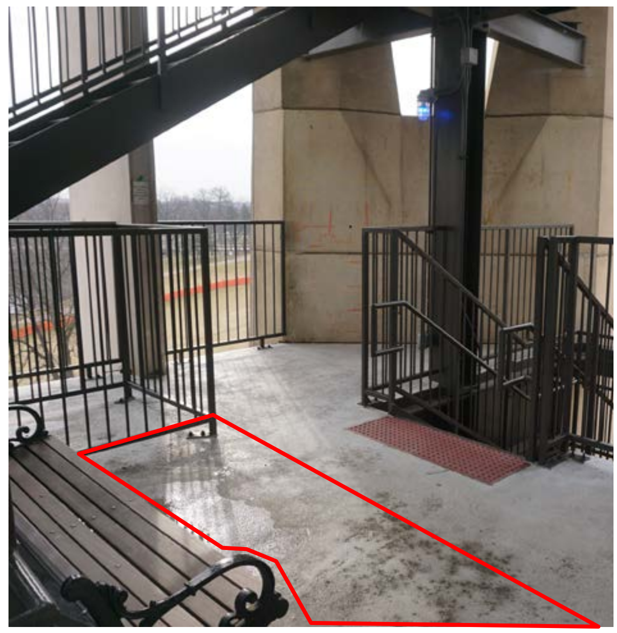


FIGURE 22 STANDING WATER ON LEVEL +52.5.

In summary, there is very low corrosion activity recorded at the Moser Tower on both pre-cast main panels and on the reinforced concrete components. In addition, negligible level of chlorides and carbonation profiles were identified.

The absence of waterproofing membrane and flashing details will potentially aggravate the corrosion condition by allowing water ingress into the structural elements as the tower is open to the environment. The "stepping" configuration of the pre-cast panels displayed on all four elevations allows for water and snow accumulation as shown in Figure 21 and Figure 24. In addition, the connection detail from each compressive ring to the pre-cast panels are not protected as shown in Figure 23. In conclusion, a water-management plan should be implemented as part of the rehabilitation project, however caution should be noted in reducing moisture levels at certain locations as this will increase oxygen and could exasperate corrosion if not fully understood.

FIGURE 23 DETAIL OF CONNECTION BETWEEN COMPRESSIVE RING AND THE PRE-CAST PANEL

FIGURE 24 STEPPING CONFIGURATION OF THE PRE-CAST PANEL ON THE ELEVATION

3.2. REINFORCEMENT DISTRIBUTION

The reinforcement distribution determines the steel configuration of the reinforced concrete cast-in place component [concrete beam] and the pre-cast elements [Fin and Main Panel] at selected heights.

Echem's Life 52™ i-med service for material engineering detection was performed by utilizing a Surface Penetrating Radar (SPR) by using a GSSI StructureScan™ Mini HR

> *The GSSI StructureScan™ Mini HR has the ability to detect reinforcing steel up to sixteen (16) inches based on the manufacturer datasheet. In the field it is often found that when high moisture and chlorides are present in the structure being surveyed, it is very difficult to delineate construction elements beyond the first six (6) inches.*

3.2.1. PRECAST POST TENSIONED PANEL

Precast post-tensioned panels made up the elevations of the tower located at four corners as shown in Figure 25. The post-tensioning rod was identified through a probe opening that was completed by Golf Construction during the Field Work as shown in Figure 28 and Figure 29. The SPR was not able to clearly detect the location of the post-tension rod due to the presence of the reinforcement in front.

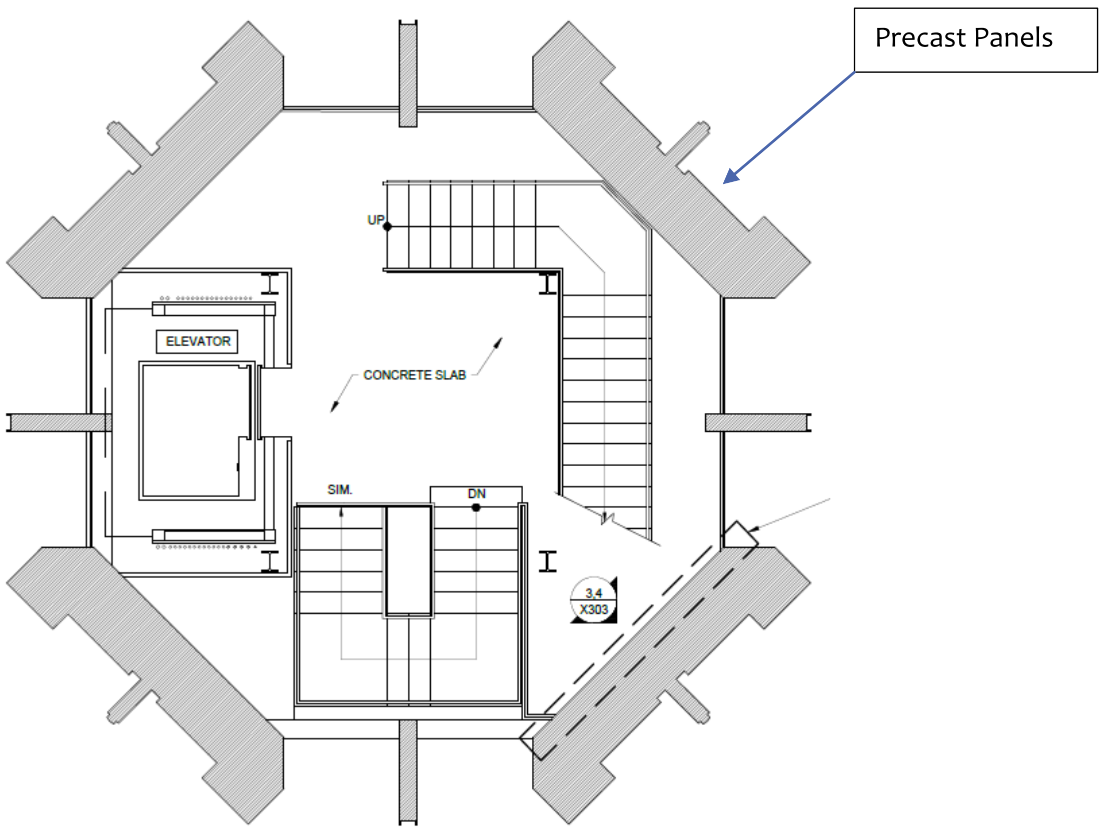


FIGURE 25 TYPICAL PLAN OF THE TOWER SHOWING THE MAIN PRE-CAST PANELS

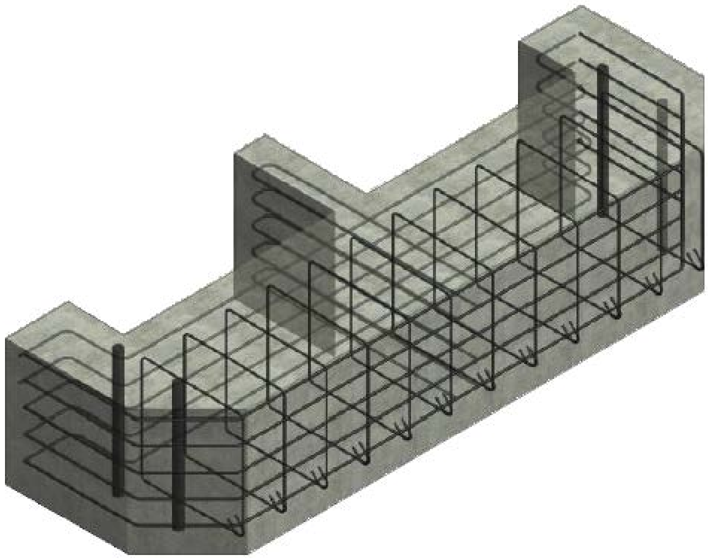


FIGURE 26 PRECAST MAIN PANEL 3D DIAGRAM REINFORCEMENT LAYOUT

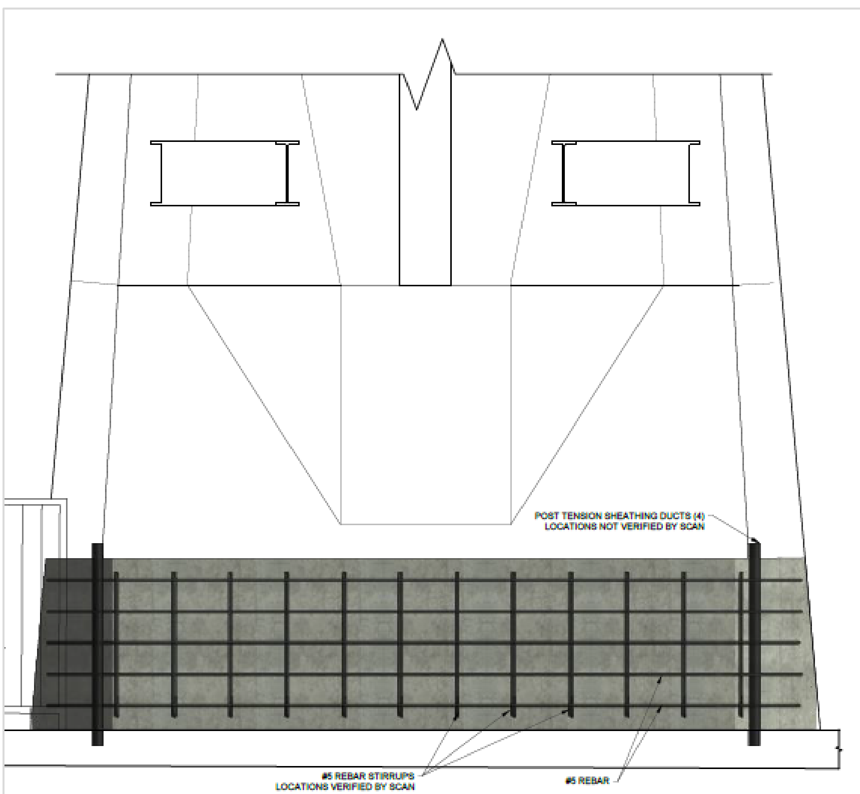


FIGURE 27 PRE-CAST MAIN PANEL ELEVATION

The reinforcement spacing detected for the precast panel is as follows:

Test Location	Drawing Reference	Average Depth of cover (inches)	Reinforcement Layout	
			Horizontal Layer Spacing (inches)	Vertical Layer Spacing (inches)
Main Precast panel	X303	4.13	12	12

TABLE 42 REINFORCING STEEL DISTRIBUTION PRECAST PANEL

3.2.1.1. PROBE OPENING

FIGURE 28 PROBE OPENING ON THE NORTH ELEVATION

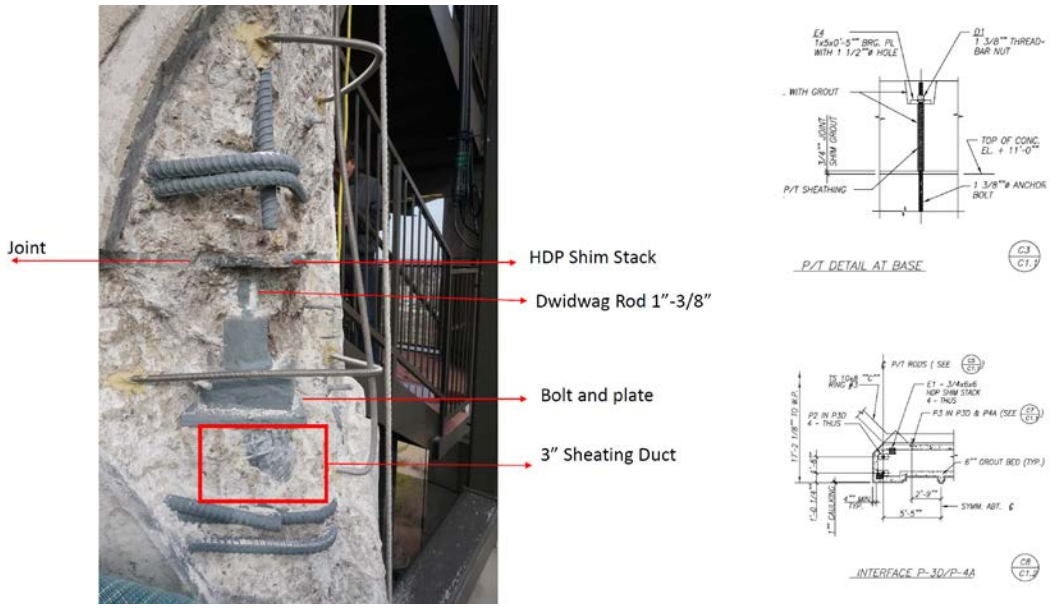


FIGURE 29 FOUND CONDITION AT PROBE OPENING OF POST-TENSIONED ROD DETAIL

3.2.2. PRECAST FINS

Precast Fins are decorative elements located at each elevations of the tower as shown in Figure 30. Each panel extends from above the stone wall to the top of the tower at four [4] locations.

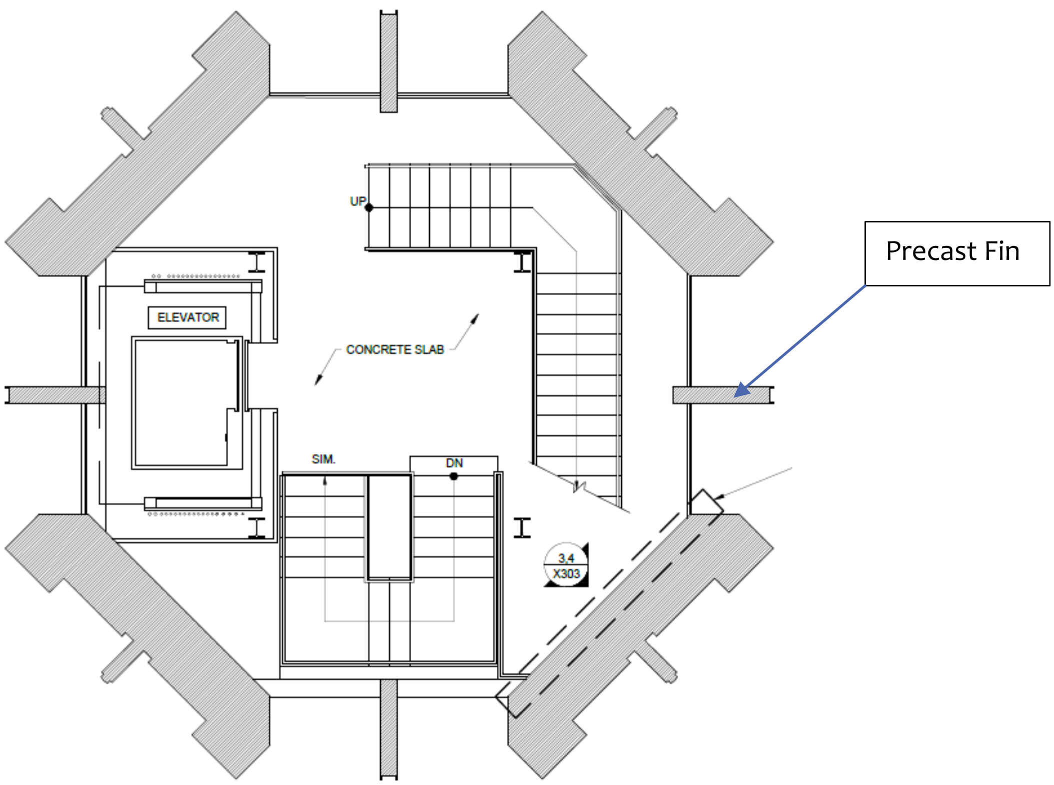


FIGURE 30 TYPICAL PLAN OF THE TOWER SHOWING THE PRECAST FIN LOCATION

FIGURE 31 PRE-CAST MAIN PANEL 3D DIAGRAM REINFORCEMENT LAYOUT

The reinforcement spacing detected for the fin pre-cast panel is as follows:

Test Location	Drawing Reference	Average Depth of cover (inches)	Reinforcement Layout	
			Horizontal Layer Spacing (inches)	Vertical Layer Spacing (inches)
Fin Pre-Cast panel	X302	4"	12"	12"

TABLE 43 REINFORCING STEEL DISTRIBUTION FIN PRE-CAST PANEL

3.2.3. CAST-IN PLACE COMPONENT [REINFORCED CONCRETE BEAM]

The reinforcement layout at the concrete beams located at the foundations and entry level of the tower was determined as follows:

Test Location	Drawing Reference	Average Depth of cover (inches)	Reinforcement Layout	
			Horizontal Layer Spacing (inches)	Vertical Layer Spacing (inches)
South Beam Reinforced Concrete	X301	1.60"	16"	4"

TABLE 44 REINFORCING STEEL DISTRIBUTION MAIN PRE-CAST PANEL

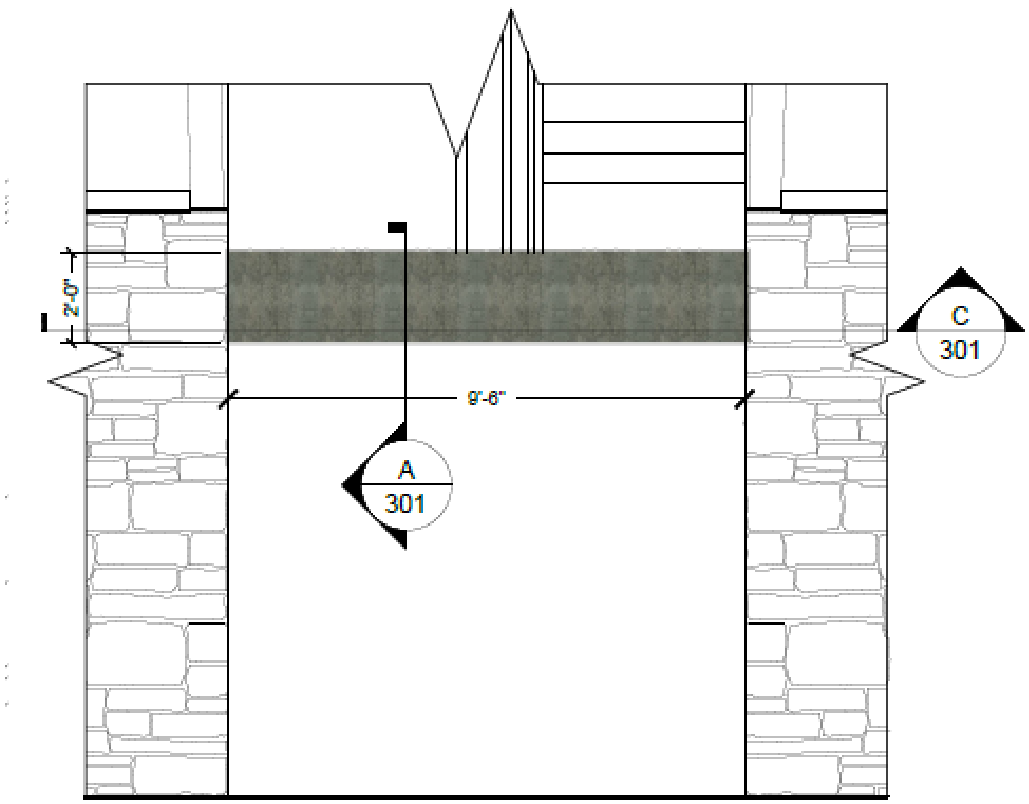


FIGURE 32 SOUTH ELEVATION SHOWING THE REINFORCED CONCRETE BEAM

FIGURE 33 REINFORCED CONCRETE BEAM 3D DIAGRAM REINFORCEMENT LAYOUT

3.3. REINFORCEMENT CONCRETE COVER

The cover distribution for the reinforcing steel is taken from the reinforcing distribution scans and can be summarized in the following tables.

Location	Minimum	Maximum	Mean	Standard Deviation	COV
Fin H	4.36	4.92	4.64	0.21	5%
Fin V	4.29	5.16	4.71	0.39	8%

TABLE 45 REINFORCING STEEL DISTRIBUTION PRECAST FIN

Location	Minimum	Maximum	Mean	Standard Deviation	COV
Beam H	1.50	2.27	1.89	0.39	20%
Beam V	1.16	1.44	1.35	0.09	7%

TABLE 46 REINFORCING STEEL DISTRIBUTION CAST IN PLACE BEAMS

Location	Minimum	Maximum	Mean	Standard Deviation	COV
Panel 1 H	3.57	3.98	3.73	0.18	5%
Panel 1 V	4.23	5.16	4.68	0.33	7%
Panel 2 H	1.82	1.82	1.82	0.00	0%
Panel 2 V	2.85	3.10	2.98	0.13	4%
Panel 3 H	4.29	5.10	4.69	0.33	7%
Panel 3 V	3.98	4.11	4.04	0.05	1%
Panel 4 H	3.42	3.42	3.42	0.00	0%
Panel 4 V	2.78	2.85	2.82	0.04	1%
Panel 5 H	3.73	3.73	3.73	0.00	0%
Panel 5 V	2.72	2.97	2.80	0.12	4%
Panel 6 H	4.48	5.35	4.92	0.32	7%
Panel 6 V	3.79	4.29	4.04	0.20	5%
Panel 7 H	6.03	7.08	6.48	0.40	6%
Panel 7 V	5.66	5.78	5.72	0.05	1%
Panel 8 H	3.92	4.17	4.04	0.09	2%
Panel 8 V	3.23	3.29	3.28	0.03	1%
Panel 9 H	3.10	4.04	3.67	0.40	11%
Panel 9 V	3.04	3.86	3.60	0.30	8%
Panel 10 H	5.47	7.39	6.67	0.69	10%
Panel 10 V	5.16	5.23	5.20	0.03	1%

TABLE 47 REINFORCING STEEL DISTRIBUTION MAIN PRE-CAST PANEL

When carrying out cover measurements typically insufficient measurements are taken to have real statistical evaluation. Due to this we carry out a monte carlo simulation of the data recorded to provide a more realistic distribution across the entire structure based on 10,000 samples.

Location	Minimum	Maximum	Mean	Standard Deviation
Beam	0	2.27	1.14	0.65
Fins	0	5.16	2.51	1.48
Panels	0	7.389	3.69	2.14

TABLE 48 REINFORCING STEEL DISTRIBUTION MONTE CARLO SIMULATION

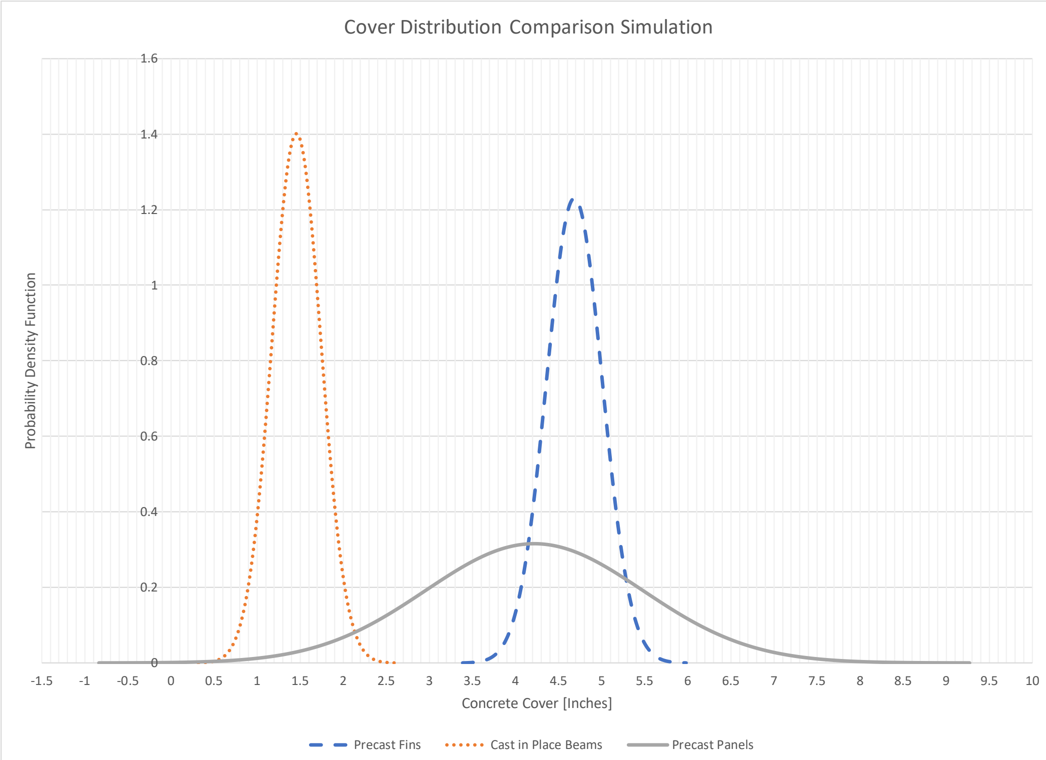


FIGURE 34 COVER DISTRIBUTION COMPARISON ACROSS COMPONENTS

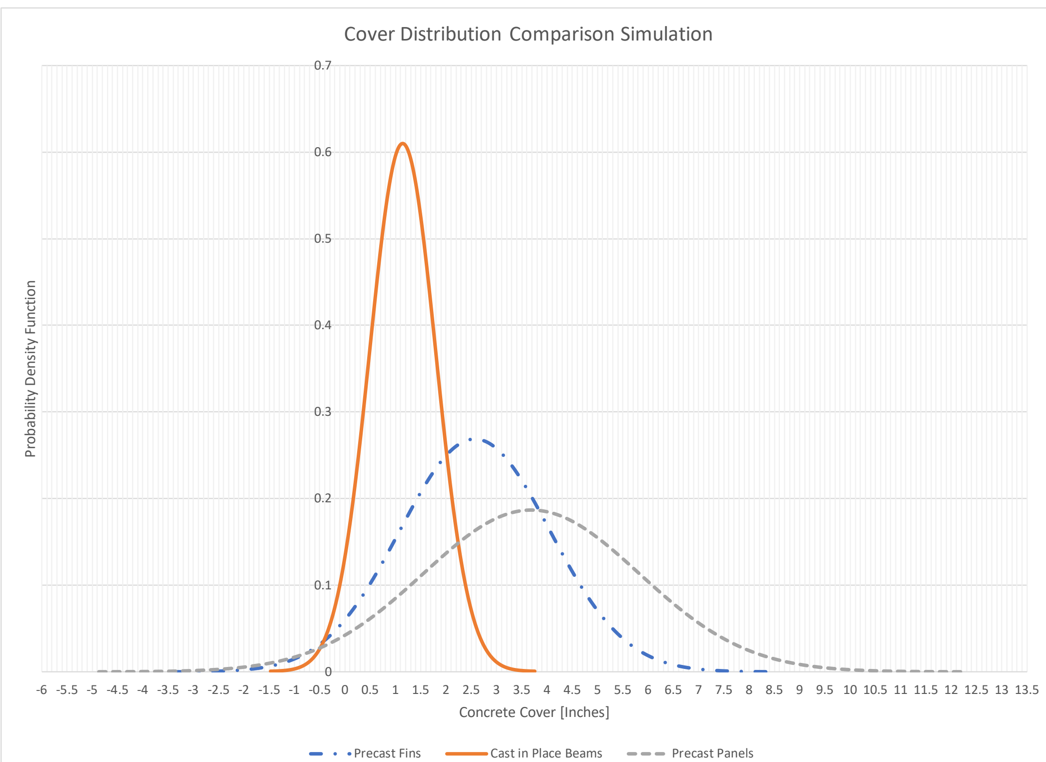


FIGURE 35 MONTE CARLO COVER DISTRIBUTION COMPARISON ACROSS COMPONENTS

SECTION 4 MATERIALS ENGINEERING (e-spec™)

Echem's e-spec™ incorporates materials engineering services across a broad spectrum. For this project our material durability Modelling for service life and durability were employed.

4.1. Material Durability Engineering (E-Dur)

Echem's material engineering encompasses the following three sub services: e-slm; e-mas; and e-dms. All three services address the structure and components in their given environment to include performance properties.

- Service life modeling of existing and new structures [e-slm].
- Material selection advice for new components in existing structures or for new structures. [e-mas]
- Durability models for new construction materials. [e-dms]

4.1.1. Service Life Modelling (E-Slm)

Service life modelling is developed as a process for modelling the structure from ongoing corrosion and degradation. Under this modelling process there is three (3) types of degradation models which are typically used as follows:

- Statistical Degradation Models

Statistical degradation models are based on physical and chemical laws of thermodynamics, and thus have a strong theoretical base. They include parameters, which have to be determined with specific laboratory or field tests. Therefore, some equipment and personnel requirements exist for the users. The application of statistical methods requires a need for a statistically sufficient number of tests. Statistical reliability methods can be directly applied with these models.

- Selected Calculation Models

Selected calculation models are based on parameters, which are available from the mix design of concrete. The asset of these models is the availability of the values from the documentation of the concrete mix design and of the structural design.

- Reference Structure Models

Reference structure model is based on statistical treatment of the degradation process and condition of real reference structures, which are in similar conditions and own similar durability properties with the actual objects. This method is suited in the case of a large network of objects, for example bridges. It is often combined with a Markovian Chain method in the classification and statistical control of the condition of structures.

For this project, we use statistical degradation models for carbonation diffusion chloride migration with reliability analysis. In addition, comparing the stress versus resistance of both contributing items and calculating the safety margin [M] for future predictions.

4.2. Statistical Degradation Models

The e-slm statistical degradation models (SDMs) include mathematical modelling of corrosion initiation due to carbonation, chloride ingress, corrosion propagation, frost (internal damage and surface scaling) and alkali-aggregate reaction.

We have carried out calculations related to carbonation, chloride and corrosion initiation/propagation which is based around data collected in the field. This report includes models which are presented on a semi-probabilistic level which include parameters obtainable through the investigations, without making use of default material and environmental data. Full-probabilistic models are applicable for service life design purposes and for existing objects, including the effect of environmental parameters but have not been used within this report.

In the present case depassivation of the reinforcement leading to corrosion propagation has been chosen as one method of reviewing the limit state. As the depassivation itself does not lead to severe consequences (structural failure) the limit state can be allocated as a serviceability limit state (SLS). Within a limit state the variable describing the resistance is being confronted with the variable describing the load, hereby also considering the variability of these variables.

By considering the depassivation of the reinforcement due to carbonation/chlorides, the concrete cover is defined as the resistance and the carbonation depth or chloride ingress is the stress. As the carbonation depth and/or chlorides increase with time, the stress variable has to be defined as time dependent.

The respective limit state equation describing the probability that depassivation takes place is given in Equation 1.

EQUATION 1 $$-p\{failure\} = p_f = p\{d_c - x_c(T) < 0\}$$

P_f: Failure Probability

dc: Concrete Cover (mm)

xc: carbonation/chloride Depth at time (T) (mm)

The service life of a component follows from the comparison of the minimum target reliability (β0) and the reliability over exposure time (t), as depicted in Figure 36 below.

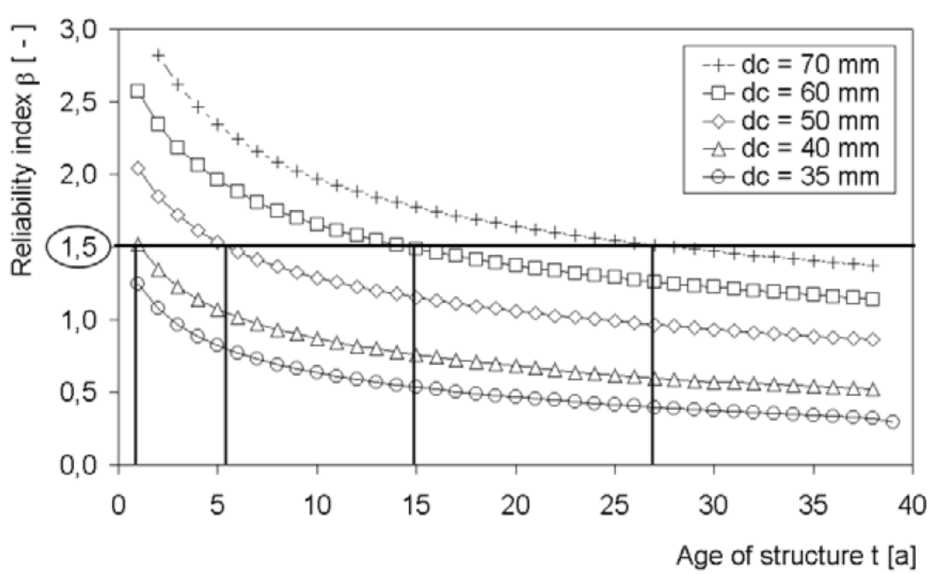


FIGURE 36 RELIABILITY INDEX β VERSUS EXPOSURE TIME T. EXAMPLE OF CHLORIDES FOR VARIOUS COVER DEPTHS (dc)

The technical service life 'T' depends strongly on the target reliability level (β_0.) The question which target reliability level is appropriate must thus be answered. In general, the following must be considered when assigning target reliabilities to possible limit states:

- possibility to detect damage
- possibility of corrective actions in case of failure
- consequence of failure

Please note, that failure is here used in the sense of exceeding a predefined limit state and not in the sense of structural collapse.

Serviceability Limit States:

With respect to reinforcement corrosion the following states are regarded as service ability limit states:

SLS 1: depassivation of reinforcement

SLS 2: crack formation

SLS 3: spalling of concrete, if no risk emerges from falling pieces (otherwise this is regarded an ultimate limit state)

The Life 52™ standardization sets up a fixed reliability index of ($\beta_{SLS,T}$ = 1.50) regardless of the type of serviceability limit state. Since passing a serviceability limit state is always accompanied by economic costs. Economic optimization of the target reliability should be performed by superposition of production costs (curve A) and maintenance costs (curve B) which are both connected to the reliability level, the optimal reliability level can be calculated from an economical point of view (economic optimization) as shown in the figure below:

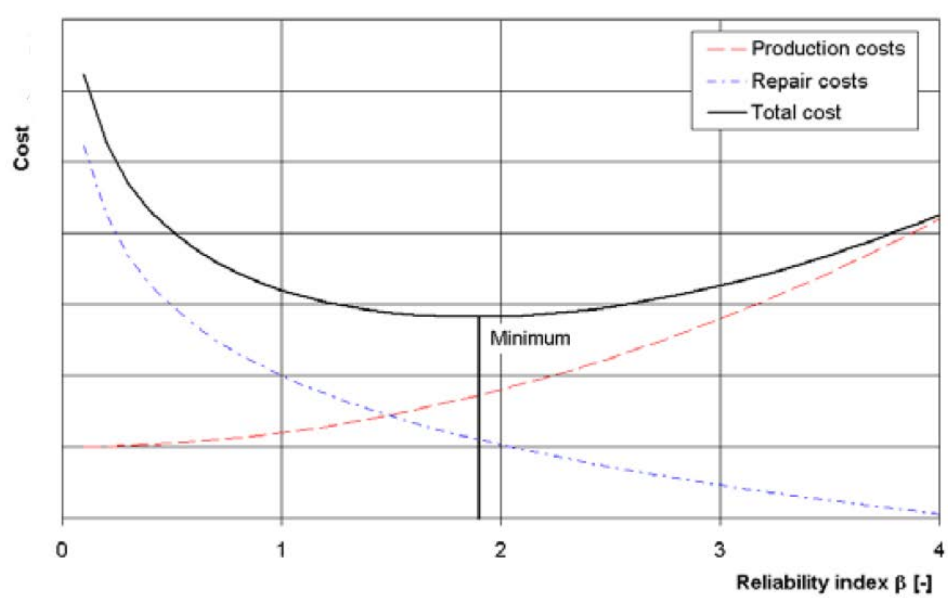


FIGURE 37 DETERMINATION OF THE OPTIMAL RELIABILITY LEVEL

4.2.1. CARBONATION MODELS

The performance of the structure for Service Life Reliability has been generated for the effects of carbonation at the reinforcing steel as seen in the following figures for the cast in place and the precast concrete. These models illustrate the performance of the structure in relation to exposure, based on a Portland cement concrete mix designs, 4500 psi compressive strength concrete and measured carbonation depth of less than 1mm at eighteen years of exposure.

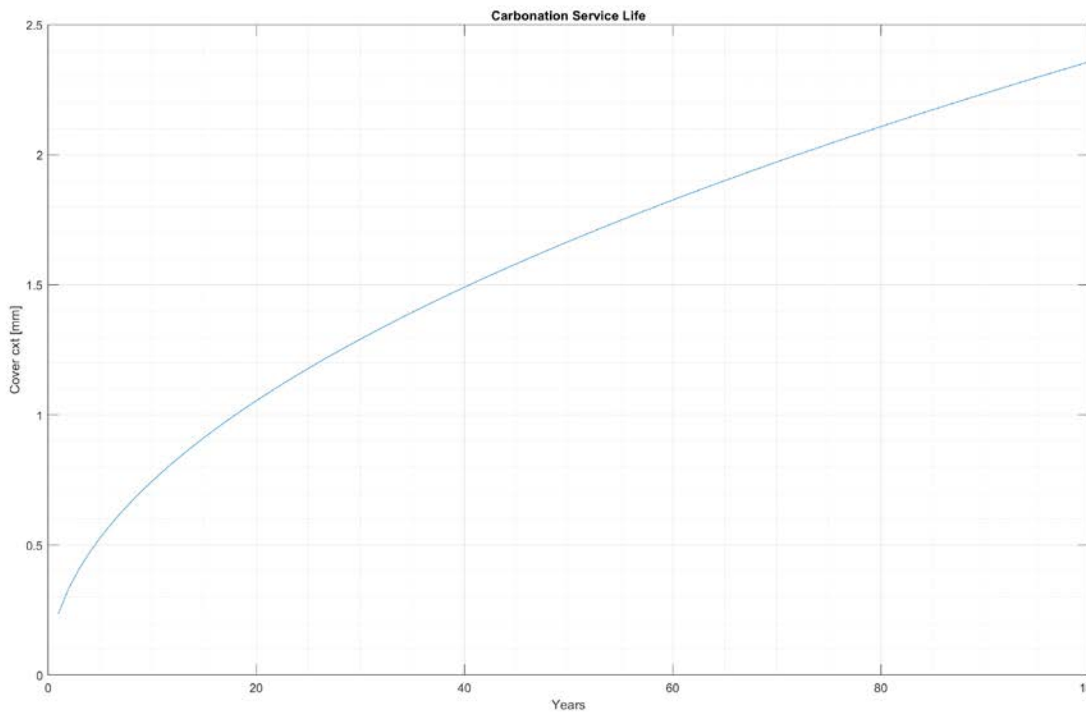


FIGURE 38 SERVICE LIFE CARBONATION PENETRATION

The above model shows that only a minimal carbonation front in the concrete is expected in the next 100 years, primarily due to concrete quality and moisture levels that currently exist. It should be noted that this is not uncommon on this type of exposed structure, however this model becomes invalid when cracks exist where carbon dioxide can migrate into the concrete effortlessly to the depth of the reinforcing steel.

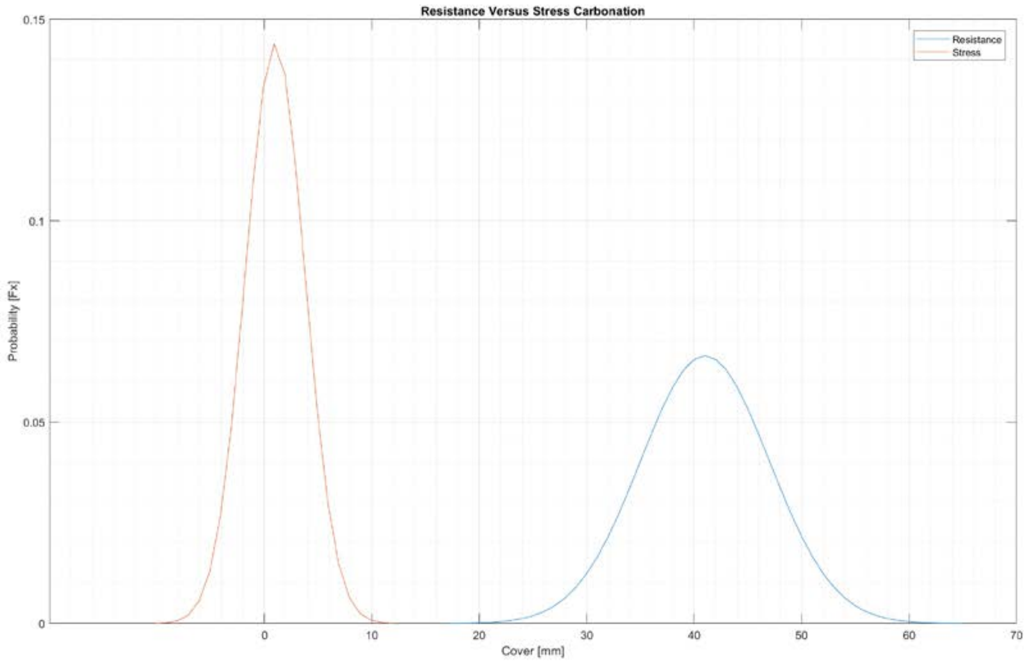


FIGURE 39 SERVICE LIFE RELIABILITY [B] CARBONATION RESISTANCE V STRESS CAST IN PLACE BEAMS

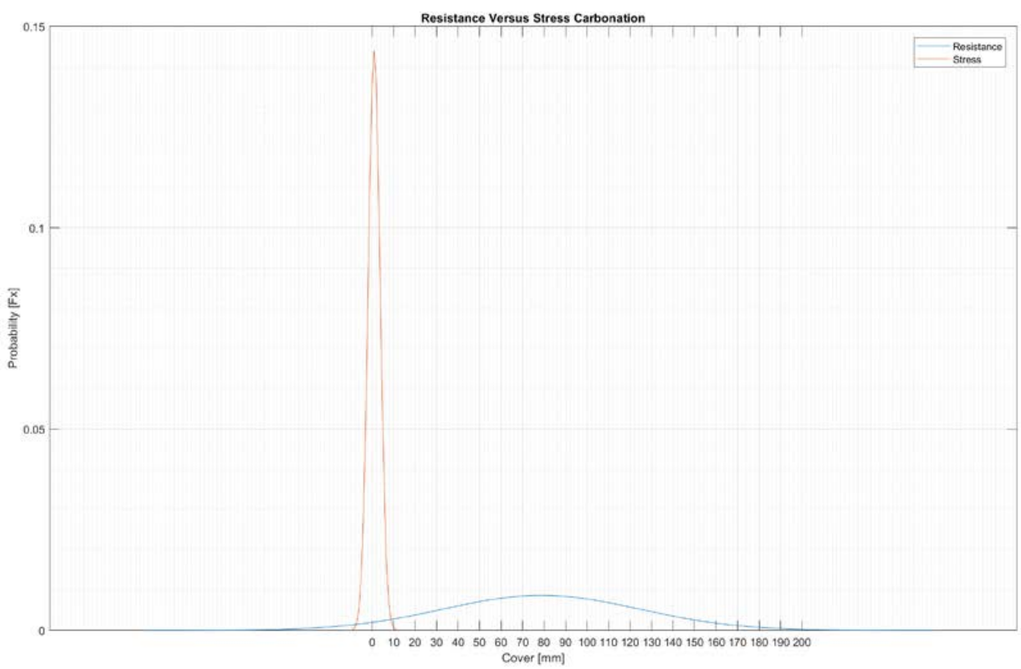


FIGURE 40 SERVICE LIFE RELIABILITY [B] CARBONATION RESISTANCE V STRESS PRECAST

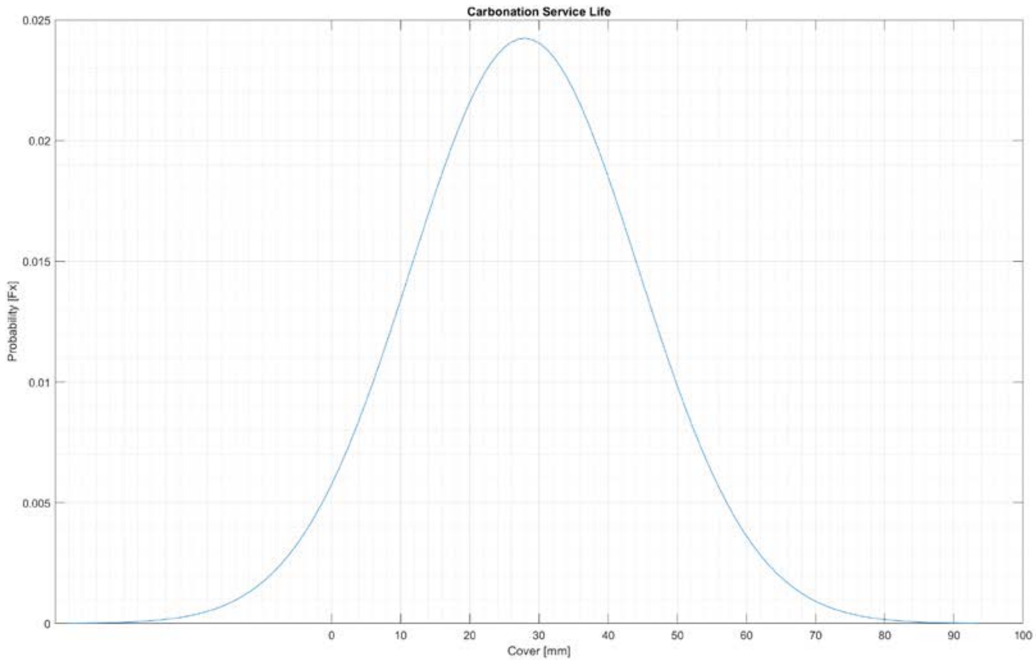


FIGURE 41 SERVICE LIFE SAFETY MARGIN [M] CARBONATION CAST IN PLACE

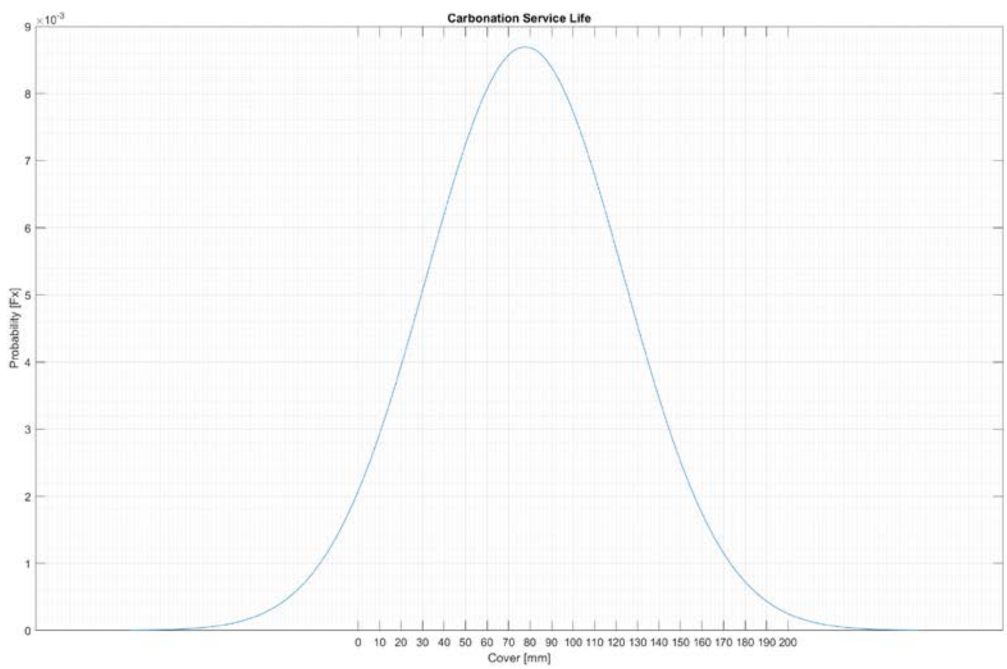


FIGURE 42 SERVICE LIFE SAFETY MARGIN [M] CARBONATION CAST IN PLACE

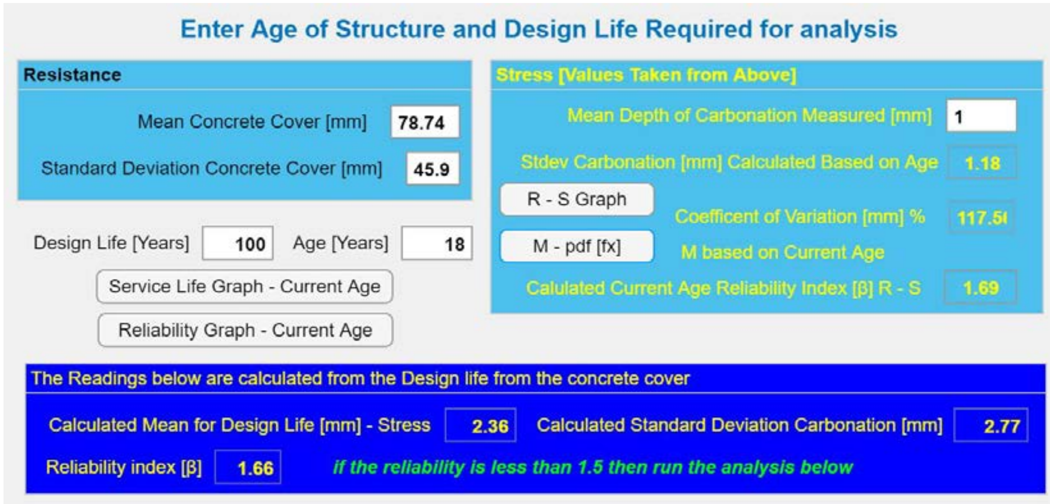


FIGURE 43 SERVICE LIFE RELIABILITY [β] PREDICTION FOR 100 YEAR DESIGN LIFE CARBONATION PRECAST

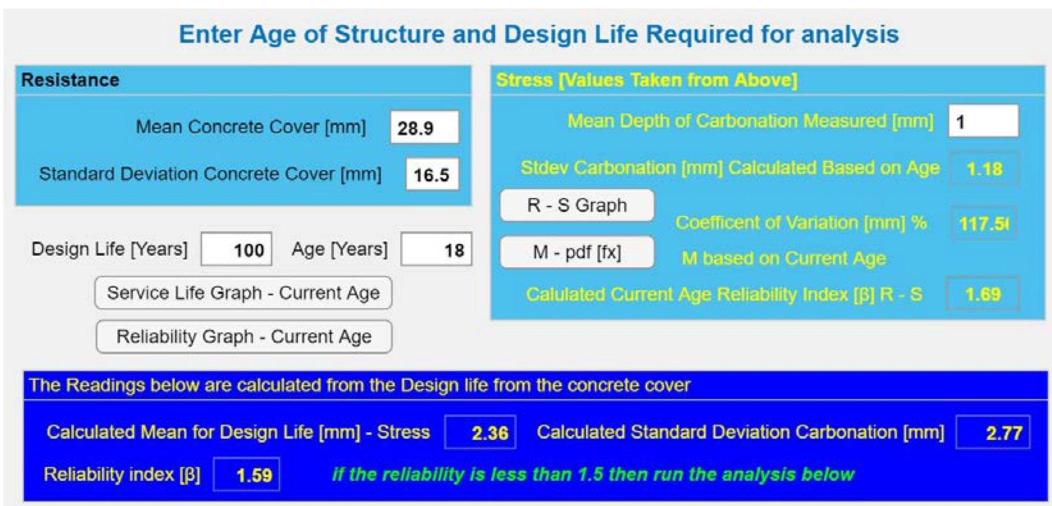


FIGURE 44 SERVICE LIFE RELIABILITY [β] PREDICTION FOR 100 YEAR DESIGN LIFE CARBONATION CAST IN PLACE

The above models clearly show slow migration of carbonation over time and shows that ultimately the structure will not be compromised with carbonation in 100 Years based on a reliability index of 1.56 for the cast in place and 1.66 for the precast.

When reviewing the carbonation resistance versus stress chart the safety margins calculated are extremely low.

4.2.2. CHLORIDE MODELS

The performance of the structure for Service Life Reliability has been generated for the effects of chloride at the reinforcing steel as seen in the following figures for the cast in place and the precast concrete. These models illustrate the performance of the structure in relation to exposure, based on a Portland cement concrete mix designs, 4500 psi compressive strength concrete, measured chloride depth at listed in Table 8 and eighteen years of exposure. The following parameters in Table 49 and Table 50 were used within the models for the respective concrete type.

Project Title	Moser Tower - Precast Concrete			
Number of Depths	2			
Number of Tests	4			
Concrete Cover [inches]	3.1	mean	1.81	Standard Deviation
Margin of Error	50.00%	Calculated Field		
Margin of Error [Required]	1.00%	10000	Number of Samples Required	
Chloride threshold	0.2	mean	0.05	Standard Deviation
Year Built	2000	Age	18	
Diffusion Coefficient	8E-07	Concrete Diffusion Coefficient cm^2/s		
cls_surface	0.3	Surface Chloride by weight of concrete		

TABLE 49 CHLORIDE SERVICE LIFE PARAMETERS FOR PRECAST CONCRETE

Project Title	Moser Tower – Cast In Place Concrete			
Number of Depths	2			
Number of Tests	1			
Concrete Cover [inches]	1.14	mean	0.65	Standard Deviation
Margin of Error	100.00%	Calculated Field		
Margin of Error [Required]	1.00%	10000	Number of Samples Required	
Chloride threshold	0.2	mean	0.05	Standard Deviation
Year Built	2000	Age	18	
Diffusion Coefficient	2E-07	Concrete Diffusion Coefficient cm^2/s		
cls_surface	0.3	Surface Chloride by weight of concrete		

TABLE 50 CHLORIDE SERVICE LIFE PARAMETERS FOR CAST IN PLACE CONCRETE

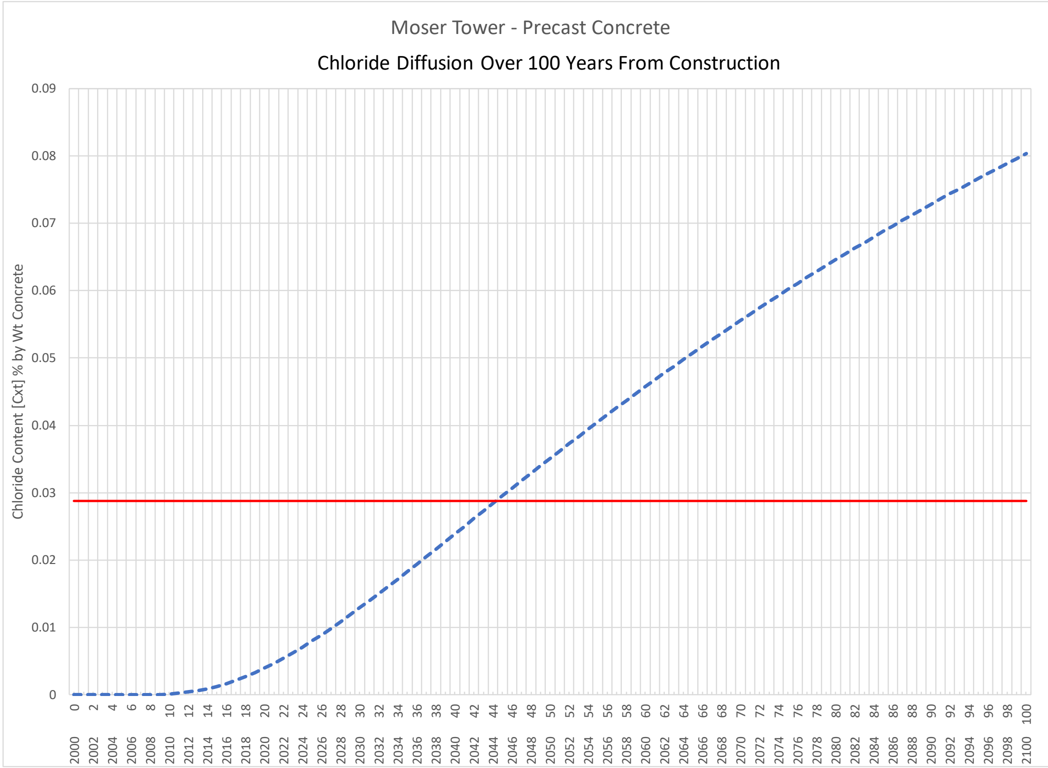


FIGURE 45 SERVICE LIFE CHLORIDE DIFFUSION PRECAST CONCRETE

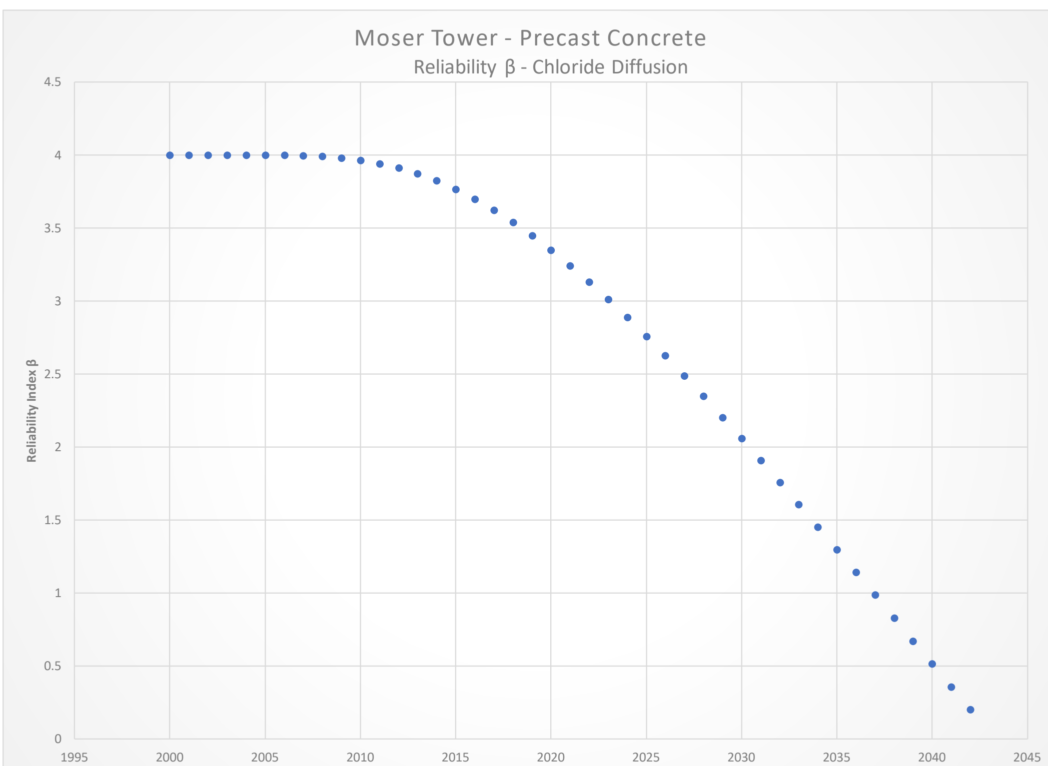


FIGURE 46 SERVICE LIFE RELIABILITY [β] PREDICTION FOR 100 YEAR DESIGN LIFE CHLORIDE DIFFUSION PRECAST CONCRETE

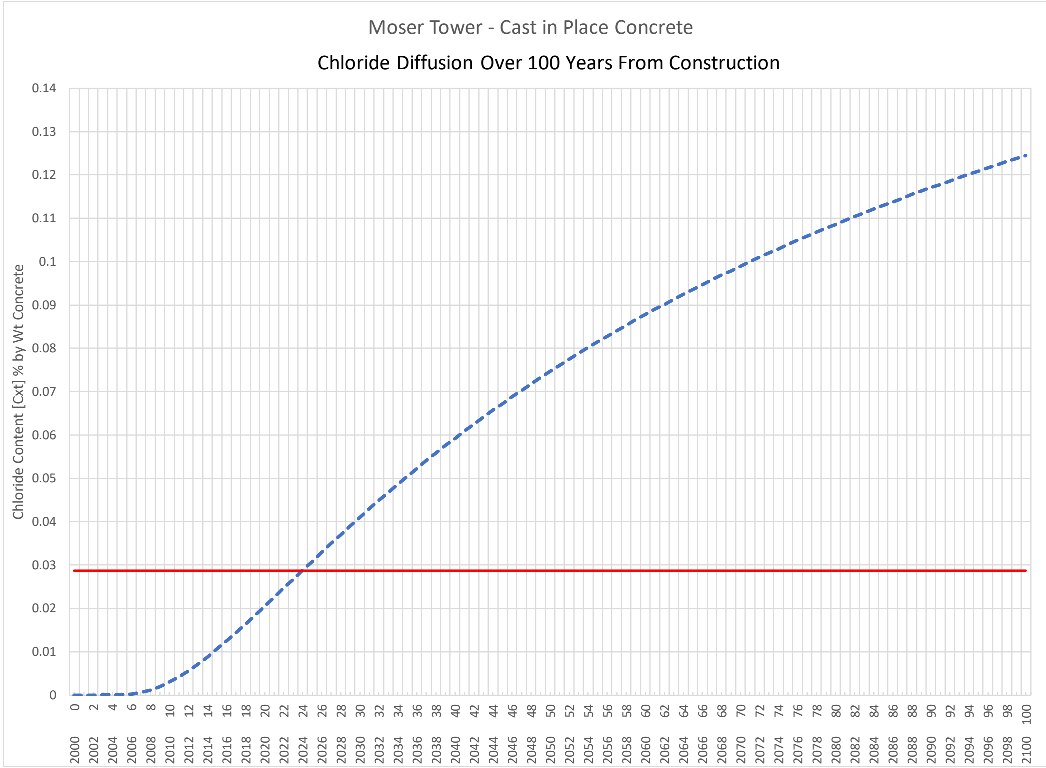


FIGURE 47 SERVICE LIFE CHLORIDE DIFFUSION CAST IN PLACE CONCRETE

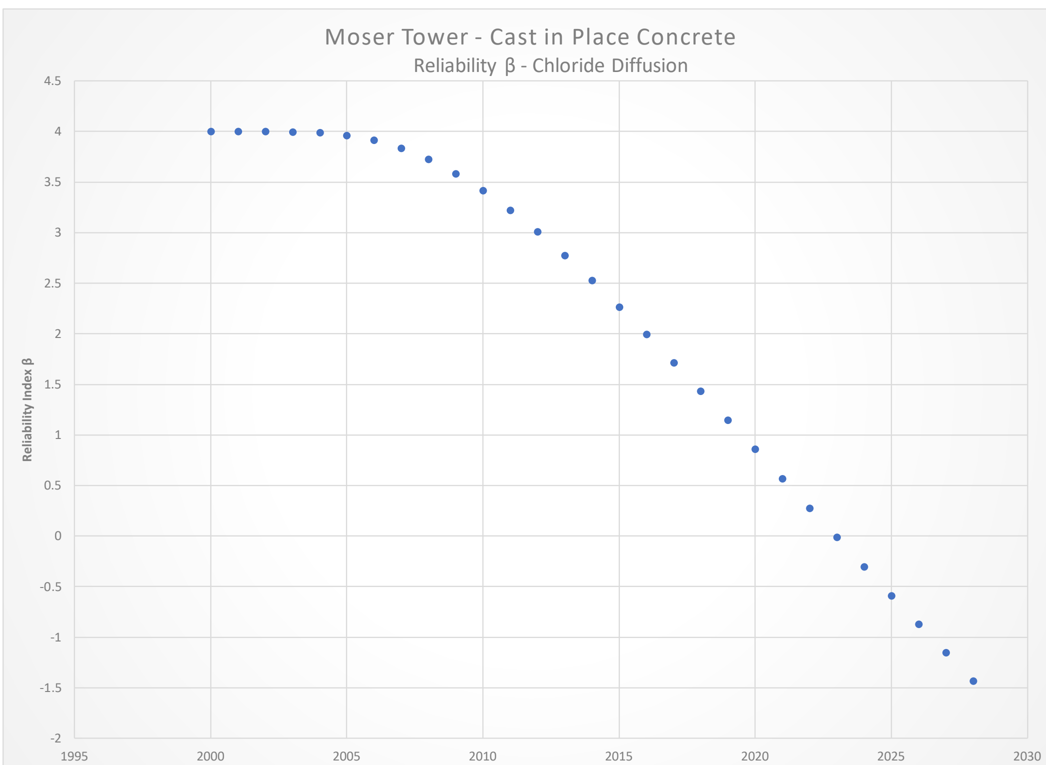


FIGURE 48 SERVICE LIFE RELIABILITY [β] PREDICTION FOR 100 YEAR DESIGN LIFE CHLORIDE DIFFUSION CAST IN PLACE CONCRETE

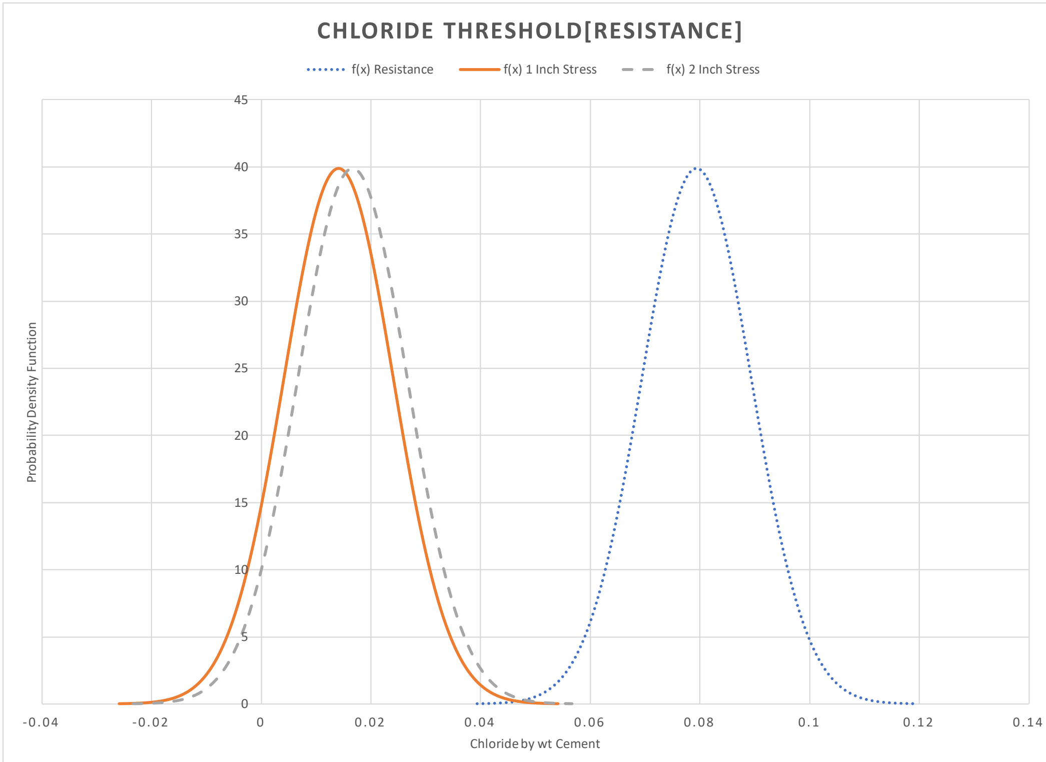


FIGURE 49 RESISTANCE VERSUS STRESS SOUTH BEAM CHLORIDES

Qcr	34.57	**Service Life Calculation ACI 365**
q1	0.26579432	
q2	2.507184893	
q3	1.089405389	
q	0.611705717	
t_i	56.52	Years

TABLE 51 ACI 365SERVICE LIFE CALCULATION FOR CAST IN PLACE CONCRETE

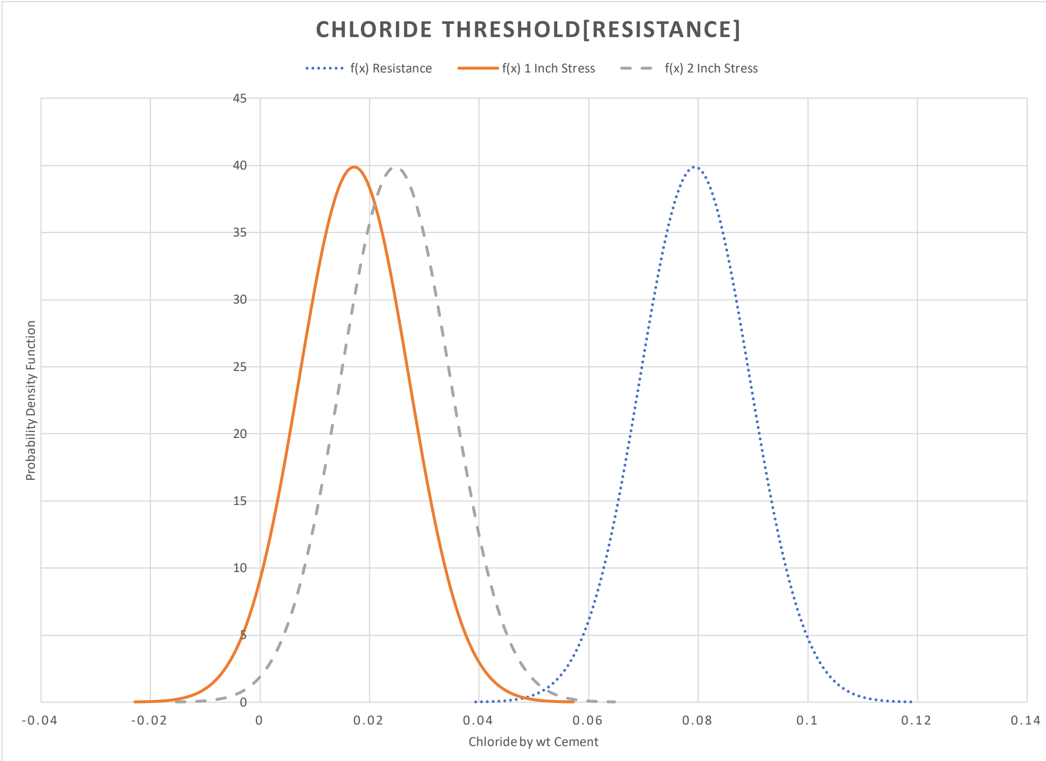


FIGURE 50 RESISTANCE VERSUS STRESS PRECAST PANEL CHLORIDES

Qcr	81.83	**Service Life Calculation ACI 365**
q1	0.02841027	
q2	2.867508195	
q3	1.345719348	
q	0.060537646	
t_i	>100	Years

TABLE 52 ACI 365SERVICE LIFE CALCULATION FOR PRECAST PANEL CONCRETE

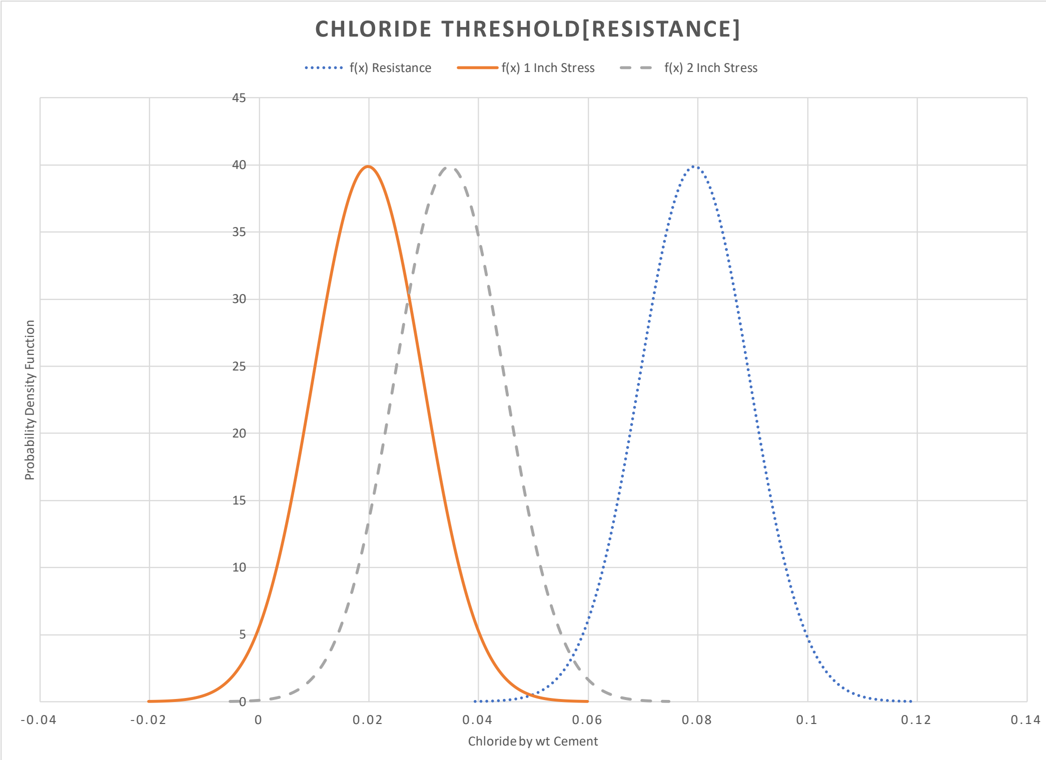


FIGURE 51 RESISTANCE VERSUS STRESS PRECAST FINS CHLORIDES

Qcr	60.81	**Service Life Calculation ACI 365**
q1	0.06961803	
q2	3.317912323	
q3	1.666111797	
q	0.138638067	
t_i	>100	Years

TABLE 53 ACI 365SERVICE LIFE CALCULATION FOR PRECAST FIN CONCRETE

The above models clearly show the affects on low cover concrete seen in the cast in place beams as the theoretical ACI model shows a time period of 56 Years compared to that of the precast of over 100 years. These models are built on environmental condition and concrete properties.

Life 52® reliability models show the cast in place concrete chloride levels has already exceeded the SLS of 1.5 requiring attention. The same model for the precast shows this level being reached in 2033 fifteen [15 years from now.

The safety margin probabilities [M] for the precast has a reliability index $\beta = 3.43$ at the depth of the reinforcing steel. The cast in place has a reliability index $\beta = 2.41$ at the depth of the reinforcing steel.

Both of these fall under the limit of 1.5 showing minimal distress from chlorides at this time.

SECTION 5 RECOMMENDATIONS

Echem Recommendations			
Type of Service	**Items**	**Description**	**Ref**
In-situ Inspection(i-spec)	Exploratory Probe at Pre-cast main panel	Exploratory coring is recommended at the post-tensioned rod located within the pre-cast main panels. This will allow to determine the grout composition and assess the type of the grout material used.	1.1
	Monitoring Loggers FOR Internal Temperature and Humidity Condition	Install embedded atmospheric loggers to monitor the internal condition of the concrete at different locations of the structure to track temperature and humidity within the pre-cast elements and cast-in place components given the level of saturation found. At least six (6) locations should be monitored for a longer period.	1.2
Engineering (e-spec)	Re-design flashing Details	As part of the rehabilitation project, design and install flashing on the external pre-cast panels where needed to avoid water ingress.	1.3
	Moisture Management Plan	Implement a moisture management plan to limit water ingress. Regular maintenance should be carried out including painting and cleaning of the drain systems.	1.4
	Pre-cast Panel Repair	All new repairs, we would recommend specifying suitable concrete repair material that is compatible with the original material. A (National Ready mix Concrete Association) NRMCA Level IV concrete material specialist should be consulted.	1.5

GENERAL NOTES

For future repair to any of the pre-cast and cast-in place components, a NRMCA Level IV Concrete/Material Specialist should be engaged. The specialist shall provide material specification and details on how to carry out the repair. Compatible concrete mix material and reinforcement should be selected to provide durable and quality repair which will contribute to extend the service life of this structure.

Using non-compatible material has the potential to accelerate damage to this structure reducing its service life. During the field work, Echem observed the probe opening carried out on the north elevation. Stainless steel rods were added as anchorage to the area to be repaired. Using Stainless steel in combination with black rebar will potentially create a galvanic cell resulting in galvanic corrosion where the black bar will corrode to protect the stainless-steel element [more noble].

SECTION 6 CONCLUSIONS

A limited in-depth testing program was performed at selective locations which included a corrosion survey, material testing and construction arrangement program.

The work carried out identified several findings that require further discussion, to enable a repair work scope to be created. Our inspection found that globally, the concrete is not being impacted by corrosion activity. The reinforcing steel was corroding at negligible rates at the areas tested. This is primarily due to high moisture content, lack of chlorides at the depth of the steel and zero carbonation.

The chloride diffusion models showed that the migration of chlorides will be a concern on the cast in place concrete before the precast due to concrete cover. This time period is expected to be within ten years and then the structure will start to corrode at an exponential rate. We are not sure where the chlorides are coming from, however this should not be overlooked. It should be noted that corrosion rates in totally exposed structures [concrete exposed to the elements on all sides] are highly affected by the availability of oxygen at the steel surface due to higher moisture content when this is a sizable component.

The precast elements also contained chlorides and we would recommend exploring more as we suspect these chlorides came from initial construction as part of curing. Calcium chloride is used as an accelerator in the concrete for curing and although most of the chlorides become bound during curing, with time these chlorides can become mobile and migrate toward the steel and causing corrosion.

We would conclude that the existing damage is primarily due to poor detailing and incorrect repairs, both in materials and design. This has led to local defects caused by the building performing in a way it was designed, where the concrete surface cracking has been caused by structural performance and not materials deficiencies. Where these repairs have been carried out they have continued to perform poorly based on material selection and design. A structural assessment and finite element model looking at stresses should be considered at these locations. It is most likely these areas are failing more in design than material failure.

In the following subsection we have put together a corrosion risk matrix where it can be seen that the precast concrete is more at risk than the cast in place even though we expect the cast in place to be more vulnerable first.

The overall survey indicated that the precast is at Condition State 3 and the cast in place is at Condition State 4. These ratings would indicate that the structures' materials are performing well and with some minor maintenance the performance in the future can be significantly improved.

It is essential to implement a moisture management plan with regular maintenance to avoid on-going corrosion. The drain system should be regularly inspected and maintained to prevent any further issues and a surface treatment should be considered to avoid any further contamination.

6.1. CORROSION ASSESSMENT ACI 201 CHECKLIST

ACI 201 provides is a guide for conducting a visual inspection of concrete in service. The following table is the items covered within our inspection report.

	Corrosion Specific Test	ASTM Tests	Other Test	Durability Modelling - Life 52
Recommended OR Approved	◉ Half-Cell	◉ C856 Petrographic Cores	○ Ultrasonic Pulse	◉ Chloride Diffusion
	◉ Linear Polarization	◉ C457 Air Void Analysis	○ Rebound Hammer	◉ Carbonation
	◉ Resistivity	○ C1218 Water Soluble Chloride	○ Impulse Echo	○ Time to Cracking
	◉ Cover Survey	◉ C1152 Acid Soluble Chloride	○ Dynamic Response	○ Degradation
	◉ GPR Survey	○ C42 Compressive Strength	○ Thermography	◉ Service Life
		◉ C1202 Rapid Chloride		○ Depassivation Probability

FIGURE 52 ACI 201 CHECKLIST AND LIFE 52® MODELLING

When any of the above test are performed they are typically limited to selective areas of the structure and an overview is assessed on the long-term performance expected in the future.

This project restriction included accessibility and budget which allowed the team to select areas on site over a series of days. A small sample size has been documented however, it is felt sufficient information has been collected to make long term prediction on the performance of the materials.

6.2. CORROSION ASSESSMENT RISK MATRIX

		Cast in Place Beams					
CONCRETE	Risk Element		High	Low	Mean	Standard Deviation	Risk Factor Score
	Chloride		0.113 by wt cem	0.097 by wt cem	0.105 by wt cem	0.011 by wt cem	2
	Carbonation		0 mm	0 mm	0 mm	0 mm	1
	Water Cement Ratio				0.4		1
	Air %				3 - 5%		2
	Strength		Not Measured				3
	Permeability Resistance				1234 C		2
REINFORCING STEEL	Corrosion Activity [i-vos]		-38 mV w.r.t $Cu/CuSo_4$	126 mV w.r.t $Cu/CuSo_4$	33.8 mV w.r.t $Cu/CuSo_4$	31.51 mV w.r.t $Cu/CuSo_4$	1
	Corrosion Rates [i-cos]		4.41 um/yr^{-1}	0.1 um/yr^{-1}	0.5 um/yr^{-1}	0.9 um/yr^{-1}	2
	Concrete Cover [i-cov]		2.27”	1.16”	1.62”	0.24”	3
Moisture		Humidity	100 %	100 %	100 %	0	3
						Total	**20**

	Precast Fins					
CONCRETE	Risk Element	High	Low	Mean	Standard Deviation	Risk Factor Score
	Chloride	0.3 by wt cem	0.09 by wt cem	0.189 by wt cem	0.011 by wt cem	3
	Carbonation	0 mm	0 mm	0 mm	0 mm	1
	Water Cement Ratio			0.5		2
	Air %			2 - 3%		3
	Strength	Not Measured				3
	Permeability Resistance			Not Tested		3
REINFORCING STEEL	Corrosion Activity [i-vos]	-352 mV w.r.t $Cu/CuSo_4$	-17 mV w.r.t $Cu/CuSo_4$	-133 mV w.r.t $Cu/CuSo_4$	-72 mV w.r.t $Cu/CuSo_4$	3
	Corrosion Rates [i-cos]	10.1 um/yr^{-1}	0.1 um/yr^{-1}	0.96 um/yr^{-1}	1.79 um/yr^{-1}	3
	Concrete Cover [i-cov]	5.16"	4.29"	4.67"	0.3"	1
Moisture	Humidity			Not Tested		3
					Total	**25**

	Precast Panels					
CONCRETE	Risk Element	High	Low	Mean	Standard Deviation	Risk Factor Score
	Chloride	0.184 by wt cem	0.113 by wt cem	0.145 by wt cem	0.033 by wt cem	3
	Carbonation	0 mm	0 mm	0 mm	0 mm	1
	Water Cement Ratio			0.55		3
	Air %			5 - 7%		1
	Strength	Not Measured				3
	Permeability Resistance			1829 C		2
REINFORCING STEEL	Corrosion Activity [i-vos]	-232 mV w.r.t Cu/CuSo$_4$	-27 mV w.r.t Cu/CuSo$_4$	-156 mV w.r.t Cu/CuSo$_4$	-58 mV w.r.t Cu/CuSo$_4$	3
	Corrosion Rates [i-cos]	15 um/yr^{-1}	0.1 um/yr^{-1}	1.2 um/yr^{-1}	1.3 um/yr^{-1}	4
	Concrete Cover [i-cov]	5.16”	4.29”	4.67”	0.3”	1
Moisture	Humidity			98%		3
					Total	**24**

Very High Risk > 41	High Risk 31 – 40	Medium Risk 21 – 30	Low 16 -20	Very Low 10 - 15

GLOSSARY OF TERMS

Acid: Containing an excess of hydrogen ions over hydroxyl ions.

Alkaline: Containing an excess of hydroxyl ions over hydrogen ions.

Anode Zone: An anode zone is defined as the area of the impressed current cathodic protection system which is powered by an independent DC power supply unit.

Alternative Current (AC): The movement of electric charge which periodically reverses direction.

Anion: A negatively charged ion of an electrolyte, which migrates toward the anode under the influence of a potential gradient.

Anode: The electrode of an electrochemical cell at which oxidation occurs. (Electrons flow away from the anode in the external circuit. It is usually the electrode where corrosion occurs and metal ions enter solution.

Anodic Area: The part of the metal surface that acts as an anode.

Attenuation: Electrical losses in a conductor caused by current flow in the conductor.

Carbonation: The chemical reaction between carbon dioxide and the calcium hydroxide present in Portland cement.

Carbonation of Concrete: Carbon dioxide present in the atmosphere combines with moisture in the concrete to form carbonic acid. This reacts with the calcium hydroxide and other alkaline hydroxides in the pore water resulting in a reduction in the alkalinity of the concrete. The rate at which this neutralization occurs is influenced by factors such as moisture levels and concrete quality.

Cathode: The electrode of an electrochemical cell at which reduction is the principal reaction. (Electrons flow toward the cathode in the external circuit.).

Cathodic Area: The part of the metal surface that acts as a cathode.

Cathodic Protection: A technique to protect metal from further corrosion by making the protected metal a cathode of an electrochemical cell.

Cation: A positively charged ion of an electrolyte, which migrates toward the cathode under the influence of a potential gradient.

Chloride: The negative ion in salt (sodium chloride), found in sea salt, deicing salt and calcium chloride admixture for concrete. Chloride ions promote corrosion of steel in concrete but are not used up by the process so they can concentrate and accelerate corrosion.

Chloride Diffusion: Chloride ions enter concrete by the concentration gradient from an external resource, such as sea water, deicing salts, etc. It obeys the Fick's 2nd Law.

Conductor: A substance (mainly a metal or carbon) in which electric current flows by the movement of electrons.

Copper/Copper Sulfate Reference Electrode ($Cu/CuSO_4$): A reference electrode consisting of copper in a saturated copper sulfate solution.

Corrosion: The deterioration of a material, usually a metal, that results from a chemical or electrochemical reaction with its environment

Corrosion Potential (E_{corr}): The potential of a corroding surface in an electrolyte relative to a reference electrode under open-circuit conditions (also known as *rest potential, open-circuit potential, or freely corroding potential*).

Corrosion Rate: The weight loss of a corrosion coupon after exposure to a corrosive environment, expressed as mils (thousandths of an inch) per year penetration.

Current Density: The electric current flowing to or from a unit area of an electrode surface.

Direct Current (DC): Unidirectional flow of electrical charge.

Depolarization: The removal of factors resisting the current in an electrochemical cell.

Electrical Continuity: A closed circuit (unbroken electrical path) between metal components under consideration.

Electrical Isolation: The condition of being electrically separated from other metallic structures or the environment.

Electrochemical Cell: Electrochemical cell consisting of an anode and a cathode immersed in an electrolyte. The anode and cathode may be separate metals or dissimilar area on the same metal.

Electrode Potential: The potential of an electrode in an electrolyte as measured against a reference electrode. (The electrode potential does not include any resistance losses in potential in either the electrolyte or the external circuit. It represents the reversible work to move a unit of charge from the electrode surface through the electrolyte to the reference electrode.)

Electrolyte: A chemical substance containing ions that migrate in an electric field.

Electromotive Force Series (EMF): A list of elements arranged according to their standard electrode potentials. A sign being positive for elements whose potentials are cathodic to hydrogen and negative for those anodic to hydrogen.

Energizing: The process of initially applying power to turn on an electrical system.

Equilibrium Potential: The electrode potential with reference to a standard equilibrium.

External DC Power Source: DC Current is provided by an external transformer rectifier or battery.

Extraneous: Existing on or coming from the outside or not forming an essential or vital part.

Foreign Structure: Any metallic structure that is not intended as a part of a system under cathodic protection or electro osmosis.

Galvanic Anode: A metal provides sacrificial protection to another metal that is nobler in the galvanic series when coupled in an electrolyte due to its relative position in the galvanic series. These anodes are the current source in galvanic/sacrificial cathodic protection.

Galvanic Series: A list of metals and alloy arranged according to their relative potentials in a given environment.

General Corrosion: Corrosion in a uniform manner, usually quite predictable.

Half Cell: A half-cell is half of an electrolytic or voltaic cell, where either oxidation or reduction occurs. The half-cell reaction at the anode is oxidation, while the half-cell reaction at the cathode is reduction.

Immediate Voltage Shift: The difference between the potential value when the power source is on and the instant off value. (This is also referred to as *IR Drop*.)

Impressed Current: The connection of an external DC power source between the anode and the cathode.

Instant-Off Potential: The polarized half-cell potential of an electrode taken immediately after the cathodic protection current is stopped, which closely approximates the potential IR drop (i.e., the polarized potential) when the current was on.

Insulating Coating System: All components comprising the protective coating provide effective electrical insulation of the coated structure.

Ion: An electrically charged atom or group of atoms.

IR Drop: The voltage across a resistance in accordance with Ohm's Law.

Linear Polarization Resistance: A technique to measure corrosion rate. It requires to polarize the steel with an electric current and monitors its effect on the half cell potential. The change in the half cell potential is simply related to the corrosion current by the equation $I_{corr}=B/R_p$ when B is a constant and R_p is the polarization resistance. It is decided by the change in potential divided by applied current.

Mixed Potential: A potential resulting from two or more electrochemical reactions occurring simultaneously on one metal surface.

Native Potential: See Corrosion Potential.

Open-Circuit Potential: The potential of an electrode measured with respect to a reference electrode or another electrode in the absence of current.

Oxidation: Loss of electrons by a constituent of a chemical reaction.

Passive Film: The passive film is a thin, dense layer or iron oxides and hydroxides with some mineral content. It is initially formed when bar steel is exposed to oxygen and water but then protects the steel from further corrosion due to its high density which doesn't allow humidity and oxygen to reach the steel.

Passivation: The process by which steel in concrete is protected from corrosion by the formation of a passive film due to the highly alkaline environment in concrete.

Pitting Corrosion: Localized corrosion of a metal surface that is confined to a small area and takes the form of cavities called pits.

Polarization: The change from the open-circuit potential as a result of current across the electrode/electrolyte interface.

Polarization Decay: The decrease in electrode potential with time resulting from the interruption of applied current.

Potential Survey: Obtaining potentials with respect to a reference electrode at multiple locations on the surface of the concrete structure.

Protection Current: The current made to flow into a metallic structure, with respect to a specified reference electrode in an electrolytic environment, to effect cathodic protection of the structure.

Reaction: A process of chemical or electrochemical change, particularly taking place at or near an electrode in an electrochemical cell.

Rectifier: An electrical device for converting alternating current (AC) to direct current (DC).

Reduction: Gain of electrons by a constituent of a chemical reaction.

Reference Electrode: An electrode whose open-circuit potential is constant under similar conditions of measurement, which is used for measuring the relative potentials of other electrodes.

Rest Potential: See Corrosion Potential.

Rust: Corrosion product consisting of various iron oxides and hydrated iron oxides. (This term properly applies only to iron and ferrous alloys.)

Sacrificial Protection: Reduction or prevention of corrosion of a metal in an electrolyte by galvanically coupled it to a more anodic metal.

Silver/Silver Chloride Reference Electrode [Ag/AgCl]: A reference electrode consisting of silver, coated with silver chloride, in an electrolyte containing chloride ions.

Step-and-Touch Potentials: The electrical potential gradients that may exist between two points on the electrolyte surface equal to one pace (one meter) or between a grounded metallic object and a point on the electrolyte surface separated by the distance equal to a human's normal reach (one meter).

Stray Current: Current through paths other than the intended circuit.

Stray Current Corrosion: Corrosion resulting from direct current flowing through paths other than the intended circuit.

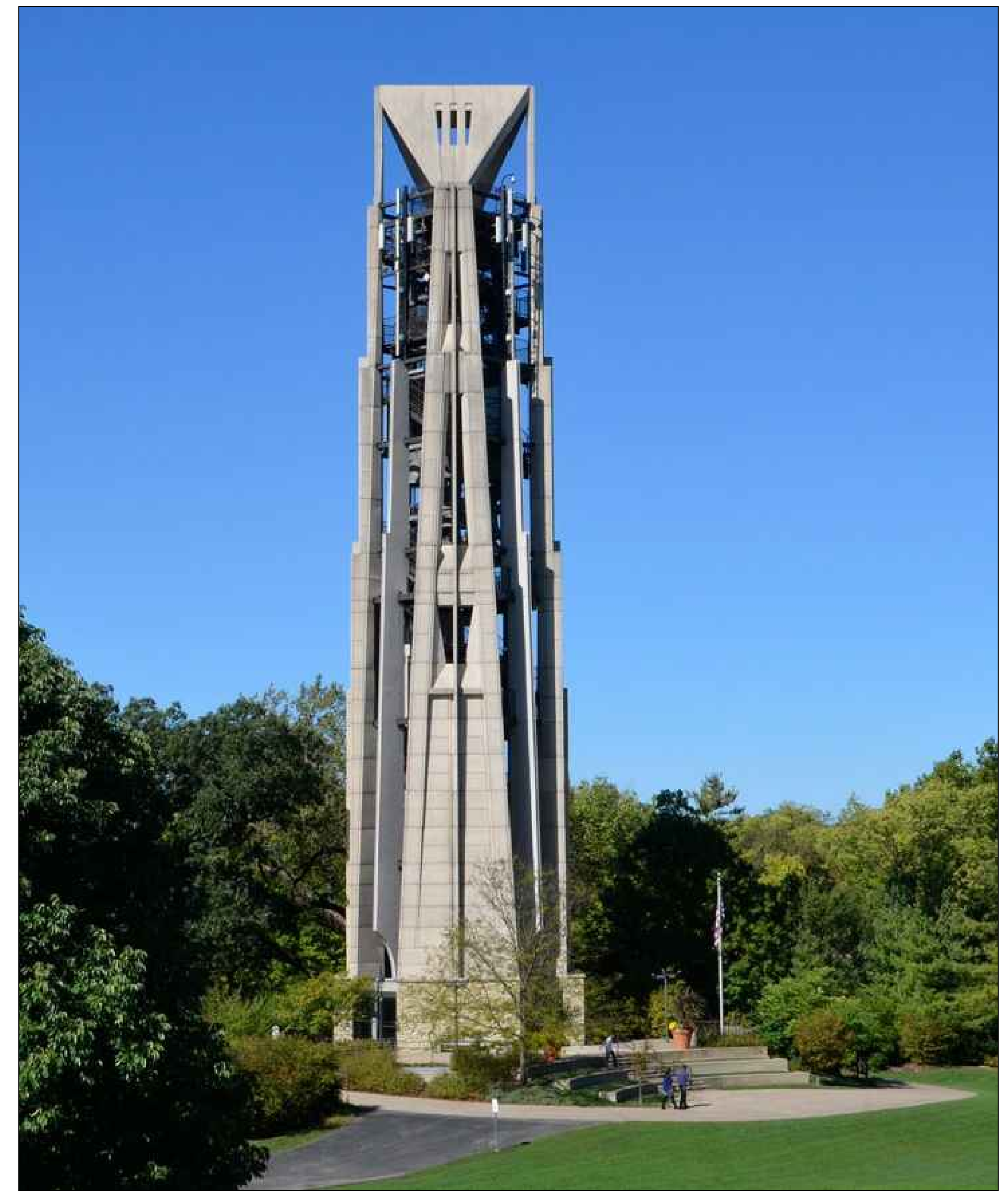
Structure-to-Electrode Potential: The voltage difference between a buried or embedded metallic steel and electrolyte that is measured with reference to an electrode in contact with the electrolyte.

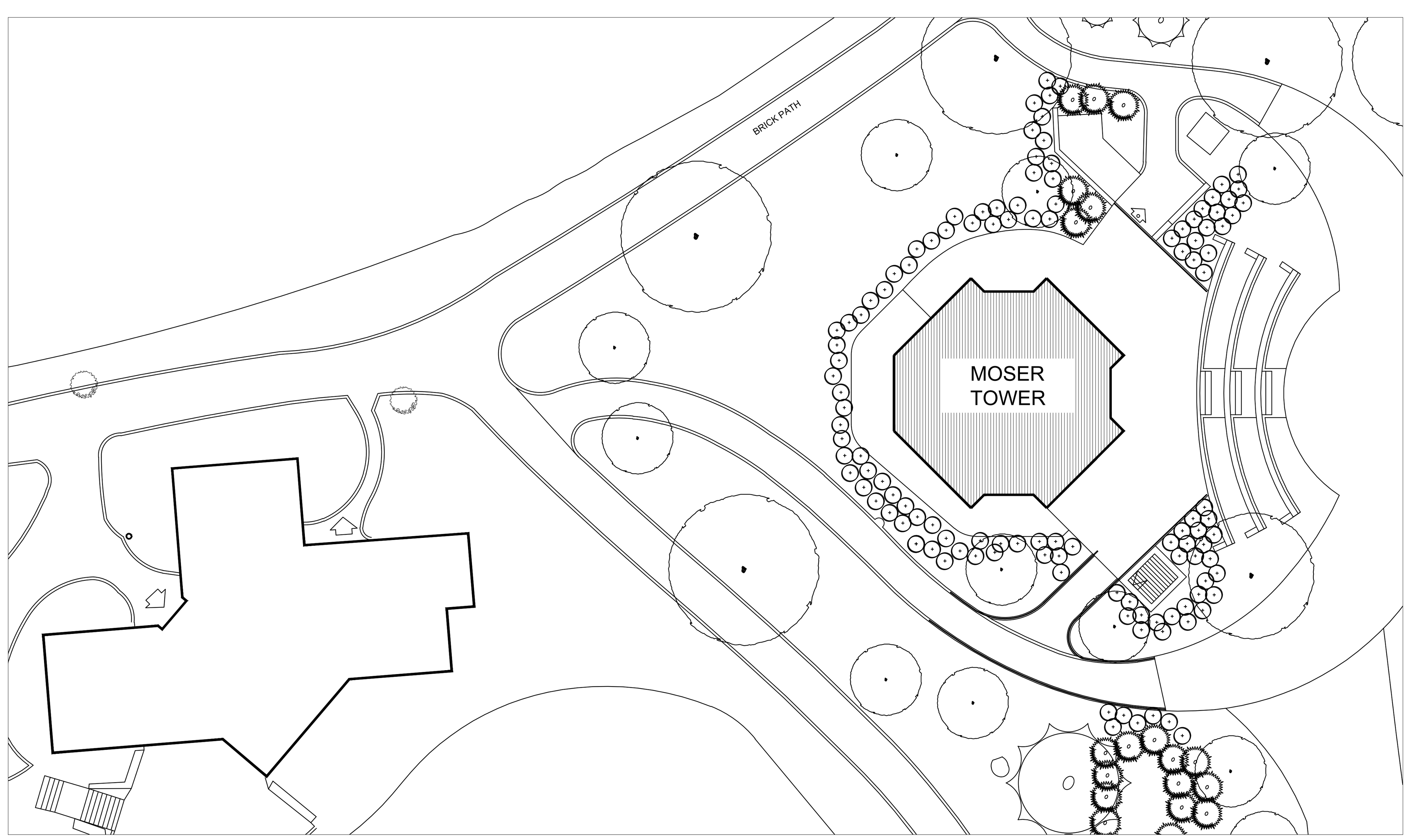
APPENDIX A: DRAWINGS

Moser Tower & Carillon

Naperville, Illinois

CORROSION ASSESSMENT



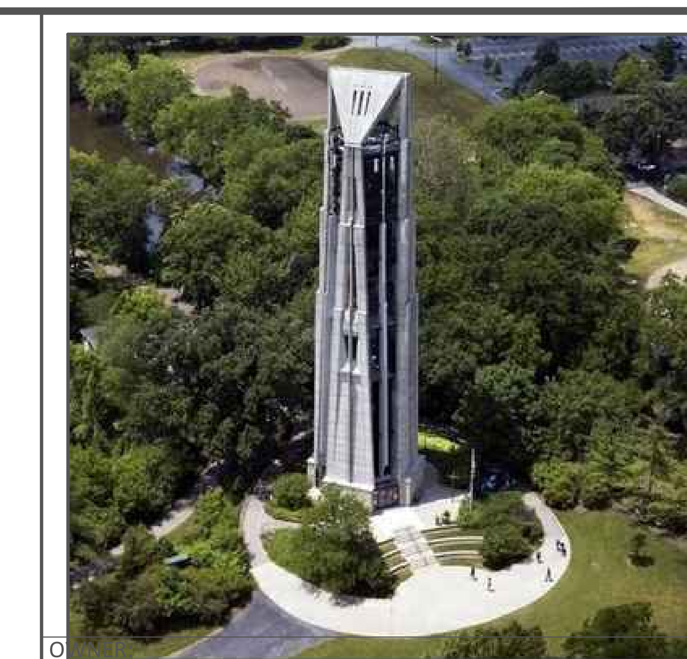


1 **SITE MAP**
NTS

DRAWING REGISTER

SHEET No.	SHEET TITLE	ISSUE 08/24/18
X001	COVER SHEET	00
X010	GENERAL NOTES	00
X101	LOWER LEVEL PLAN	00
X102	FIRST FLOOR PLAN	00
X103	BUILDING PLANS	00
X104	BUILDING PLANS	00
X201	EAST & NORTH ELEVATIONS	00
X-202	WEST, SOUTHWEST & SOUTH ELEVATIONS	00
X-301	REINFORCEMENT LAYOUT - FIRST FLOOR BEAM	00
X-302	REINFORCEMENT LAYOUT - PRE-CAST FIN PANEL	00
X-303	REINFORCEMENT LAYOUT - PANEL	00
X401	SOUTHWEST PRE-CAST CORROSION RATE (i-COS) & RESISTIVITY (i-ROM) CONTOUR MAPS	00
X402	INTERIOR PRE-CAST PANELS CORROSION RATE (i-COS) CONTOUR MAPS	00
X-403	INTERIOR RESISTIVITY (i-ROM) LOCATIONS / MAPS	00
X404	NORTH / BEAMS CORROSION POTENTIALS (i-VOS), CORROSION RATES (i-COS) & RESISTIVITY (i-ROM) CONTOUR MAPS	00
X501	CORE LOCATIONS	00
X-502	CHLORIDE TESTING / TEMPERATURE & HUMIDITY TESTING	00

2 **DRAWING REGISTER**



CITY OF NAPERVILLE
NAPERVILLE, ILLINOIS

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CHICAGO, IL. 60618

ENGINEER OF RECORD:
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MATERIAL CONSULTANT:
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Poughkeepsie, NY 12601

e²chem CONSULTANTS LLC

ISSUE:	DESCRIPTION:	DATE:	DRWN	CHK	APPR
	ISSUED TO CLIENT	08/24/18	NMF	IM	PN

DWG SCALE:
CAD DWG FILE: RE-E17258-NF-V1.0_Moser Tower.dwg

KEY MAP:

PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

SHEET TITLE:
COVER SHEET

PROJECT REFERENCE NUMBER:
E17258

SHEET NUMBER: X-001

REVISION: 00

ANSI D SIZE DRAWING 22 X 34

GENERAL NOTES:

MATERIALS CONDITION SURVEY TESTS AND IDENTIFICATION OF LOCATIONS

1. LPR, ELECTRICAL RESISTIVITY AND REINFORCEMENT LAYOUT (SPR).

CORROSION RATE TESTING

MEASUREMENTS OF THE RATE OF CORROSION ARE USUALLY MADE USING SENSORS ON THE SURFACE OF THE CONCRETE.

THE MOST COMMON SURFACE METHOD FOR DETERMINING CORROSION RATES IS BY UTILIZING A METHOD KNOWN AS LINEAR POLARIZATION RESISTANCE (LPR). THE LPR MEASUREMENT IS WHERE A SMALL (20 MV) POTENTIAL DIFFERENCE IS APPLIED BETWEEN THE STEEL AND A SECONDARY ELECTRODE ON THE SURFACE WHICH RESULTS IN A SMALL CURRENT FLOW. THIS IS THEN PROPORTIONAL TO THE INVERSE OF THE POLARIZATION RESISTANCE AND HENCE IS DIRECTLY PROPORTIONAL TO THE CORROSION RATE.

AS PART OF THIS TEST THE STEEL POTENTIAL IS ALSO MEASURED. THE COMBINATION OF THESE TESTS ALLOW FOR AN ASSESSMENT OF THE MOST ACTIVE AREAS OF CORROSION TO BE IDENTIFIED. WITH LPR TESTING AND AN UNDERSTANDING OF THE STRUCTURE, THE LOCATION OF UNDERLYING FACTORS THAT MAY BE DIRECTLY AFFECTING THE CORROSION OF THE STEEL MAY BE MORE EASILY IDENTIFIED. EARLY DETECTION OF THESE FACTORS CAN GUIDE THE REPAIR DESIGN PROCESS FOR THE MOST EFFECTUAL TREATMENTS.

THE LOSS OF SECTION IS DETERMINED FROM MEASURING THE AMOUNT OF STEEL DISSOLVING AND FORMING OXIDE (RUST). THIS IS CARRIED OUT BY DETERMINING THE ELECTRIC CURRENT GENERATED AT THE ANODIC REACTION. THE FOLLOWING CHARTS SHOW RATES, CURRENT PENETRATION, AND DAMAGE.

CORROSION RATE TESTING

CORROSION RATES OF STEEL IN CONCRETE AND MASONRY

RATE OF CORROSION	CORROSION CURRENT DENSITY (I_{corr})$\mu A/cm^2$	CORROSION PENETRATION M/YR
HIGH	10-100	100-1000
MEDIUM	1-10	10-100
LOW	0.1-1	1-10
PASSIVE	<0.1	<1

CORROSION RATE AND REMAINING SERVICE LIFE

icorr ($\mu A/cm^2$)	icorr ($\mu A/in^2$)	SEVERITY OF CORROSION DAMAGE
<0.2	<0.031	NO DAMAGE EXPECTED
0.2-1.0	0.031 TO 0.155	DAMAGE POSSIBLE IN 10 TO 15 YEARS
1.0-10	0.155 TO 1.55	DAMAGE EXPECTED IN 2 TO 10 YEARS
>10	>1.55	DAMAGE EXPECTED IN 2 YEARS OR LESS

TYPICAL SECTION LOSS

THE FOLLOWING TABLE IS BASED ON AN AVERAGE OF 3 TIMES THE VOLUME OF OXIDE.

icorr ($\mu A/in^2$)	METAL LOSS (mpy)	SECTION LOSS
<0.0155	0.04334	SECTION LOSS 0.13MPY RUST GROWTH
0.0775	0.22458	SECTION LOSS 0.67MPY RUST GROWTH
0.155	0.45704	SECTION LOSS 1.37MPY RUST GROWTH
1.55	4.5704	SECTION LOSS 13.7MPY RUST GROWTH

NOTE: THE EXPANSIVE OXIDE GROWTH BETWEEN 0.394MIL (10μm) AND 3.94MIL (100μm) (0.01 to 0.1mm WILL CAUSE CRACKING).

ELECTRICAL RESISTIVITY

ELECTRICAL RESISTIVITY TESTS MEASURE THE RESISTANCE OF THE CONCRETE OR MASONRY TO FLOW CURRENT. HIGHER RESISTIVITY VALUES CORRELATE TO A GREATER RESISTANCE OF THE CONCRETE TO THE FLOW OF IONS, AND THUS A LOWER PROBABILITY THAT REINFORCING STEEL CORROSION IS OCCURRING. RESISTIVITY VALUES ARE MEASURED IN KILO OHMS/cm^2 OR KΩ cm^2.

RESISTIVITY MEASUREMENTS CAN BE USED TO ESTIMATE THE LIKELIHOOD OF CORROSION. WHEN THE ELECTRICAL RESISTIVITY (r) OF THE CONCRETE IS LOW, THE LIKELIHOOD OF CORROSION INCREASES. EMPIRICAL TESTS HAVE ARRIVED AT THE FOLLOWING TYPICAL VALUES FOR THE MEASURED RESISTIVITY WHICH CAN BE USED TO DETERMINE THE LIKELIHOOD OF CORROSION. THESE FIGURES ARE FOR ORDINARY PORTLAND CEMENT AT 20°C (RESIPOD OPERATING INSTRUCTIONS MANUAL).

WHEN ≥ 100 KΩ cm	NEGLIGIBLE RISK OF CORROSION
WHEN = 50 TO 100 KΩ cm	LOW RISK OF CORROSION
WHEN = 10 TO 50 KΩ cm	MODERATE RISK OF CORROSION
WHEN ≤ 10 KΩ cm	HIGH RISK OF CORROSION

ACCORDING TO BROOMFIELD ET. AL 1993, RESISTIVITY LEVELS IN CORRELATION WITH POSSIBLE CORROSION CURRENT OUTPUT ARE AS FOLLOWS:

>20 KΩ cm^2	LOW CORROSION RATE
10-20 KΩ cm^2	LOW TO MODERATE CORROSION RATE
5-10 KΩ cm^2	HIGH CORROSION RATE
<5 KΩ cm^2	VERY HIGH CORROSION RATE

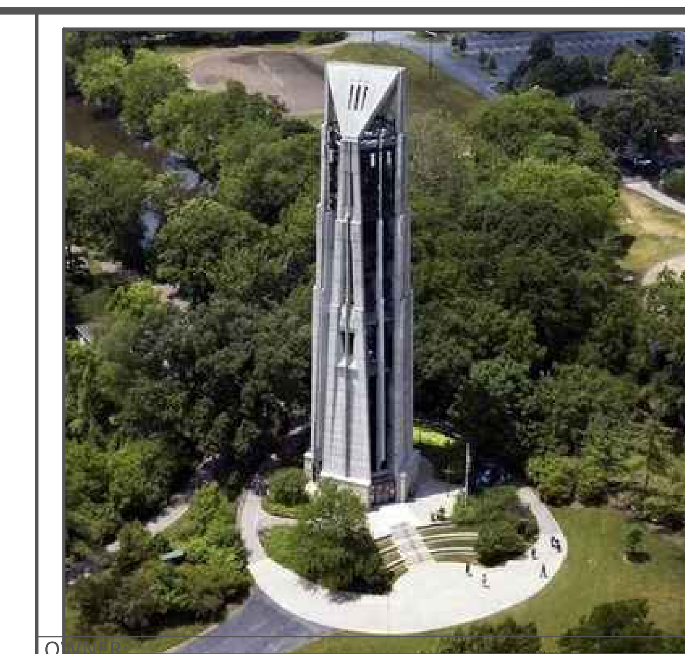
CARBONATED CONCRETE HAS A HIGHER RESISTIVITY THAN CONCRETE WITHOUT CARBONATION, HOWEVER PROVIDED THE DEPTH OF THE CARBONATED LAYER IS SIGNIFICANTLY SMALLER THAN THE PROBE SPACING, THE EFFECT OF THIS LAYER IS SMALL. CONSEQUENTLY IF THE CARBONATED LAYER IS THICK, IT MAY BE NECESSARY TO INCREASE THE PROBE SPACING TO OBTAIN GOOD RESULTS.

A MODIFIED FOUR-POINT WENNER RESISTIVITY METER IS USED. THESE PROBES ARE BASED ON SOIL RESISTIVITY METERS, AND HAVE BEEN MODIFIED FOR CONCRETE. THE MODIFIED PROBE USES FOUR EQUALLY SPACED PROBES ON A STRAIGHT LINE. THE PROBE SPACING IS APPROXIMATELY EQUAL TO THE DEPTH OF THE RESISTIVITY MEASUREMENT IN CONCRETE. WET SPONGES OR APPARATUSES ARE MOUNTED IN THE PROBES TO ENHANCE THE ELECTRICAL CONTACT BETWEEN THE PROBES AND THE CONCRETE SURFACES. THE RESISTIVITY OF THE CONCRETE IS A FUNCTION OF THE VOLTAGE DROP BETWEEN THE CENTER PAIR OF PROBES WITH THE CURRENT PROVIDED BY THE OUTSIDE PROBES.

MASONRY VALUES ARE MUCH HIGHER THAN STANDARD CONCRETE VALUES, SO THIS TEST CAN BE VERY USEFUL IN DETECTING CONTAMINATED AND SATURATED MASONRY.

SURFACE PENETRATING RADAR

SPR WORKS BY SENDING A TINY PULSE OF ENERGY INTO A MATERIAL AND RECORDING THE STRENGTH AND THE TIME REQUIRED FOR THE RETURN OF ANY REFLECTED SIGNAL. A SERIES OF PULSES OVER A SINGLE AREA MAKE UP WHAT IS CALLED A SCAN. REFLECTIONS ARE PRODUCED WHENEVER THE ENERGY PULSE ENTERS INTO A MATERIAL WITH DIFFERENT ELECTRICAL CONDUCTION PROPERTIES OR DIELECTRIC PERMITTIVITY FROM THE MATERIAL IT LEFT. THE STRENGTH, OR AMPLITUDE, OF THE REFLECTION IS DETERMINED BY THE CONTRAST IN THE DIELECTRIC CONSTANTS AND CONDUCTIVITIES OF THE TWO MATERIALS. MATERIALS WITH A HIGH DIELECTRIC WILL SLOW THE RADAR WAVE AND IT WILL NOW BE ABLE TO PENETRATE AS FAR. MATERIALS WITH A HIGH CONDUCTIVITY WILL ATTENUATE THE SIGNAL RAPIDLY. DATA IS COLLECTED IN PARALLEL TRANSECTS AND THEN PLACED TOGETHER IN THEIR APPROPRIATE LOCATIONS FOR COMPUTER PROCESSING IN A SPECIALIZED SOFTWARE PROGRAM.



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KEY MAP:

PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

SHEET TITLE:
GENERAL NOTES

PROJECT REFERENCE NUMBER:
E17258

SHEET NUMBER: X-010

REVISION: 00

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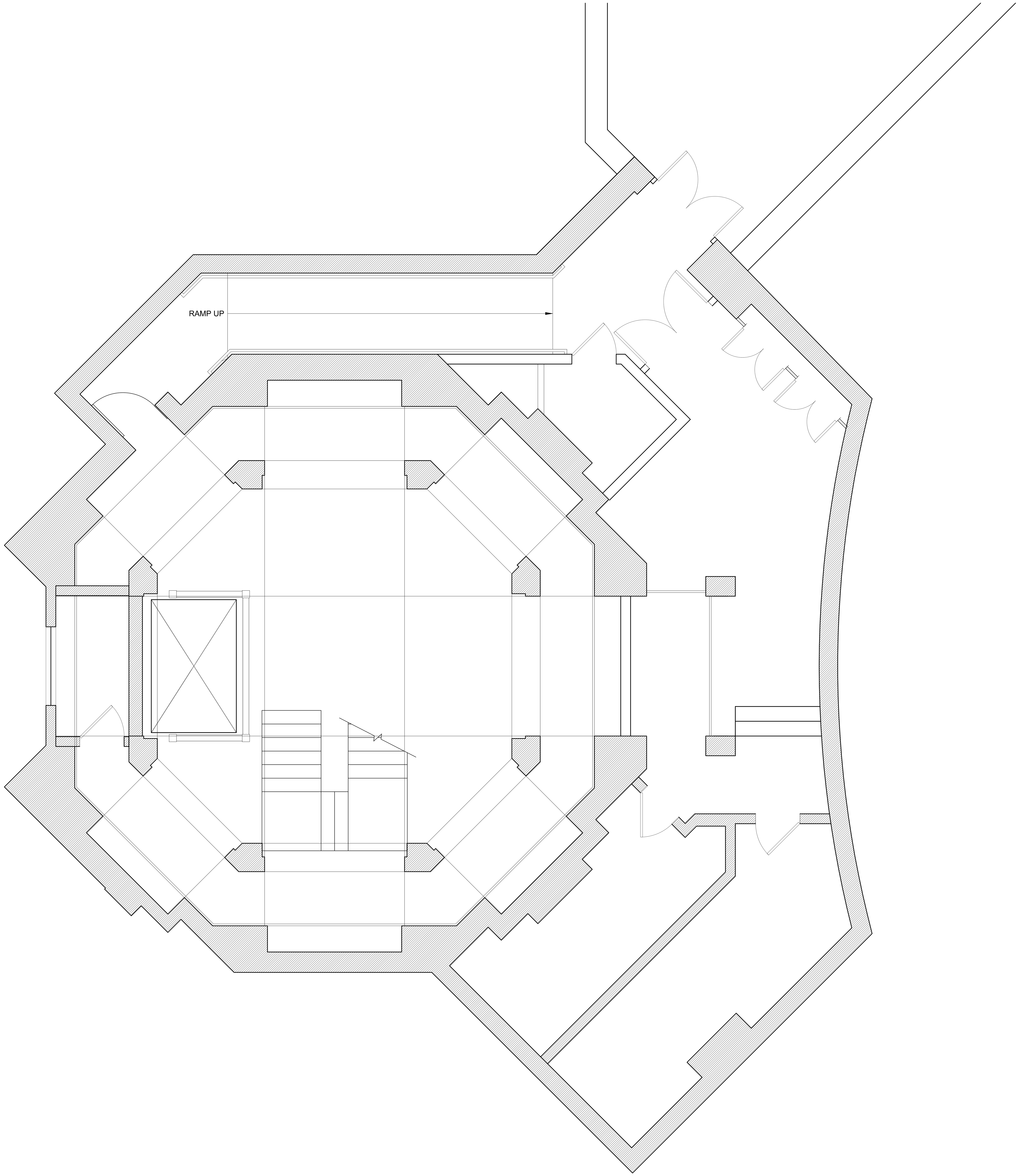
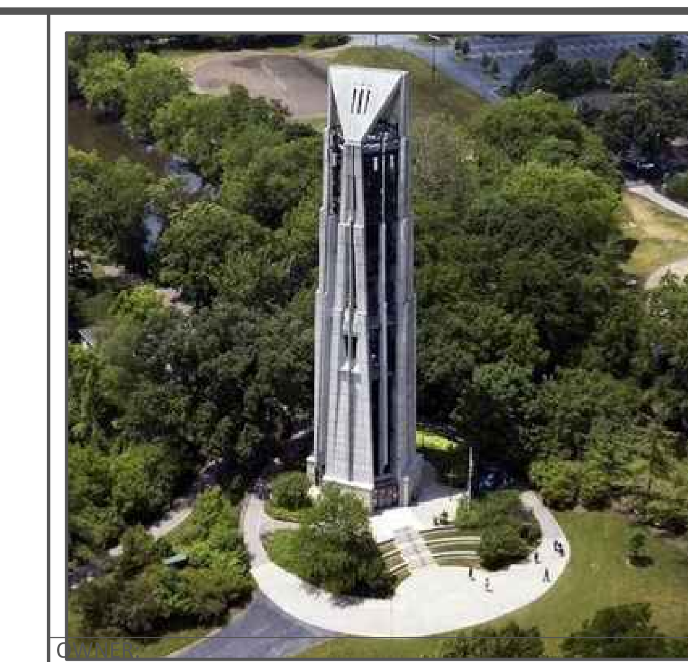


1 LOWER LEVEL BUILDING PLAN

SCALE: 1/4" = 1'-0"



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KEY MAP:

PROJECT:

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CORROSION ASSESSMENT

SHEET TITLE:

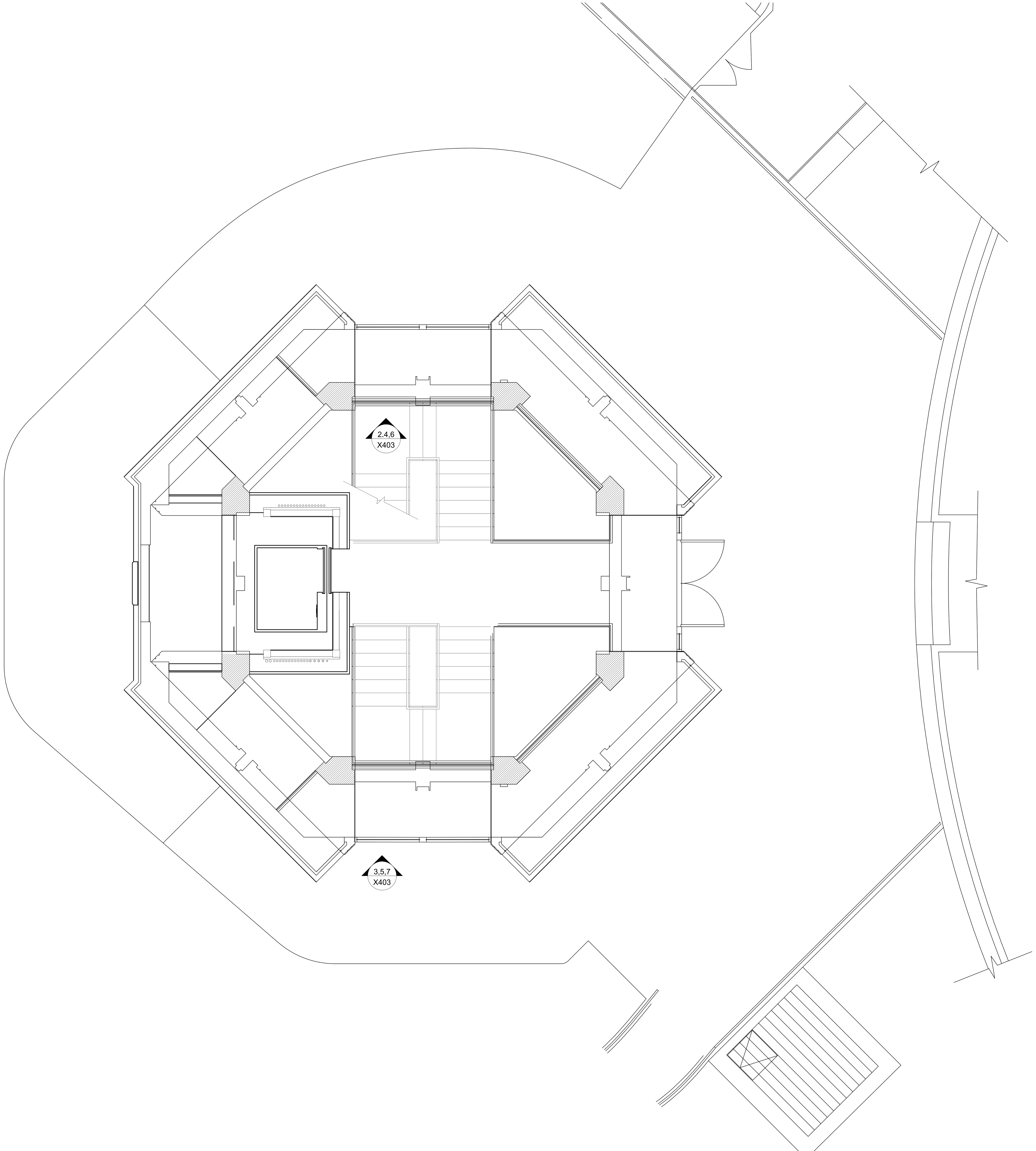
LOWER LEVEL PLAN

PROJECT REFERENCE NUMBER:

E17258

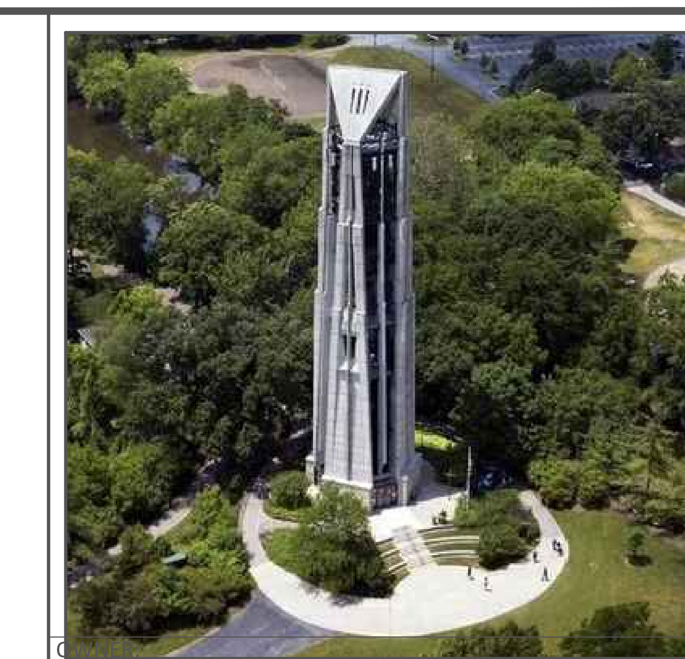
SHEET NUMBER: X-101

REVISION: 00



1 FIRST FLOOR BUILDING PLAN

SCALE: 1/4" = 1'-0"



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KEY MAP:

PROJECT:

MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

SHEET TITLE:

FIRST FLOOR PLAN

PROJECT REFERENCE NUMBER:
E17258

SHEET NUMBER: X-102

REVISION: 00

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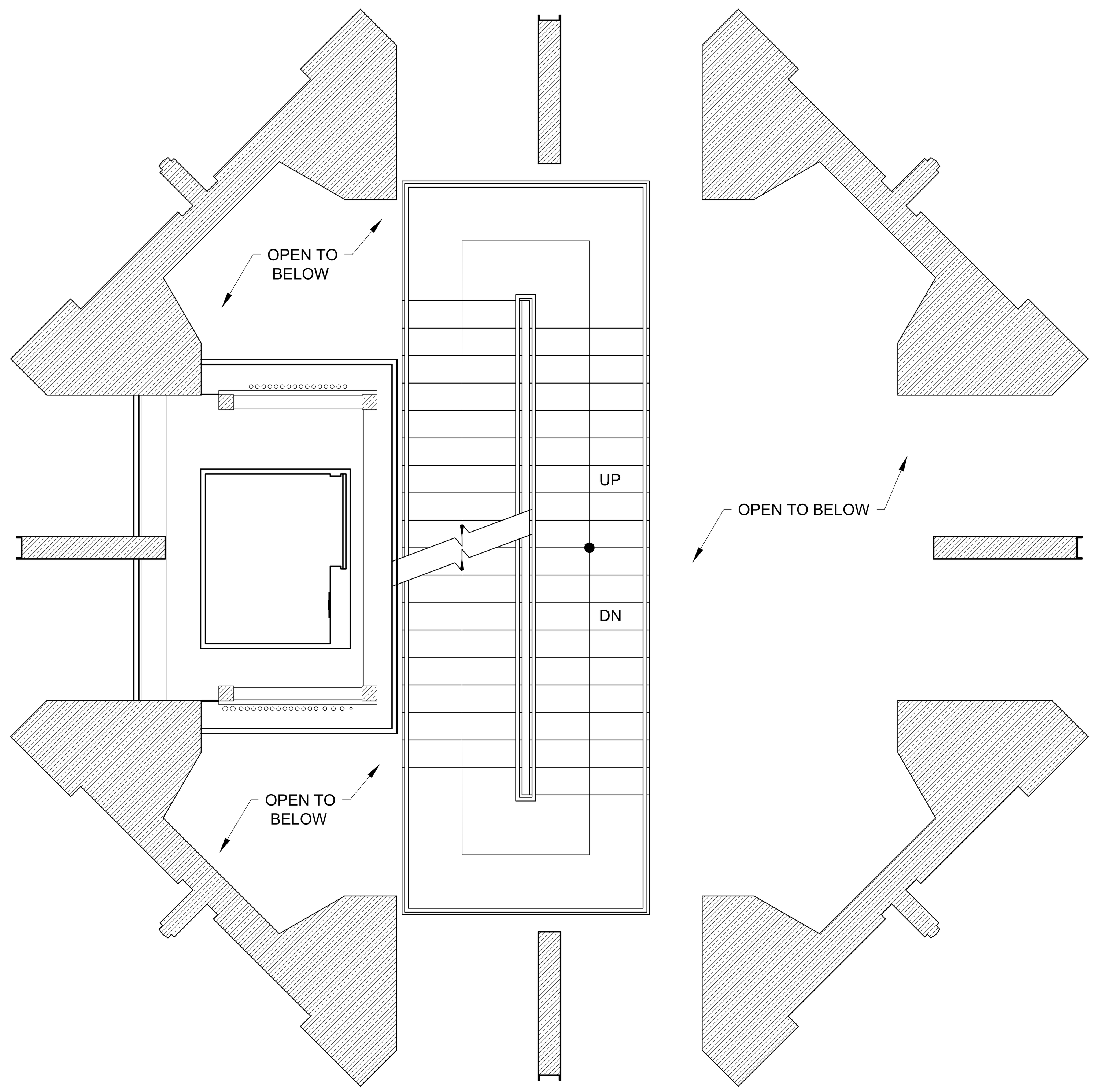
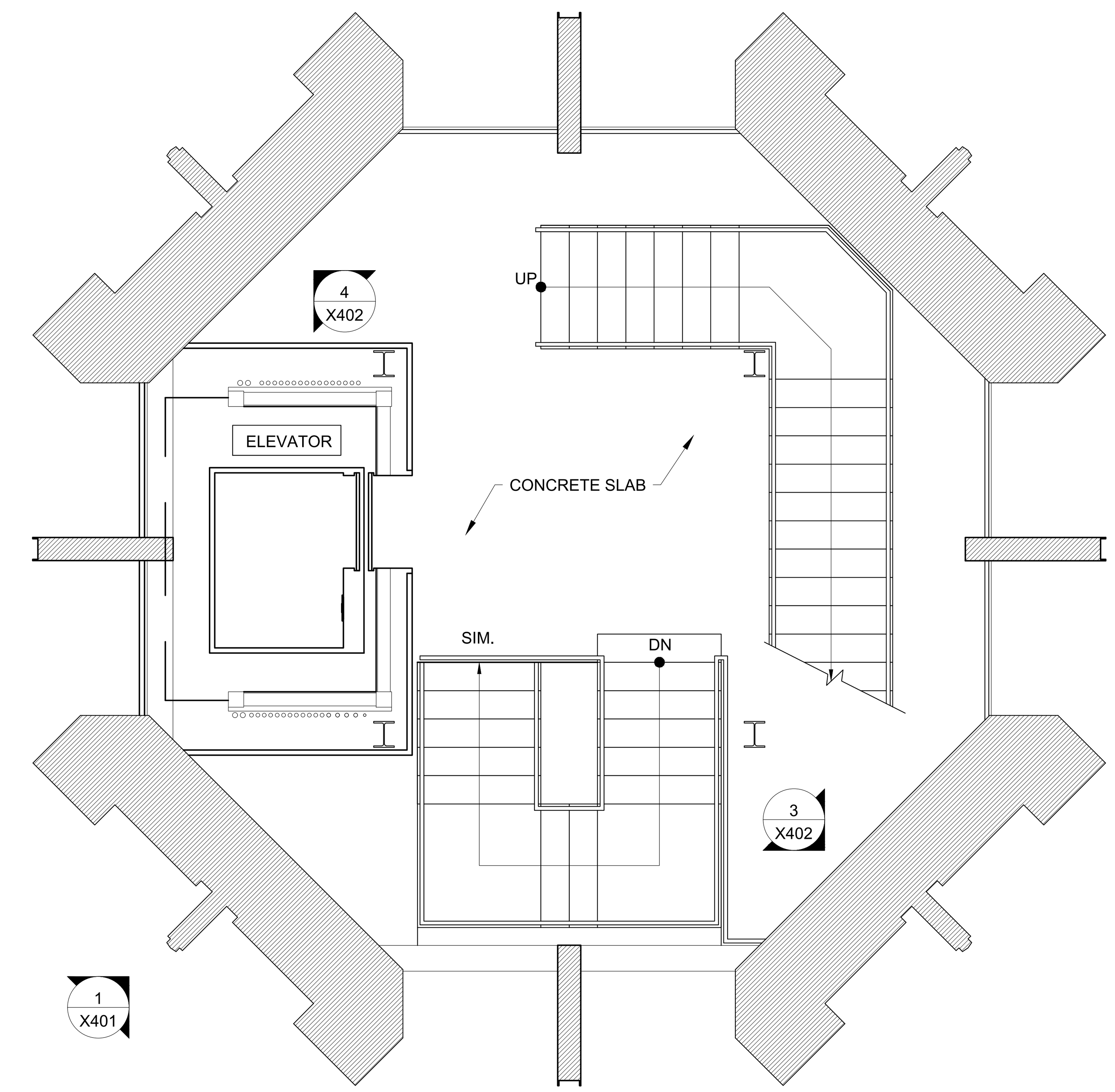
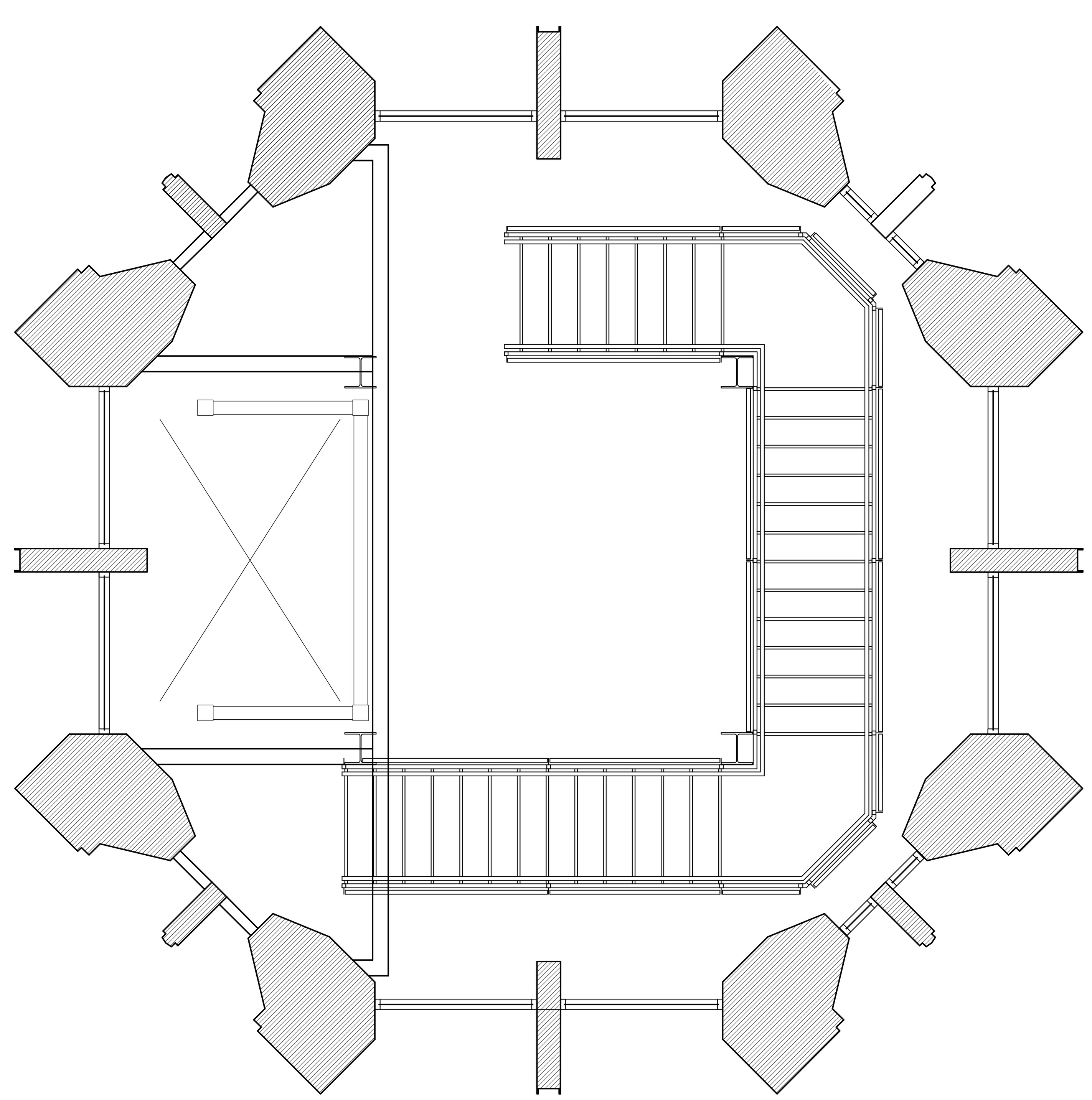


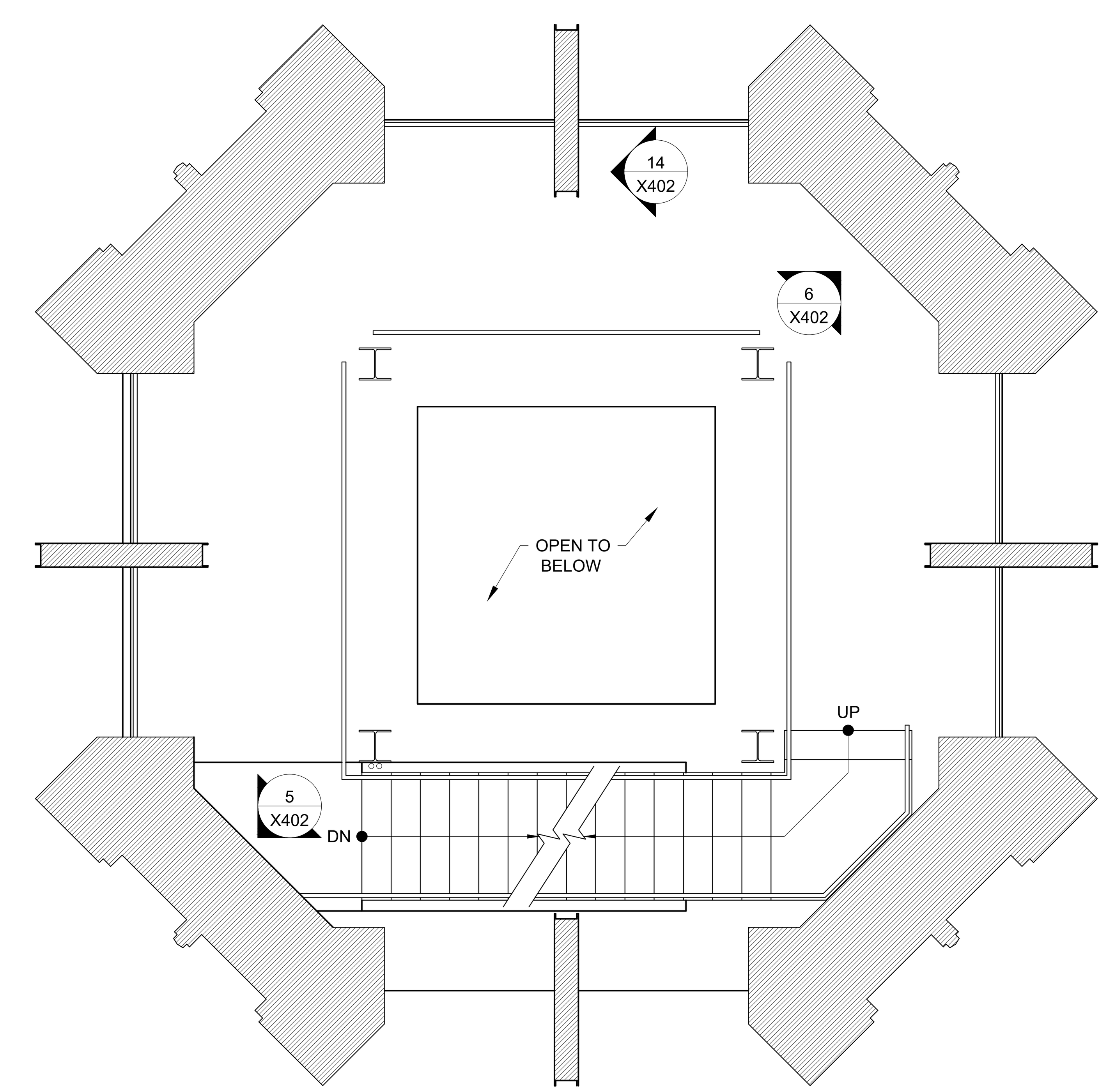
1 PLAN ABOVE ENTRY LEVEL EL. +34'-7"
SCALE: 1/4" = 1'-0"



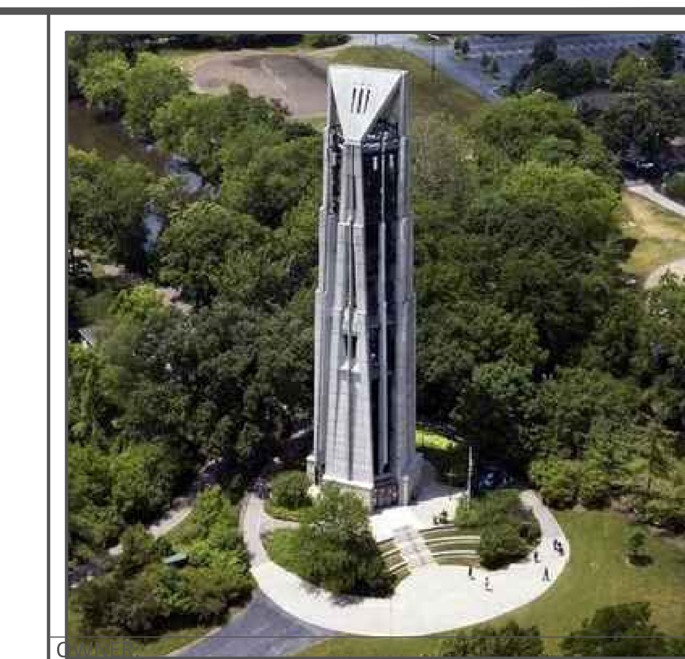
2 PLAN AT GALLERY T/ SLAB EL. +52'-6"
SCALE: 1/4" = 1'-0"



3 PLAN AT GALLERY MEZZANINE
SCALE: 1/4" = 1'-0"



4 PLAN AT BELFRY T/ SLAB EL. +72'-9"
SCALE: 1/4" = 1'-0"



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KEY MAP:

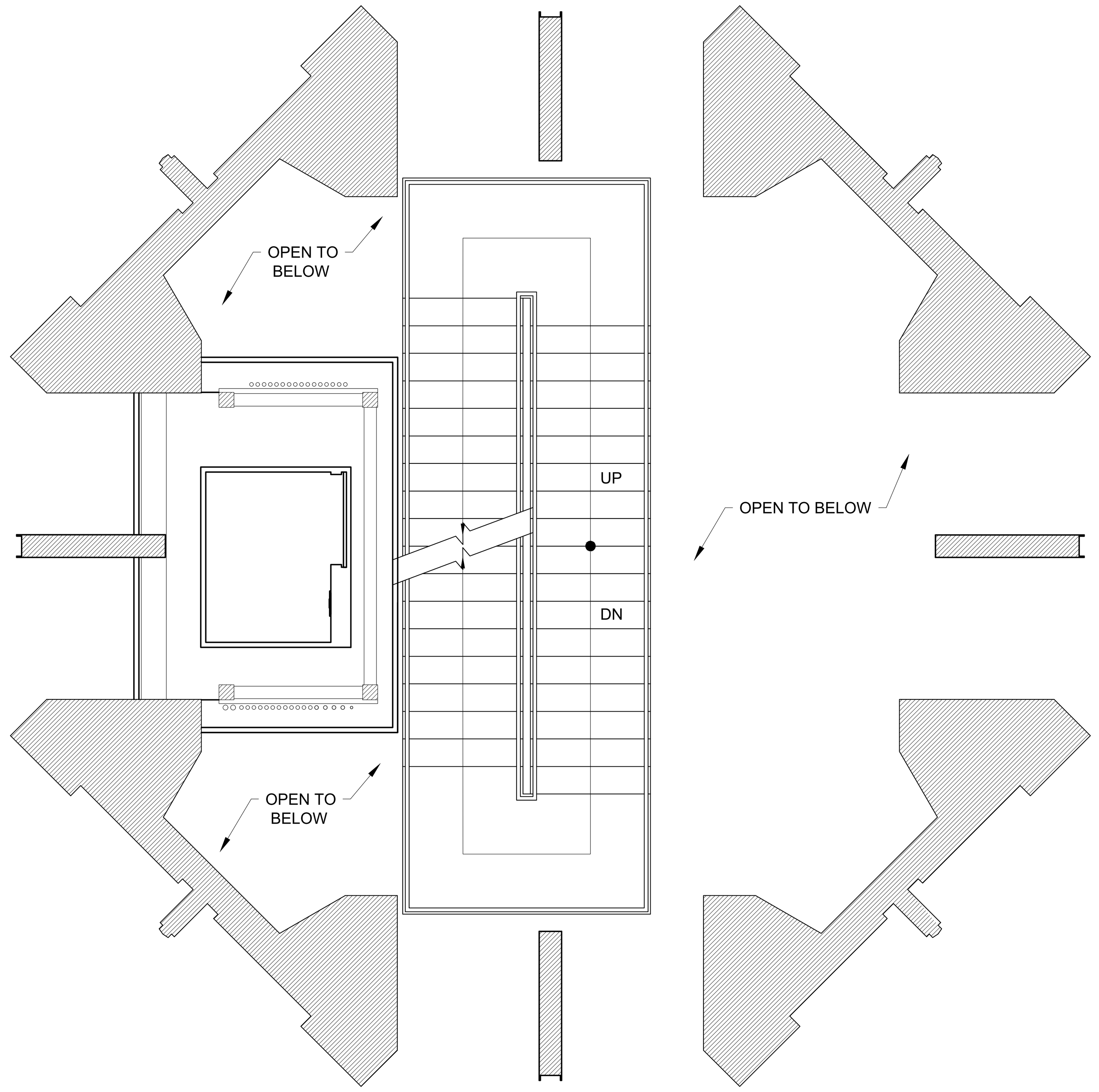
PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

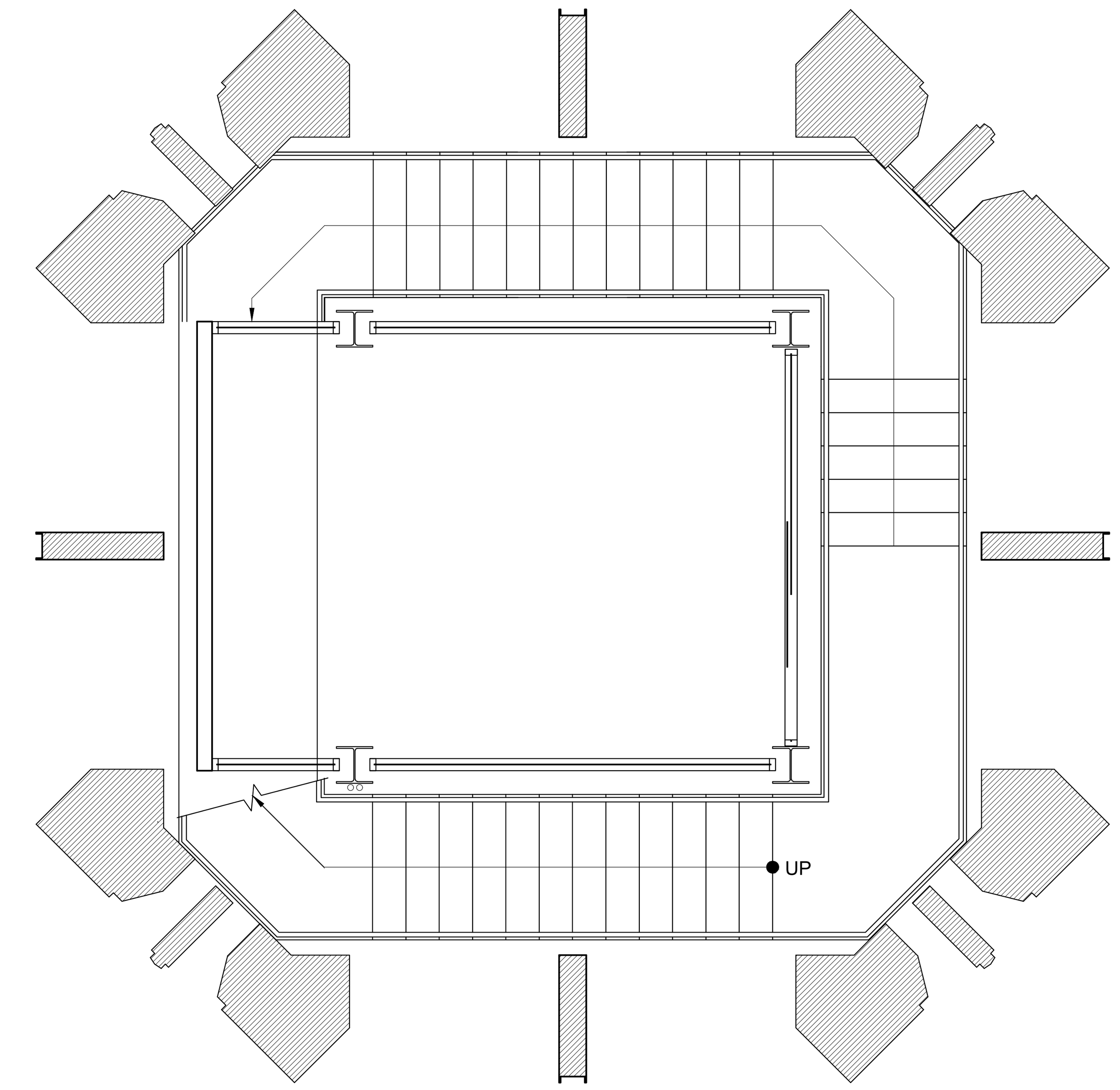
SHEET TITLE:
BUILDING PLANS

PROJECT REFERENCE NUMBER:
E17258

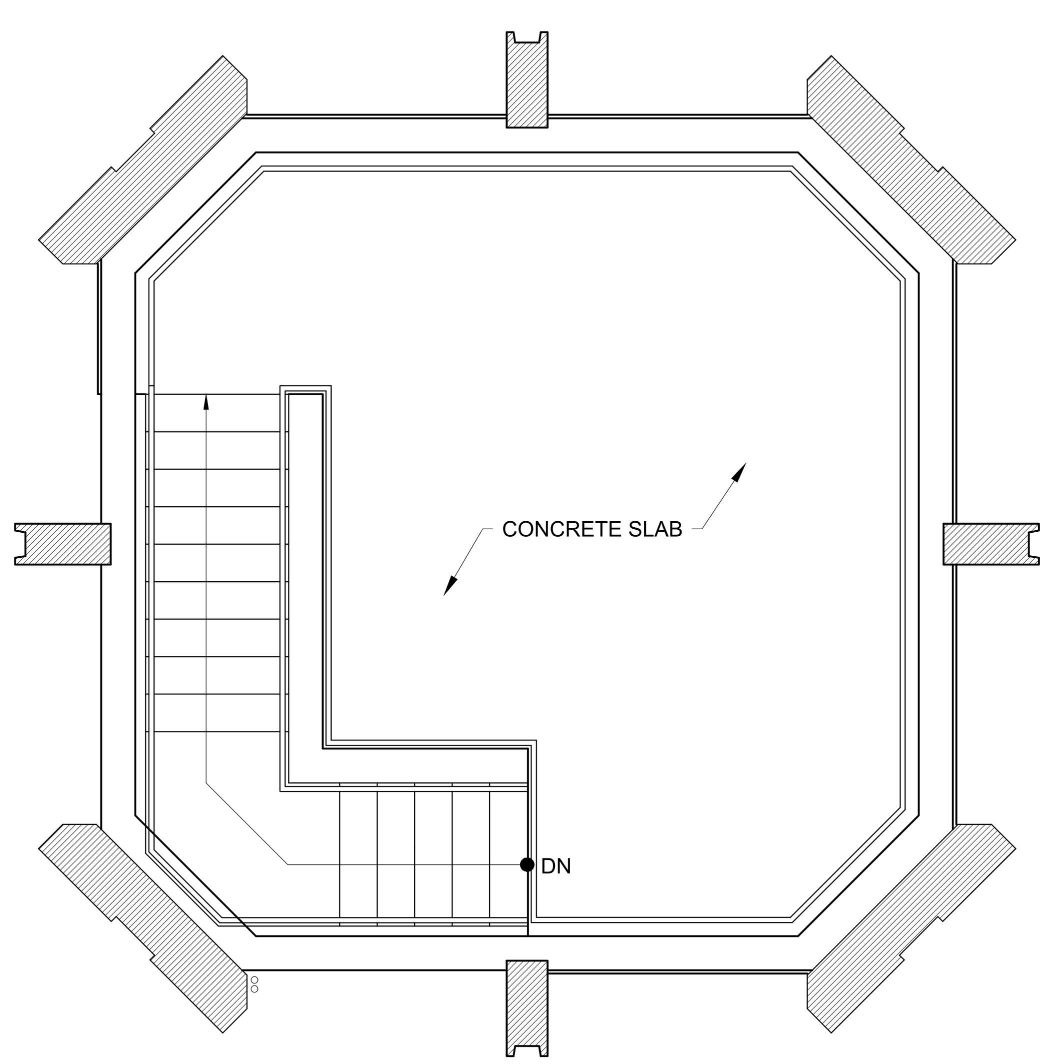
SHEET NUMBER: X-103
REVISION: 00



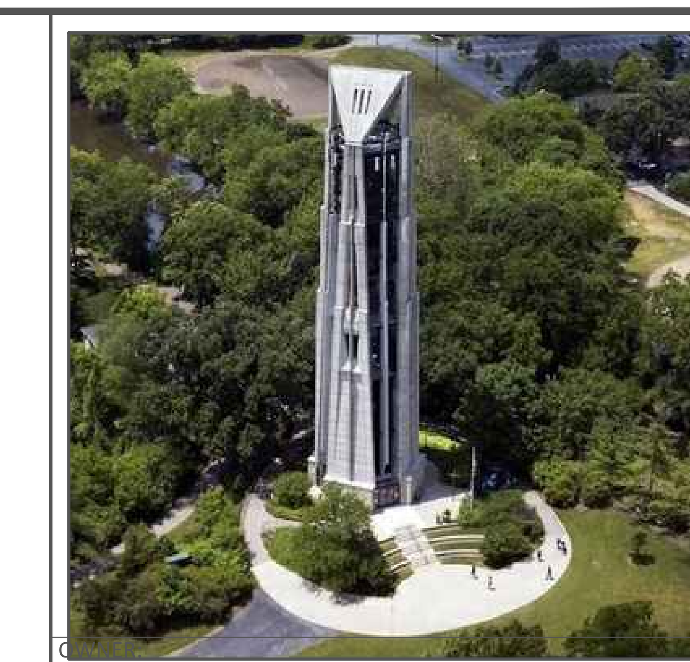
1 PLAN AT BELFRY T/ SLAB EL. +92'-4"
SCALE: 1/4" = 1'-0"



2 PLAN AT CARILLON CABIN T/ SLAB EL. +99'-7"
SCALE: 1/4" = 1'-0"



3 PLAN AT OBSERVATION LEVEL T/ SLAB EL. +136'-4"
SCALE: 1/4" = 1'-0"



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KEY MAP:

PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

SHEET TITLE:
BUILDING PLANS

PROJECT REFERENCE NUMBER:
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SHEET NUMBER: X-104
REVISION: 00

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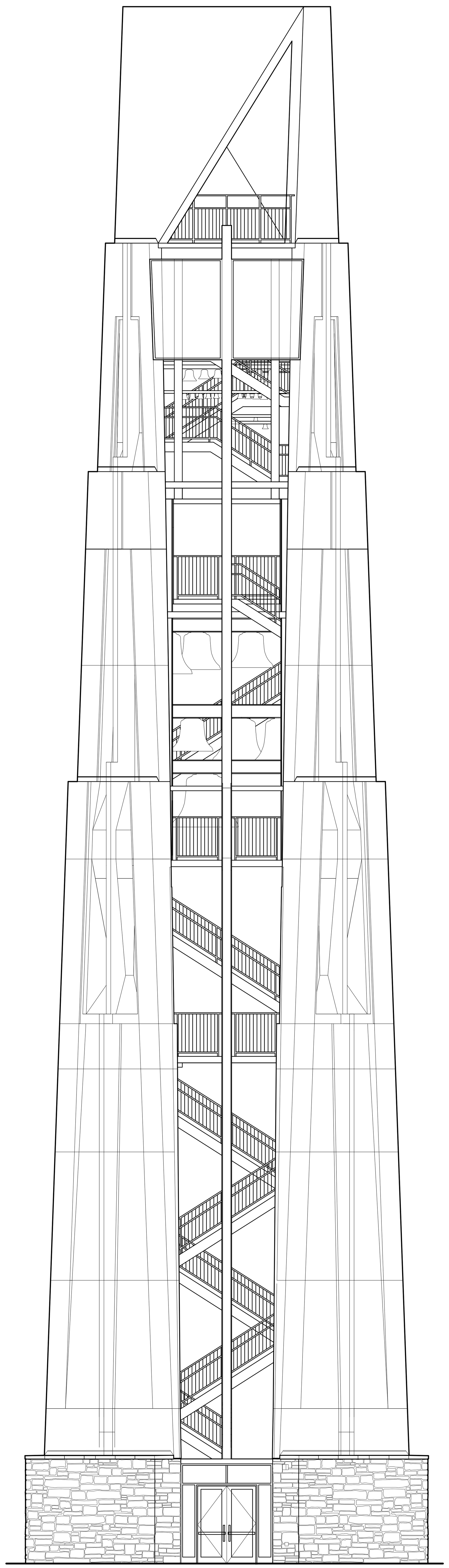
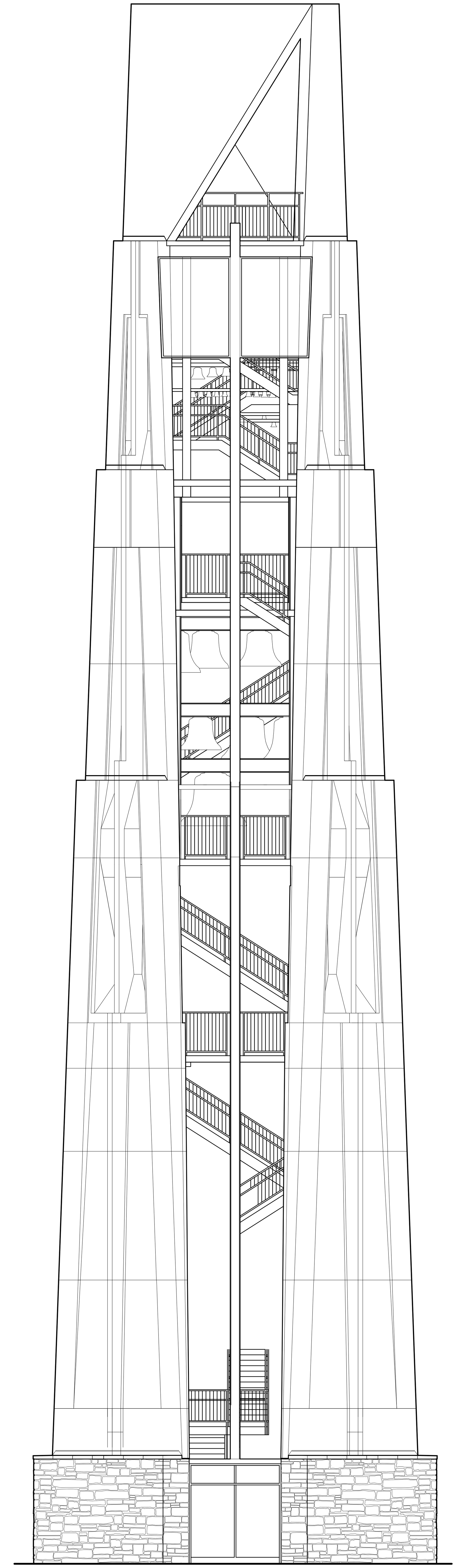
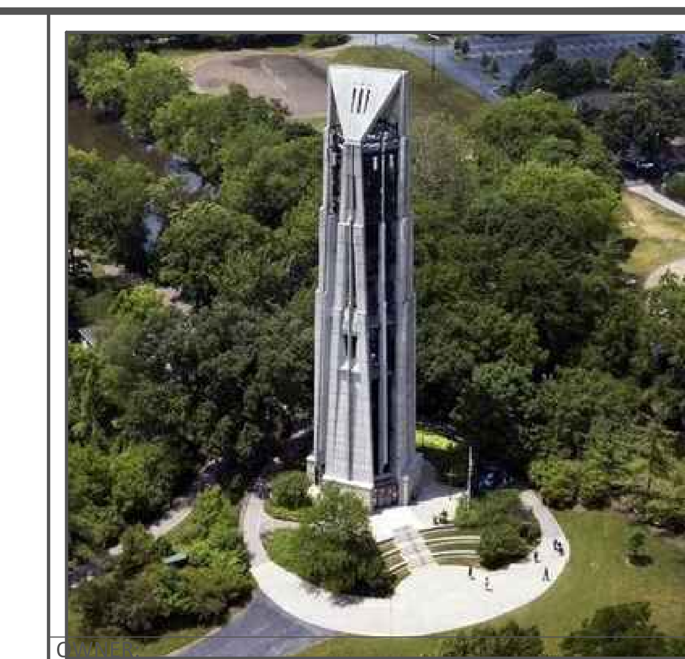


1 EAST ELEVATION (FRONT)
SCALE: 1/8" = 1'-0"



2 NORTH ELEVATION
SCALE: 1/8" = 1'-0"



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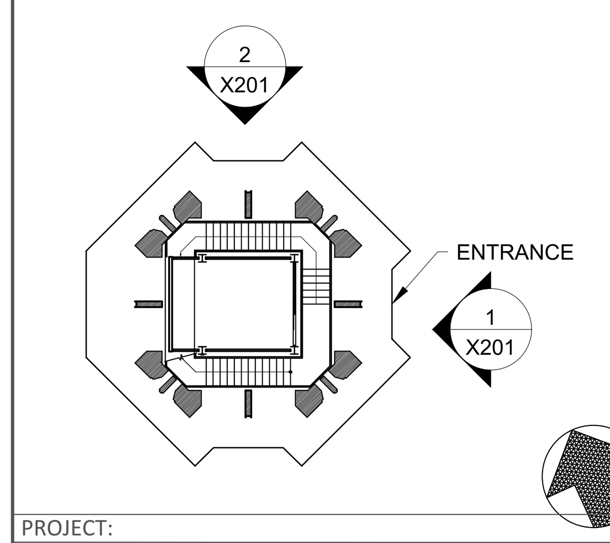


PROJECT:

MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

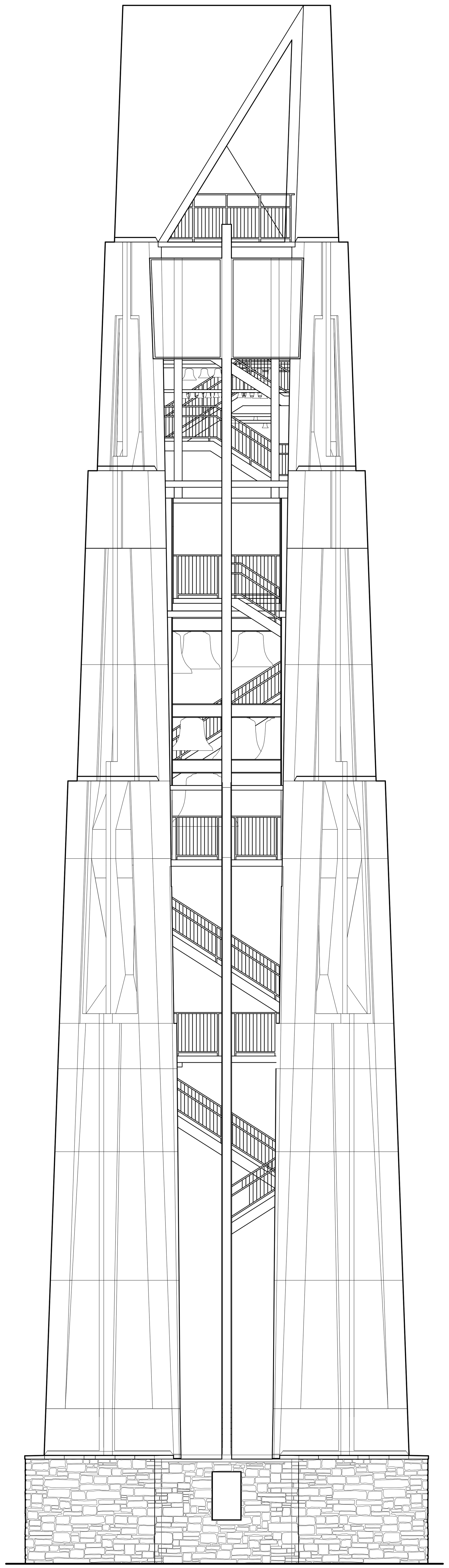
SHEET TITLE:

EAST & NORTH
ELEVATIONS

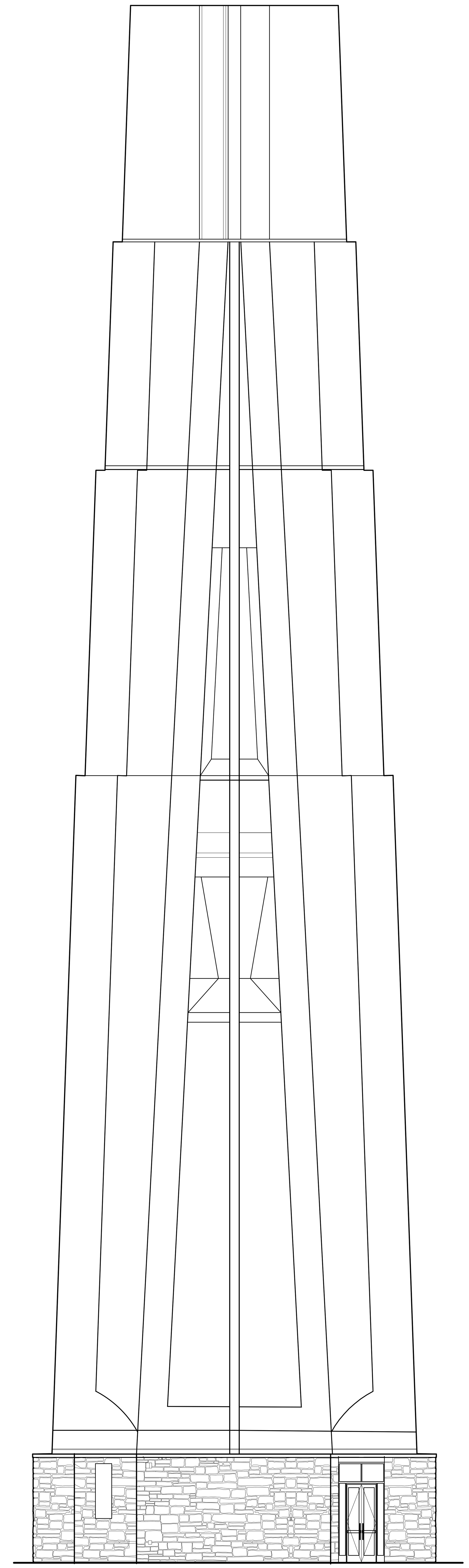
PROJECT REFERENCE NUMBER:
E17258

SHEET NUMBER: X-201
REVISION: 00

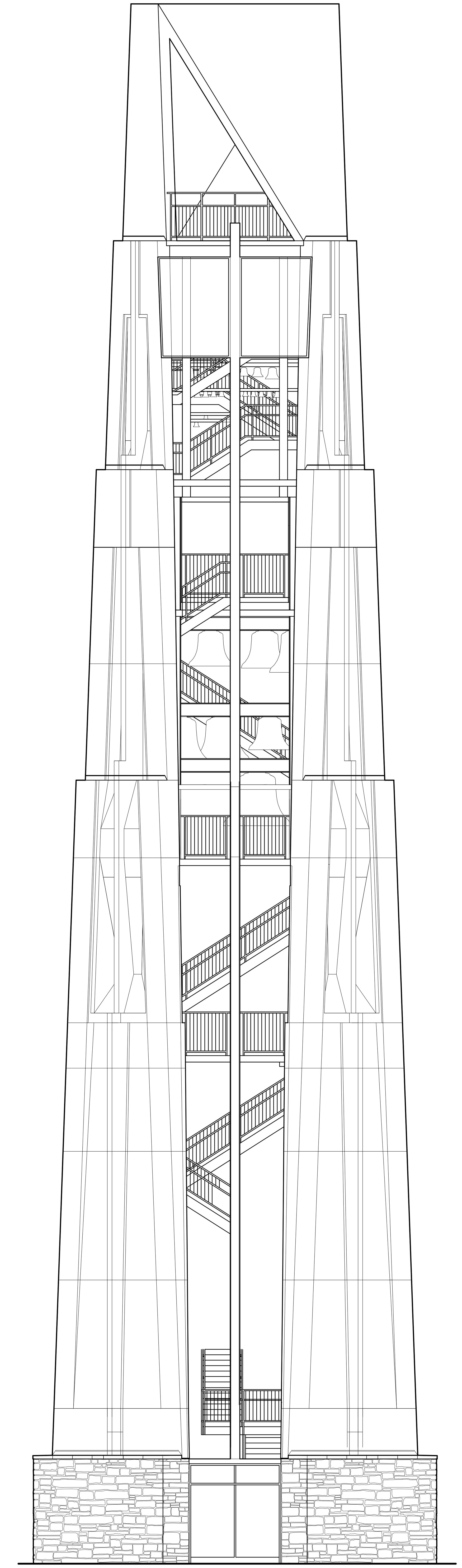
ANSI D SIZE DRAWING 22 X 34



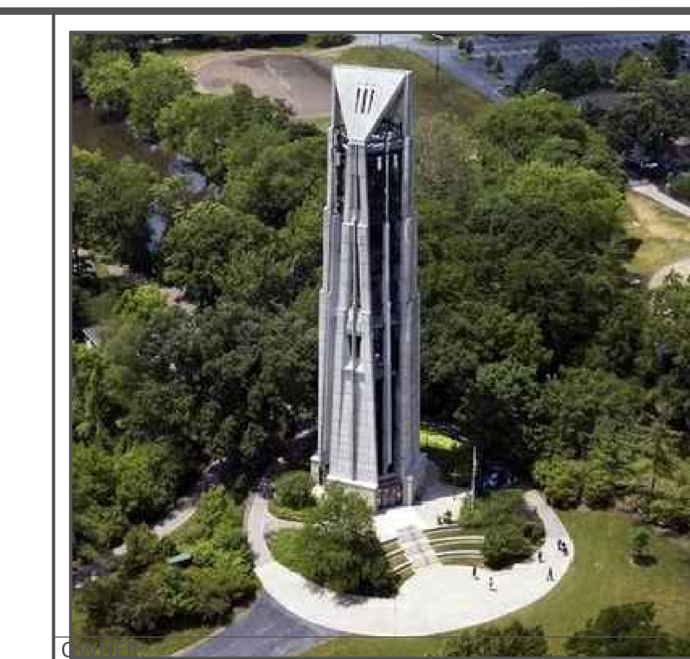
1 WEST ELEVATION (REAR)
SCALE: 1/8" = 1'-0"



2 SOUTHWEST ELEVATION
SCALE: 1/8" = 1'-0"



3 SOUTH ELEVATION
SCALE: 1/8" = 1'-0"



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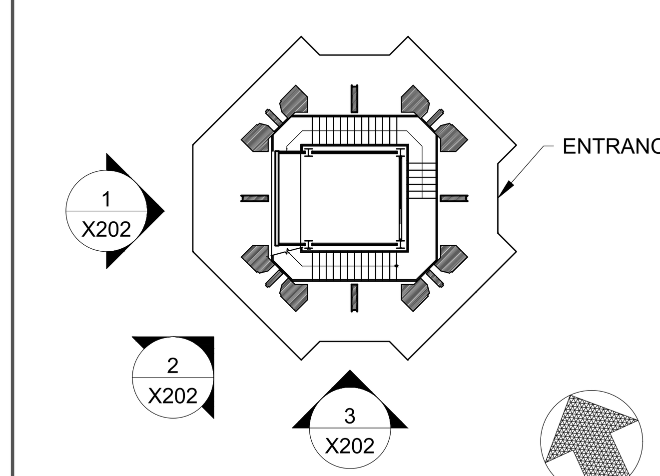


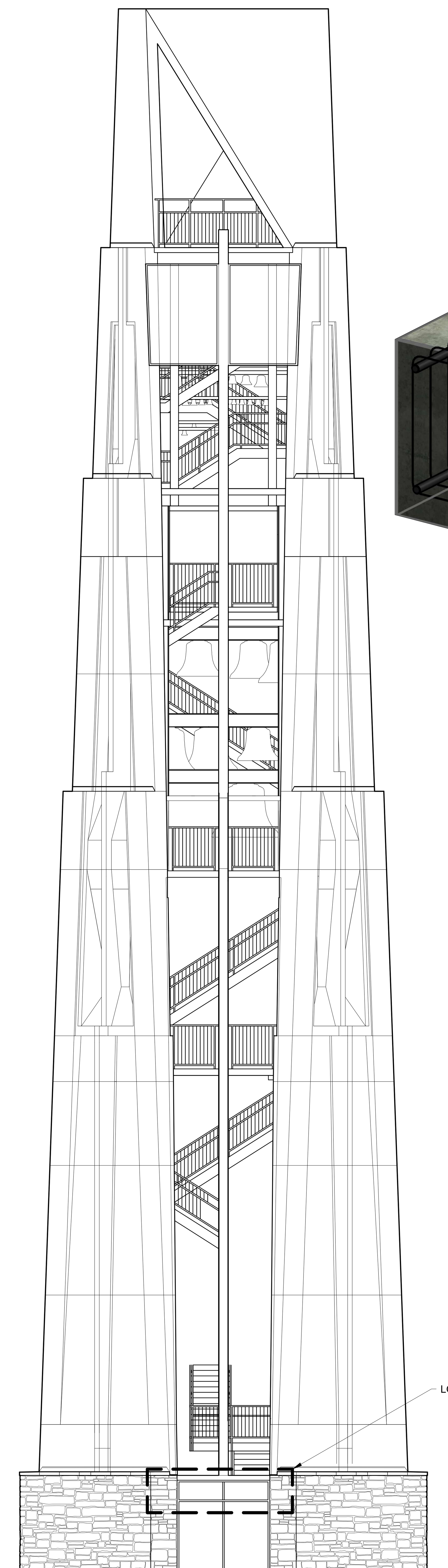
PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

SHEET TITLE:
WEST, SOUTHWEST & SOUTH ELEVATIONS

PROJECT REFERENCE NUMBER:
E17258

SHEET NUMBER: X-202
REVISION: 00



1 SOUTH ELEVATION
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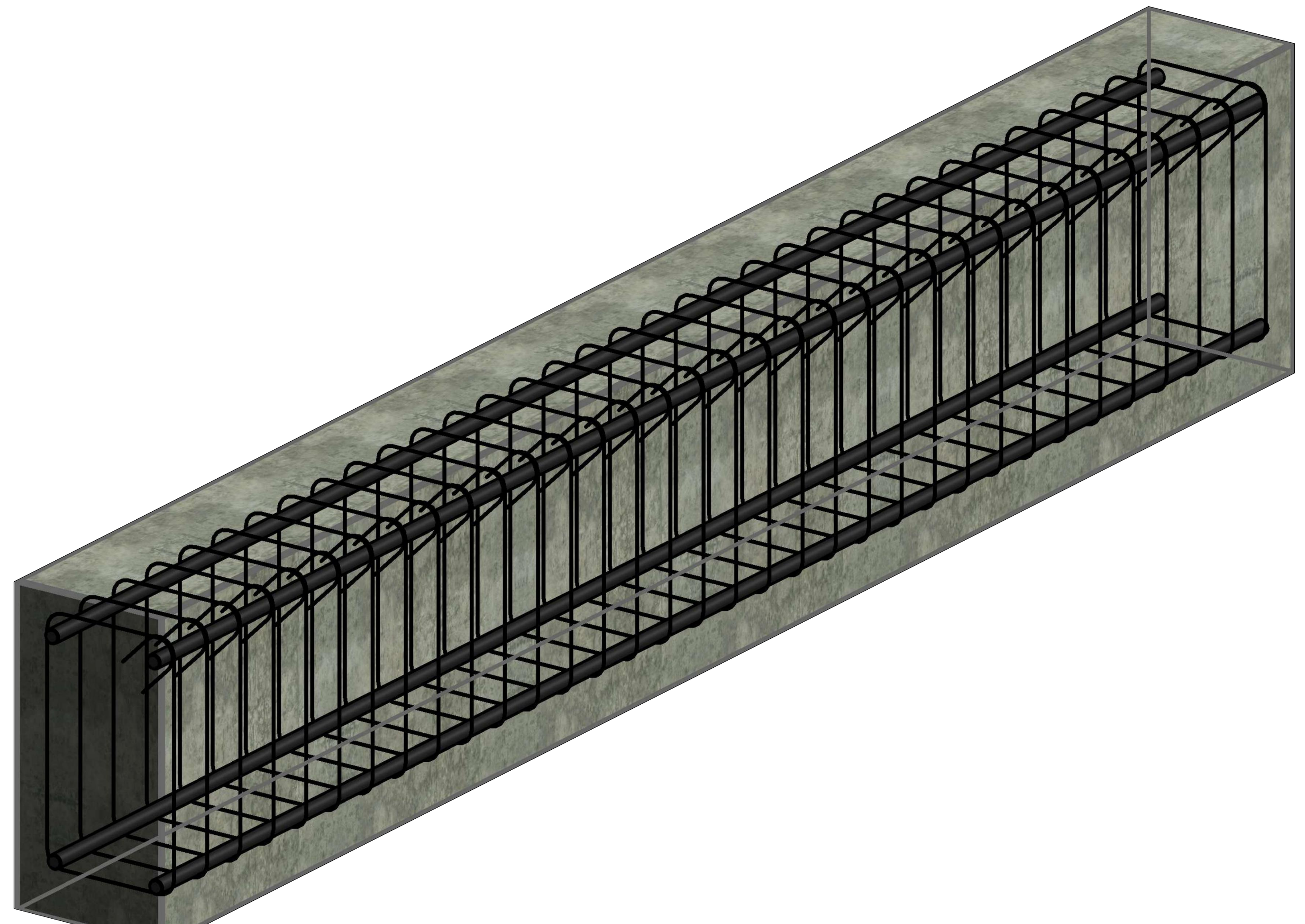
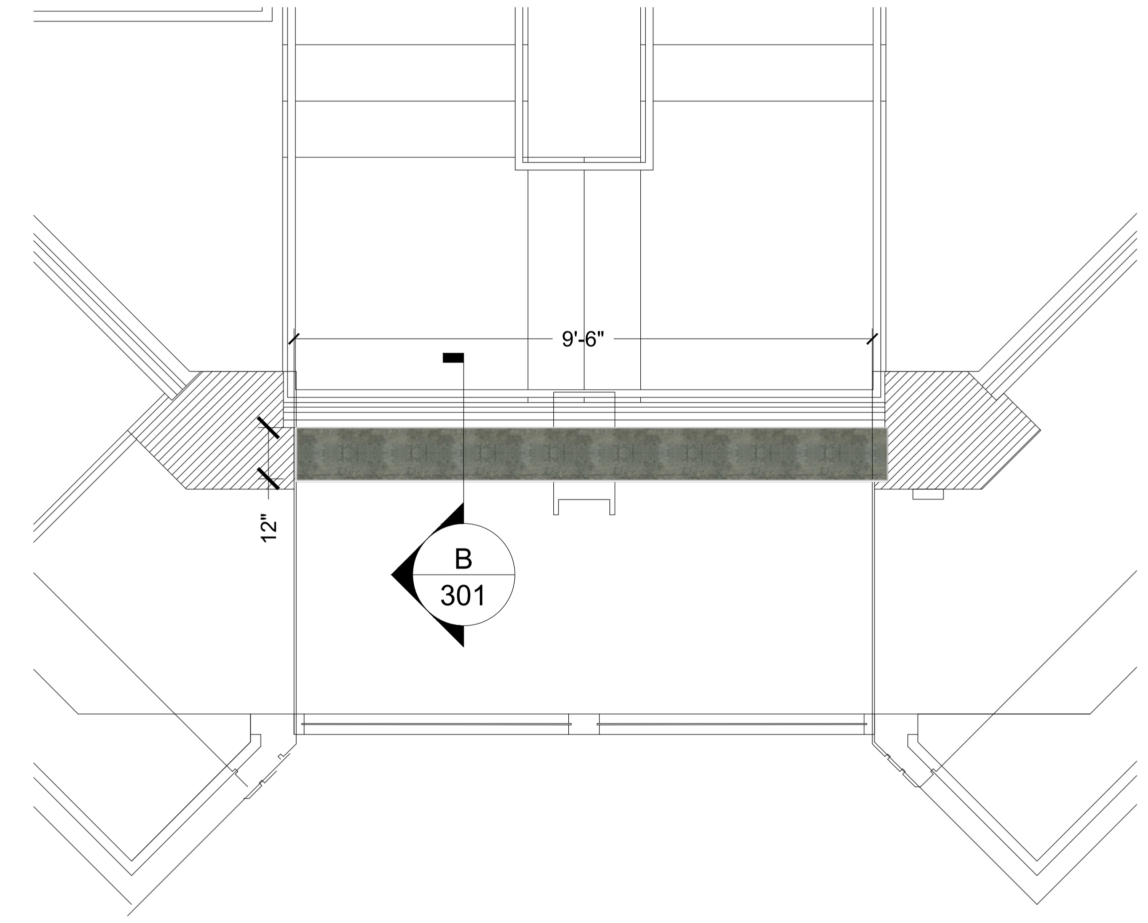
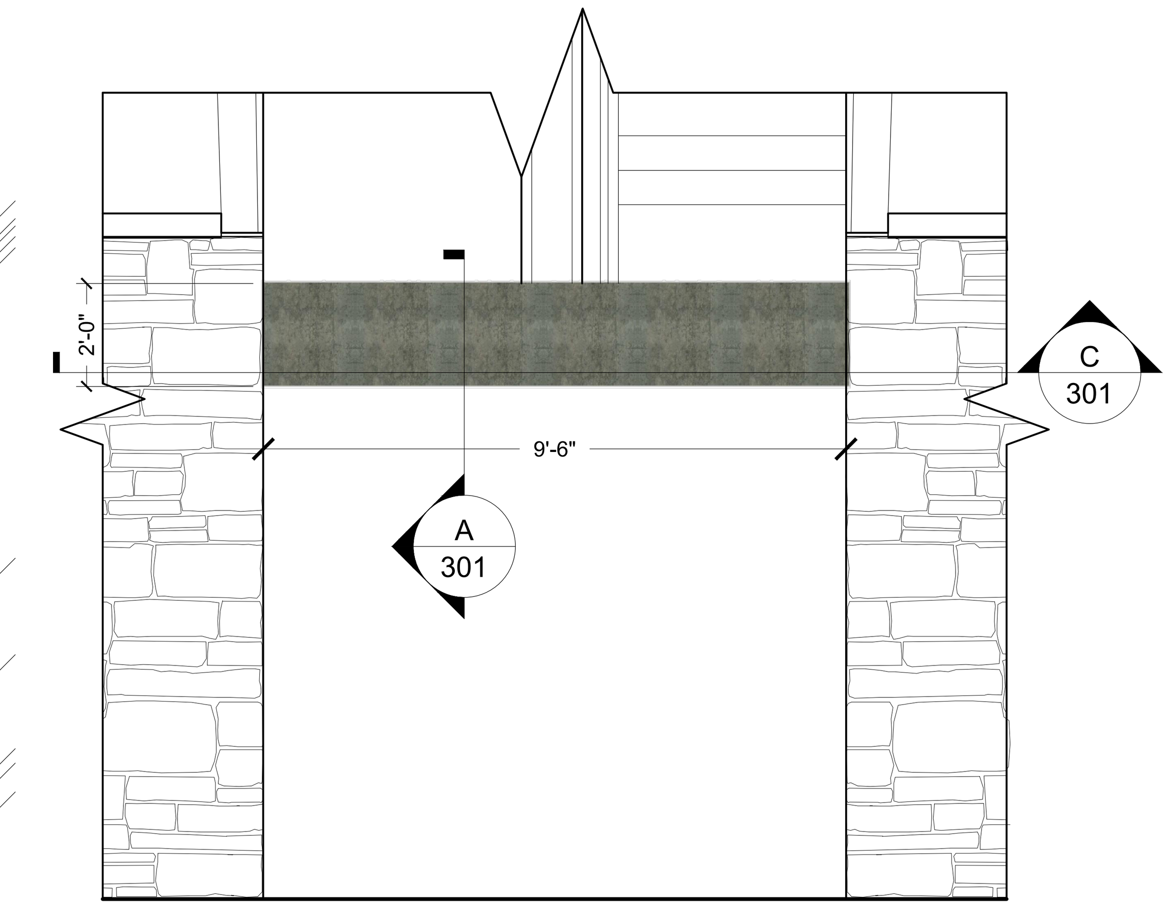


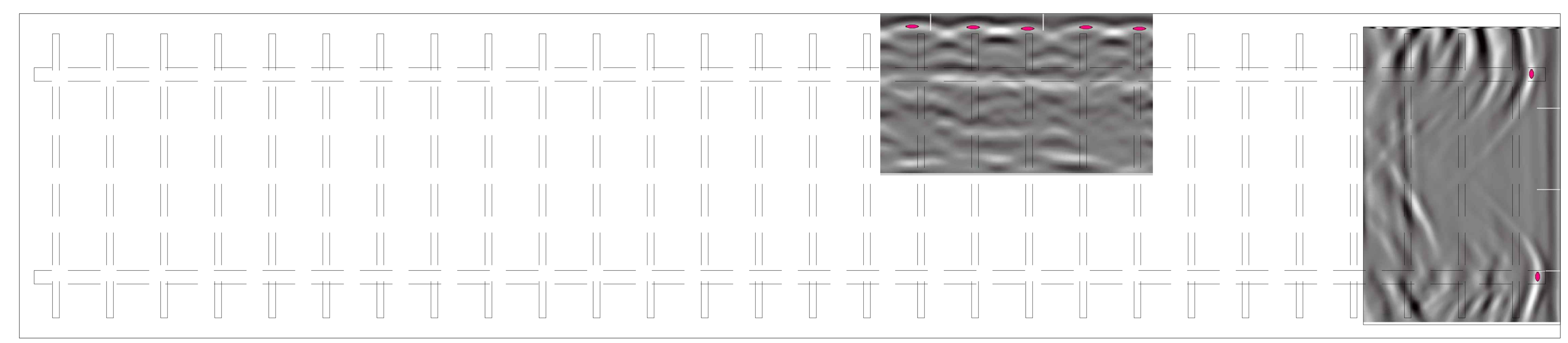
2 ISOMETRIC VIEW - SOUTH BEAM
NTS



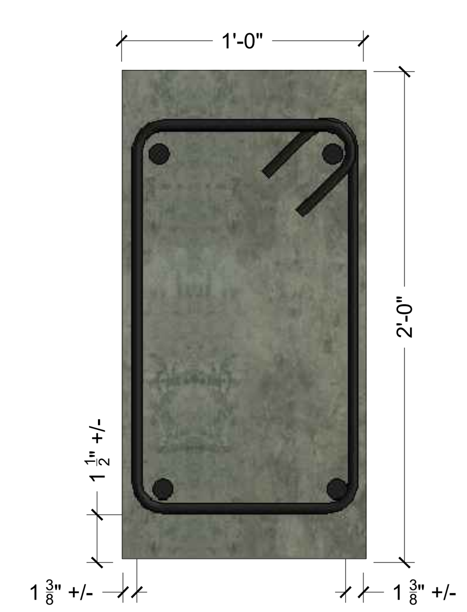
3 SOUTH BEAM - SPR SCAN LOCATION
SCALE: 3/8" = 1'-0"



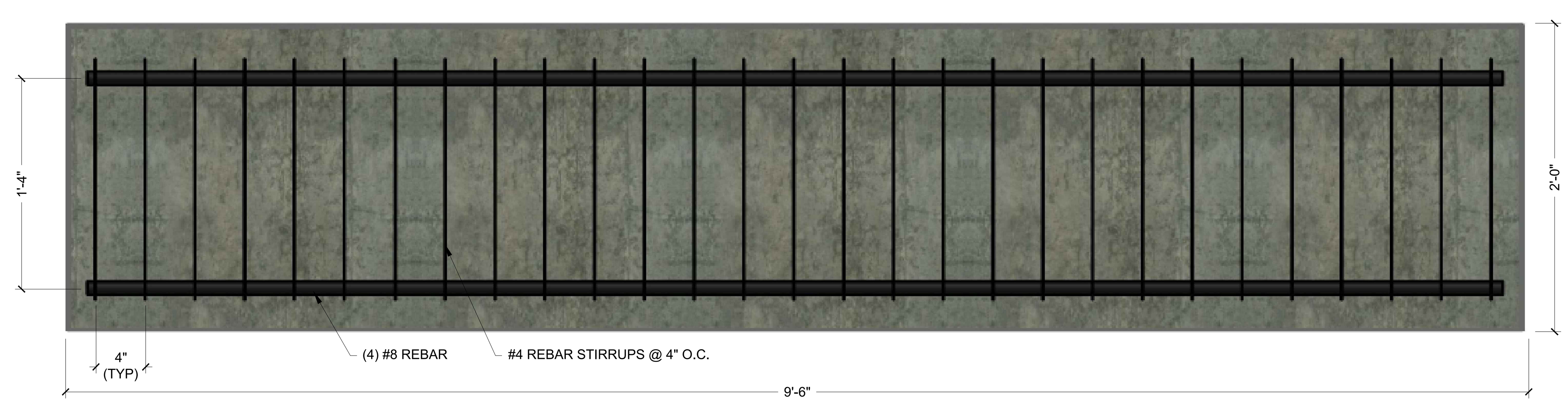
4 SOUTH ELEVATION - SPR SCAN LOCATION
SCALE: 3/8" = 1'-0"



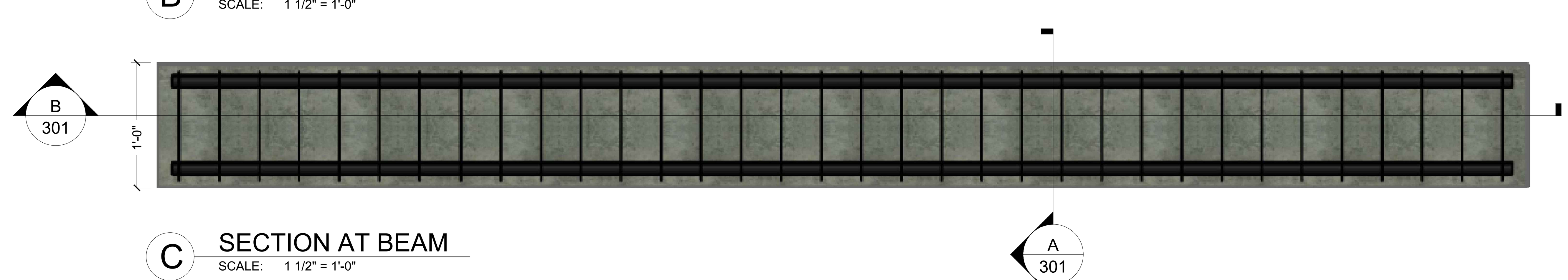
5 SOUTH BEAM - SPR SCANS
SCALE: 1 1/2" = 1'-0"



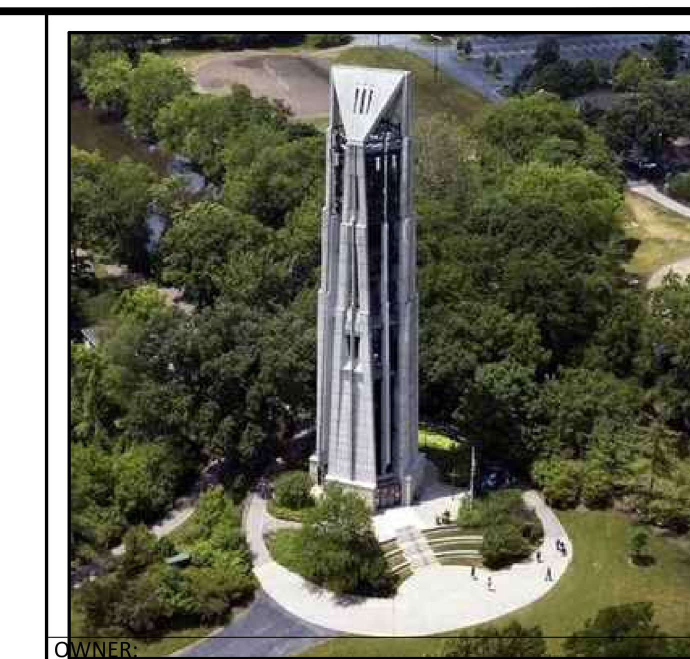
A SECTION AT BEAM
SCALE: 1 1/2" = 1'-0"



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



C SECTION AT BEAM
SCALE: 1 1/2" = 1'-0"



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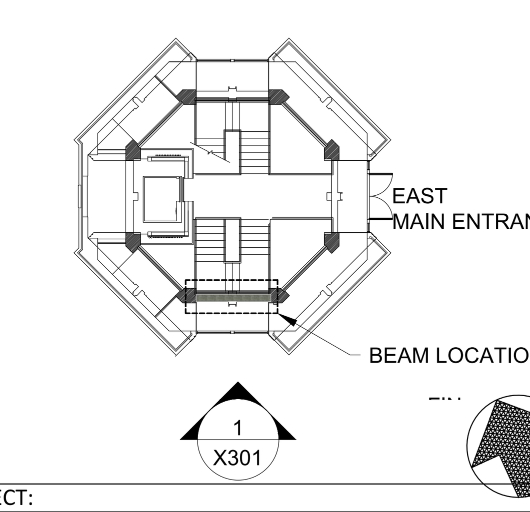


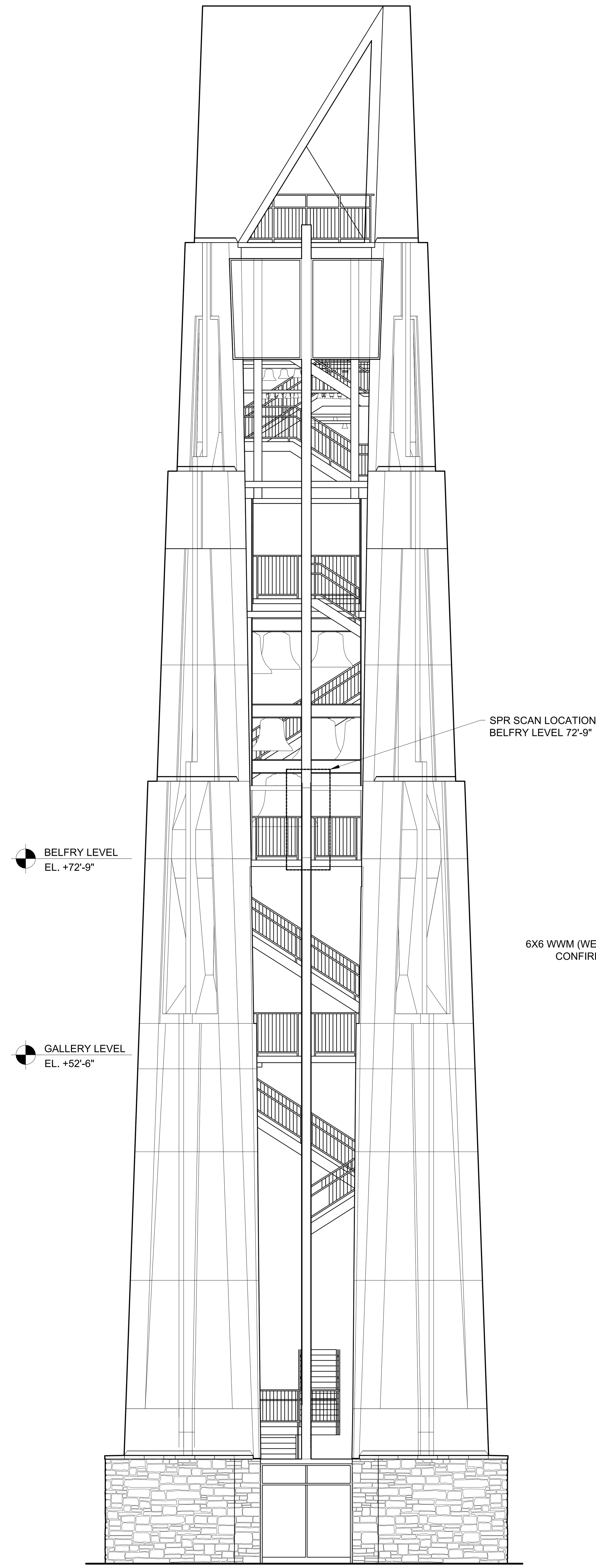
PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

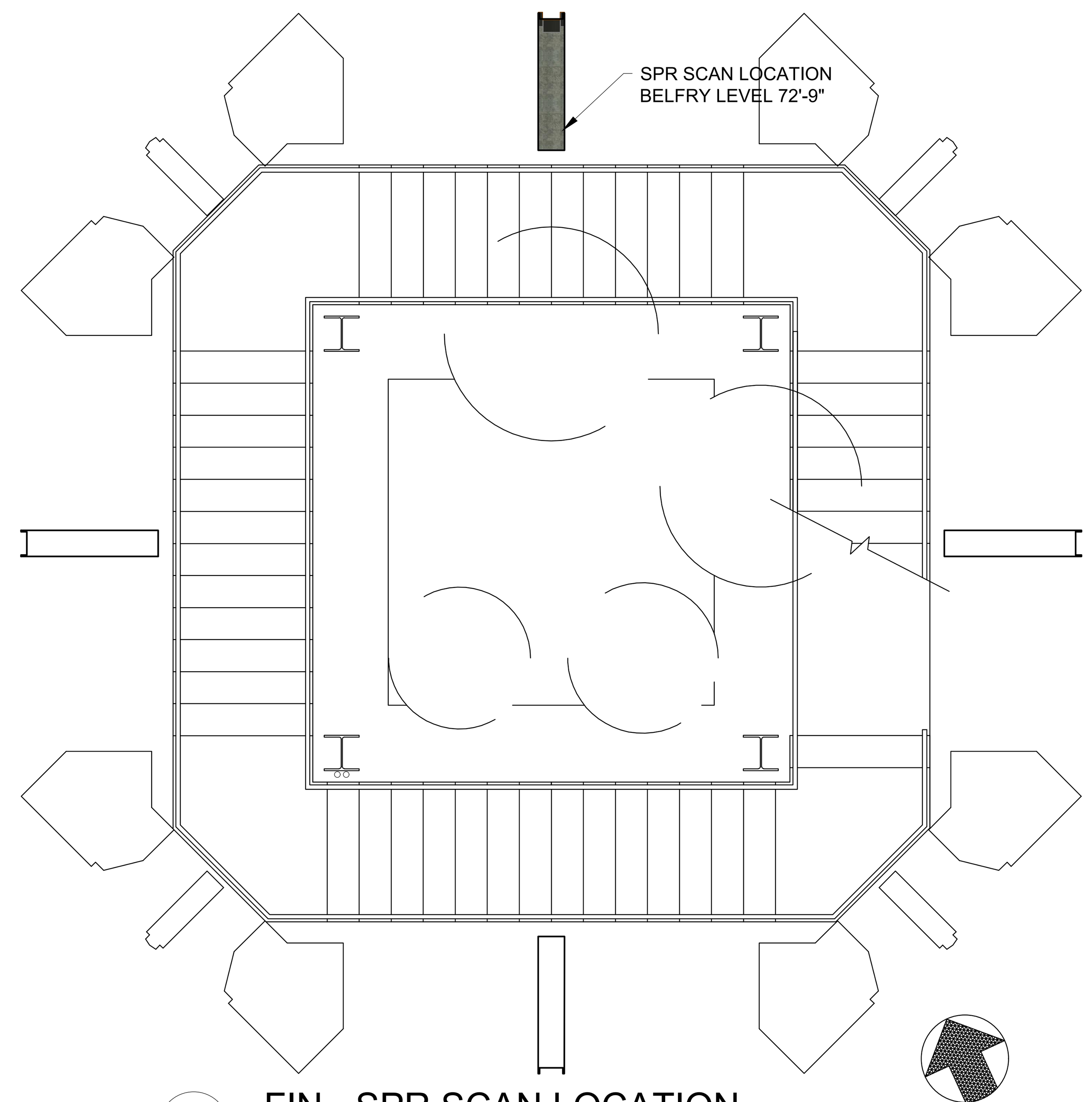
SHEET TITLE:
REINFORCEMENT LAYOUT
FIRST FLOOR BEAM

PROJECT REFERENCE NUMBER:
E17258

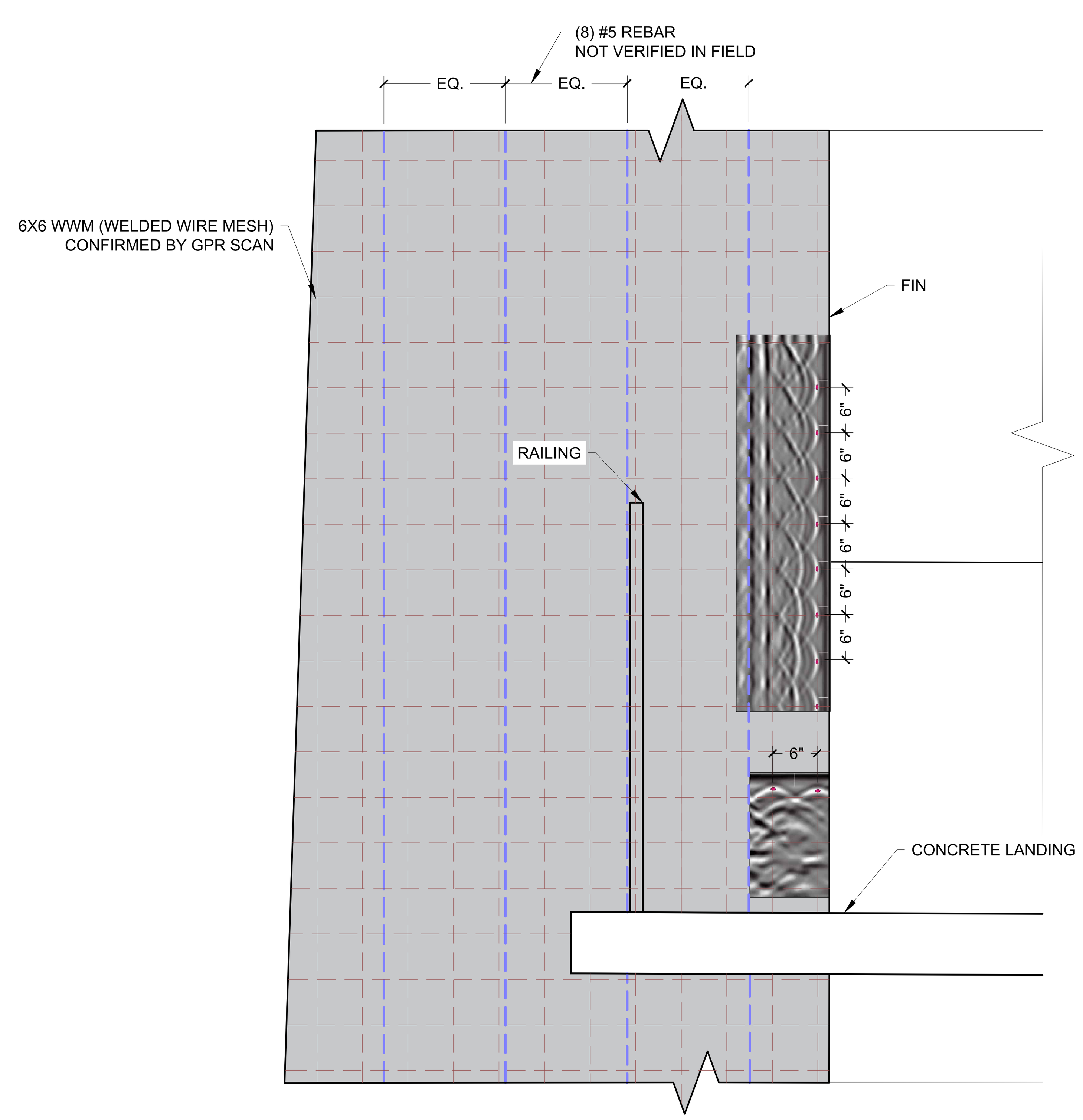
SHEET NUMBER: X-301
REVISION: 00



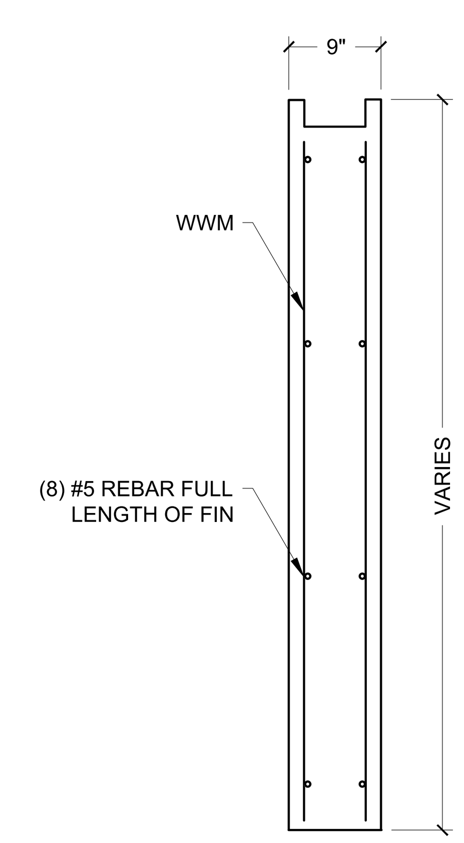
1 NORTH ELEVATION
SCALE: 1/8" = 1'-0"



2 FIN - SPR SCAN LOCATION
SCALE: 1/4" = 1'-0"

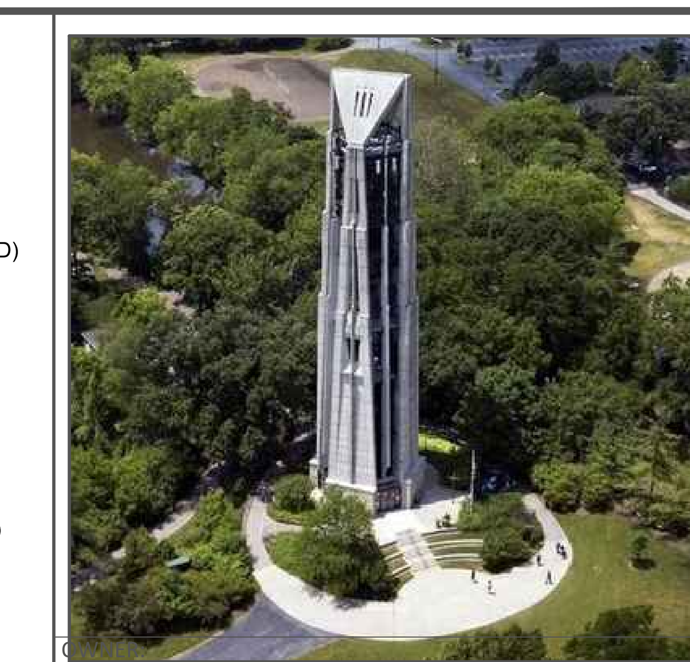


3 SECTION / ELEVATIONS AT FIN REINFORCEMENT LAYOUT
SCALE: 3/4" = 1'-0"



4 PLAN AT FIN REINFORCEMENT LAYOUT
SCALE: 3/4" = 1'-0"

5 ISOMETRIC VIEW OF FIN
NTS



CITY OF NAPERVILLE
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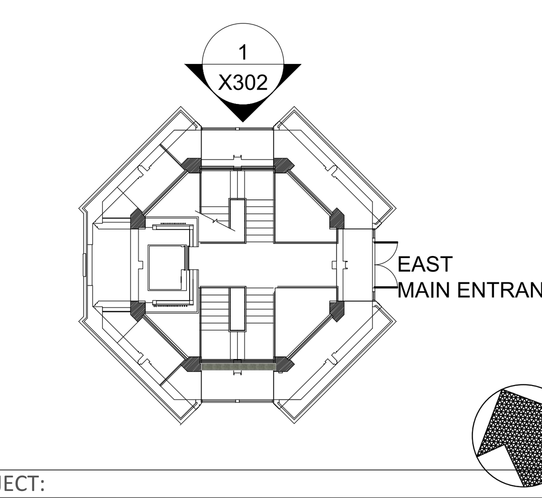
e2chem CONSULTANTS LLC

ISSUE:	DESCRIPTION:	DATE:	DRWN	CHK	APPR
	ISSUED TO CLIENT	08/24/18	NMF	IM	PN

DWG SCALE:
CAD DWG FILE: RE-E17258-NF-V1.0_Moser Tower.dwg
COPYRIGHT: ECHEM CONSULTANTS LLC
KEY MAP:



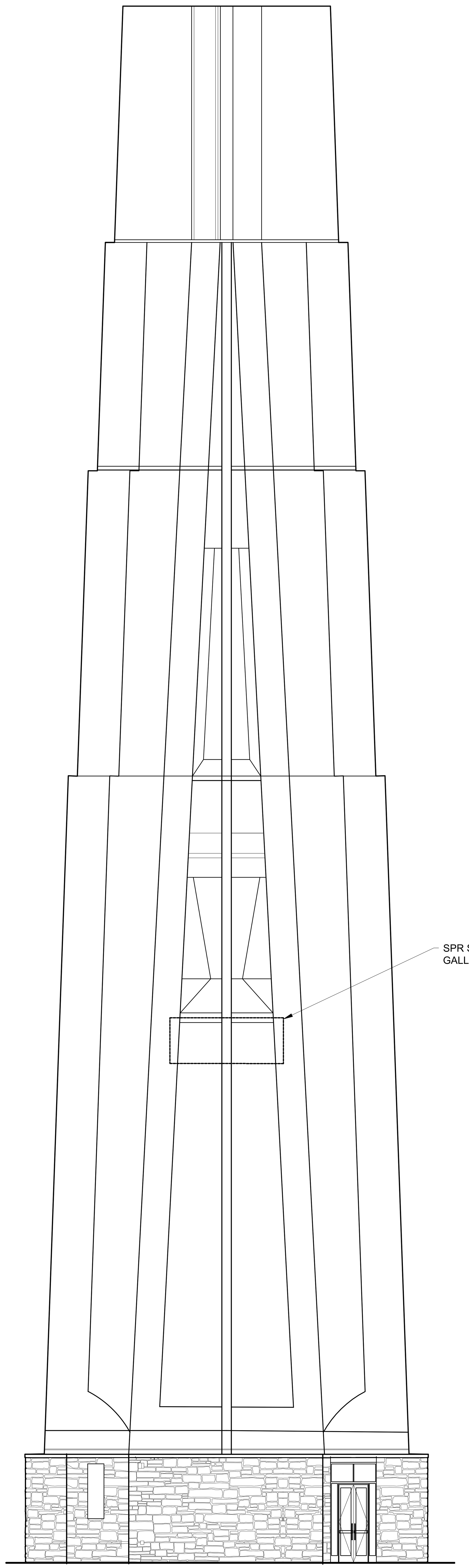
PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

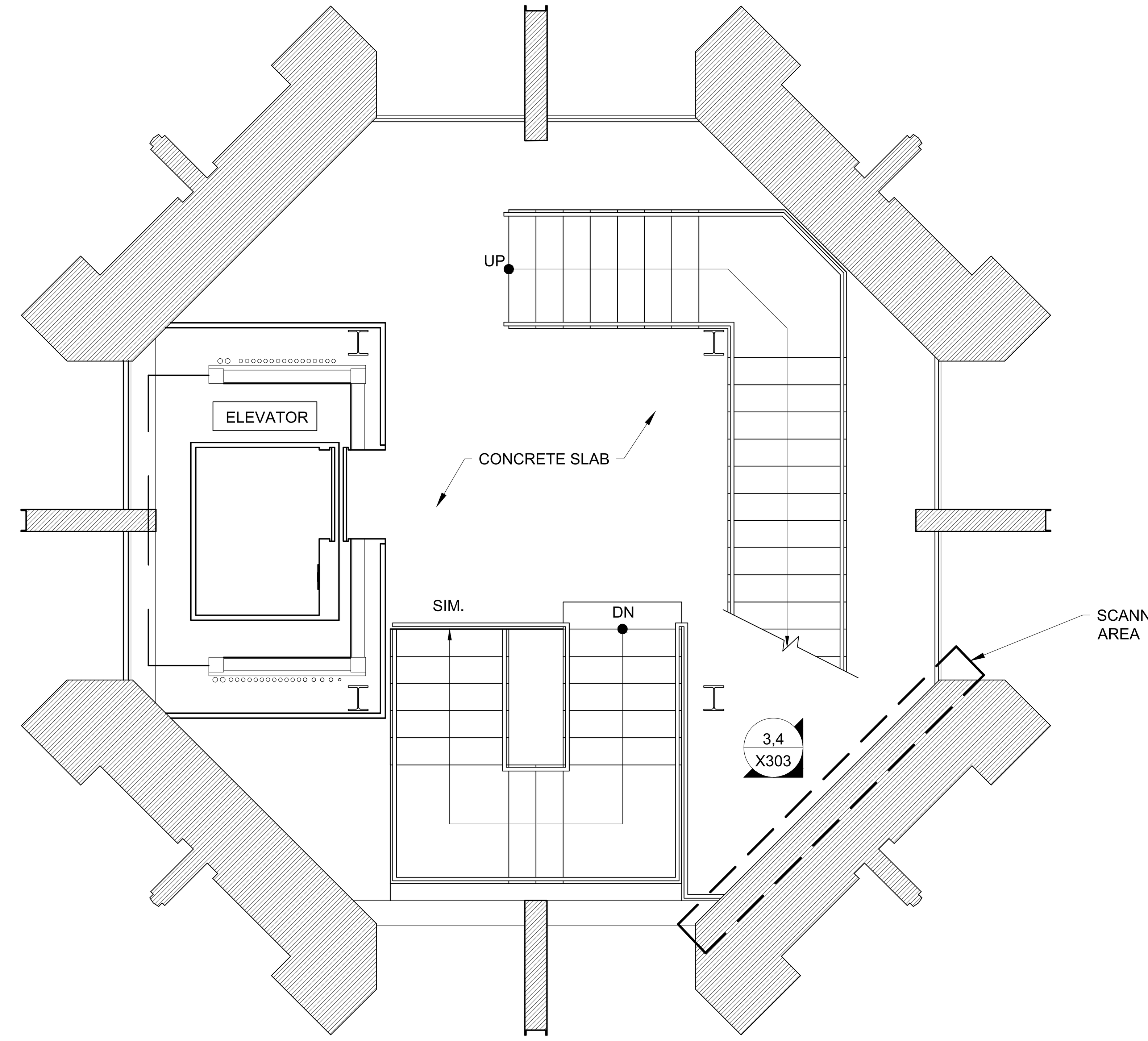
SHEET TITLE:
REINFORCEMENT
LAYOUT
PRE-CAST FIN PANEL

PROJECT REFERENCE NUMBER:
E17258

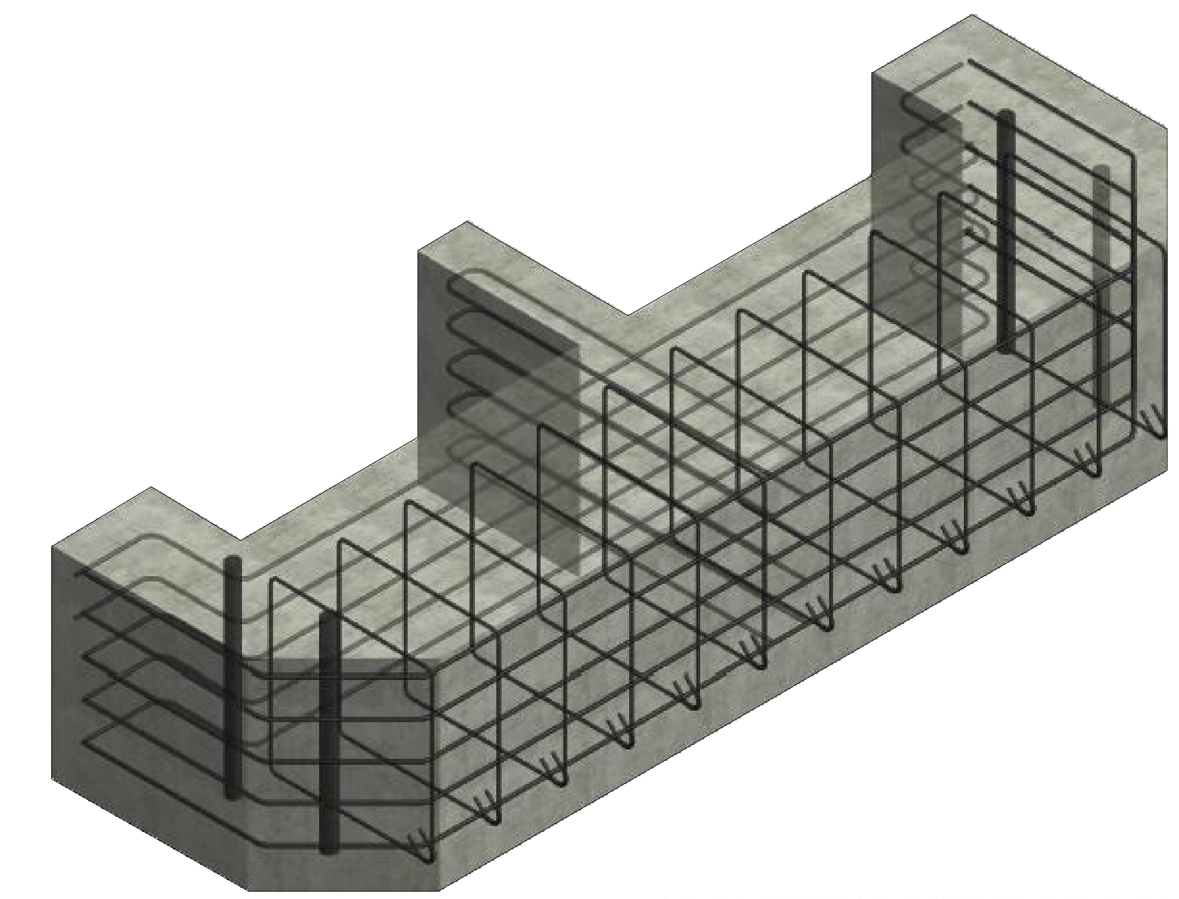
SHEET NUMBER: X-302
REVISION: 00



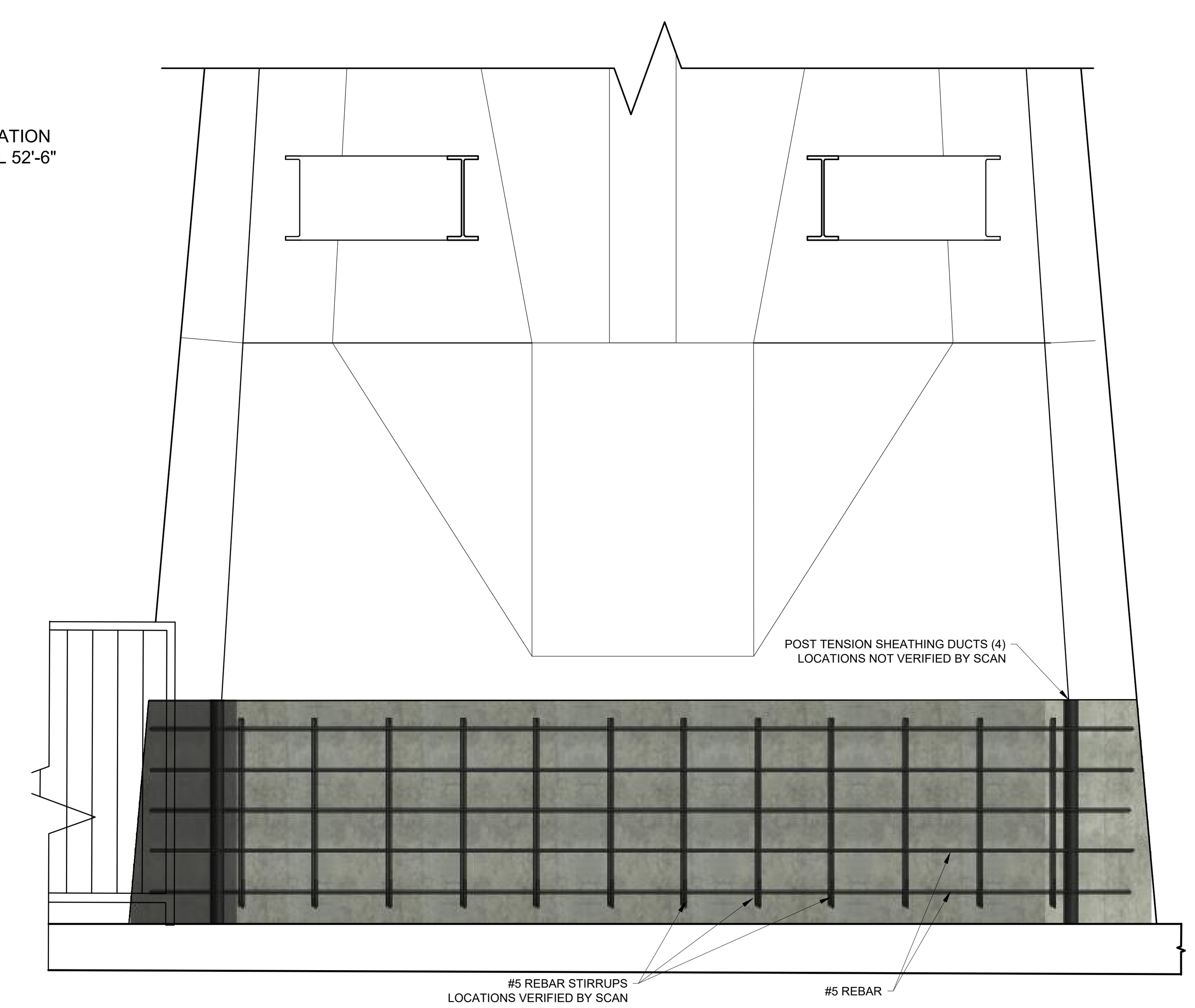
1 SOUTHWEST ELEVATION
SCALE: 1/8" = 1'-0"



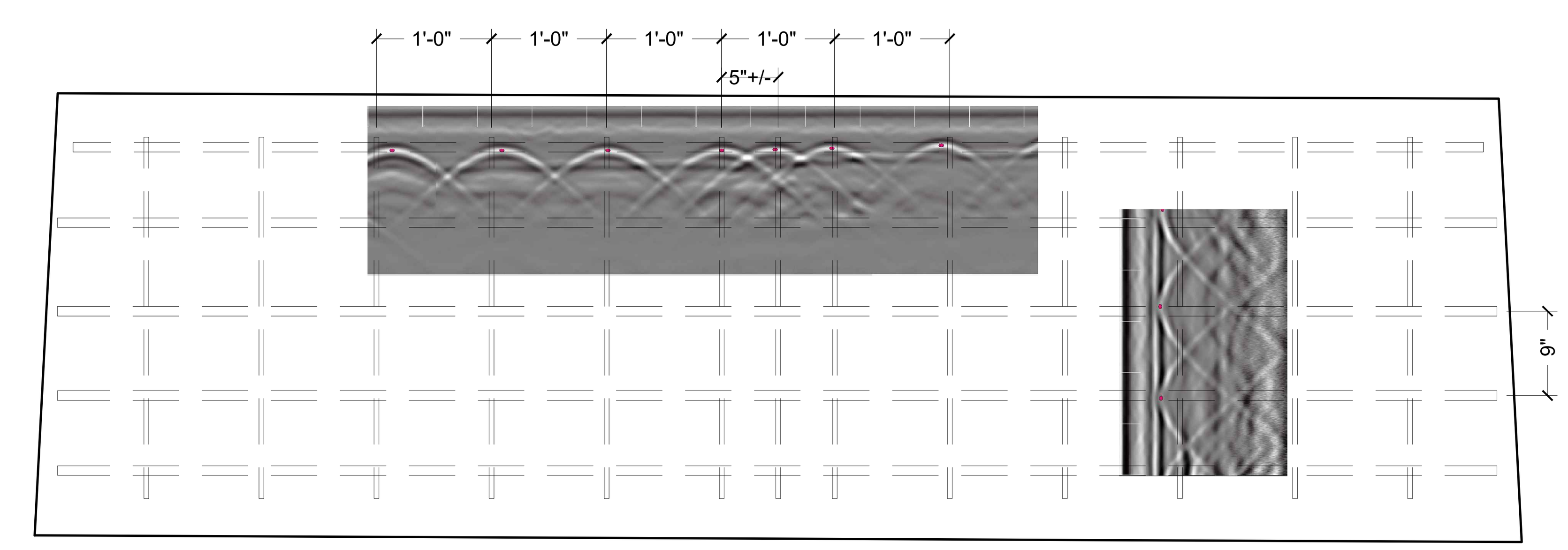
2 PLAN AT GALLERY T/ SLAB EL. +52'-6"
SCALE: 1/4" = 1'-0"



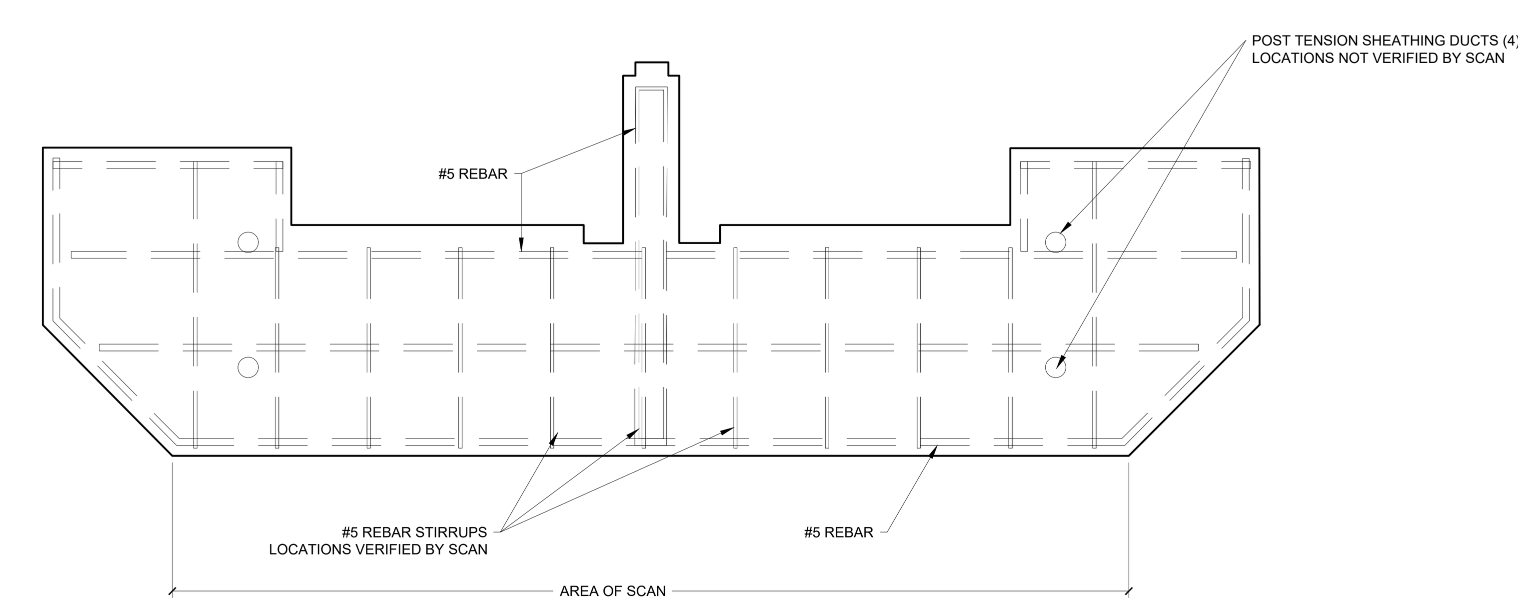
ISOMETRIC VIEW OF PANEL



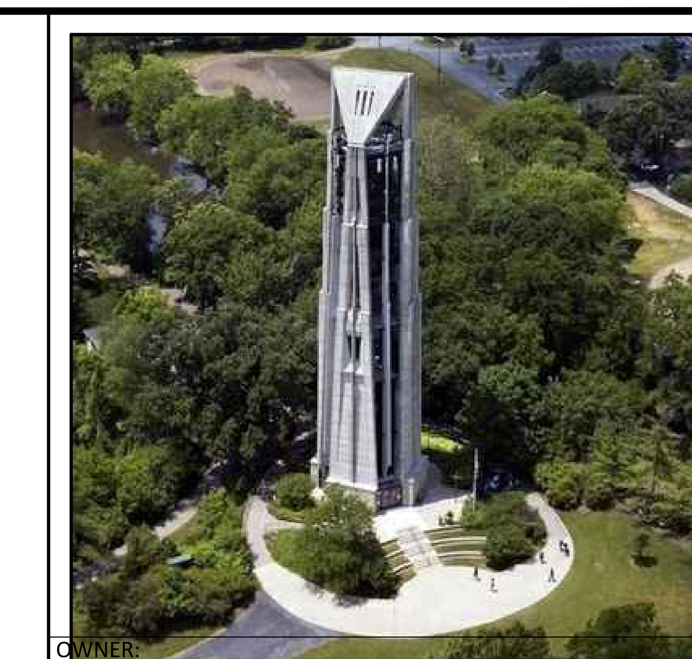
3 PANEL REINFORCEMENT LAYOUT
SCALE: 1/2" = 1'-0"



4 PANEL ELEVATION - SPR SCANS
SCALE: 3/4" = 1'-0"



5 PANEL PLAN - SPR SCANS
SCALE: 1/2" = 1'-0"



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DWG SCALE:
CAD DWG FILE: RE-E17258-NF-V1.0_Moser Tower.dwg
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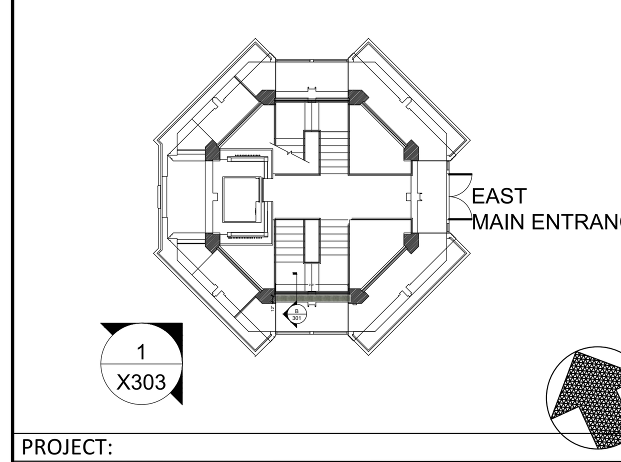
KEY MAP:



PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

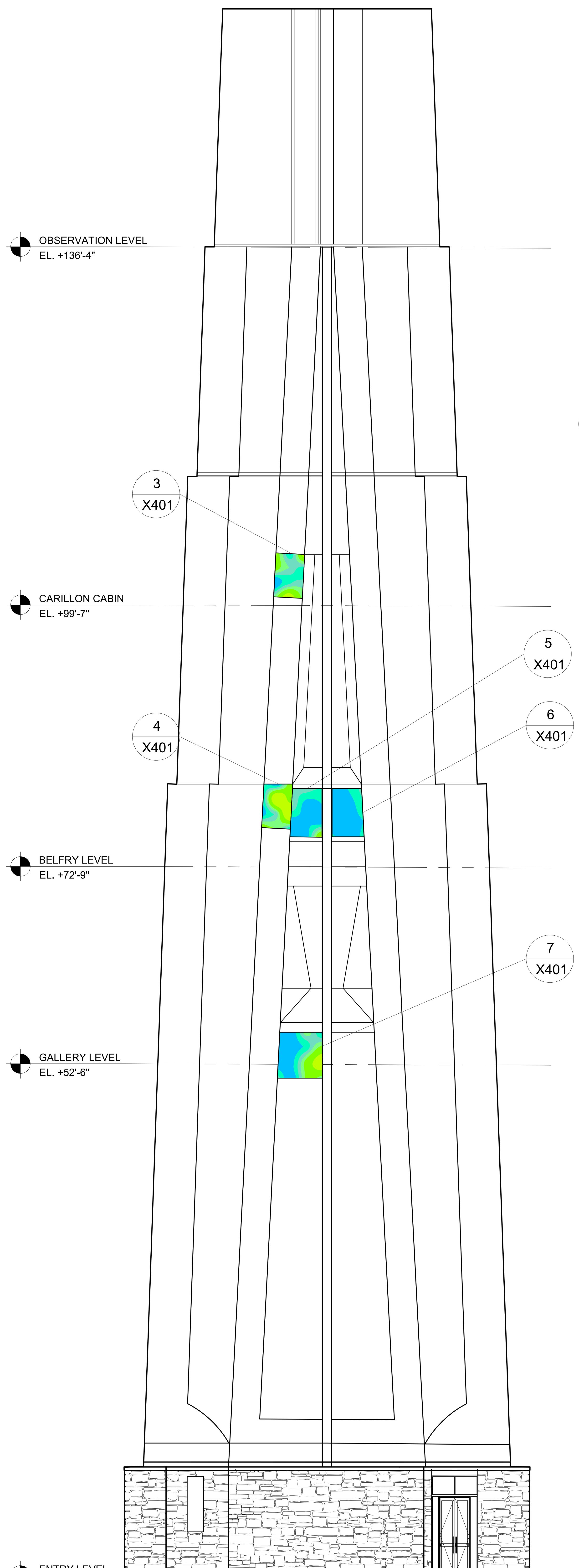
CORROSION ASSESSMENT

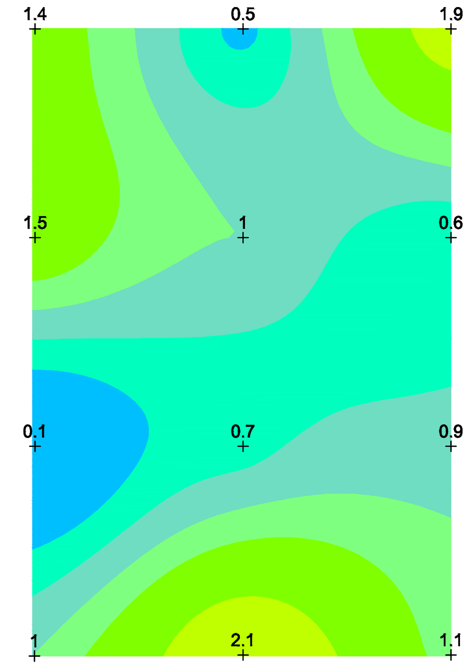
SHEET TITLE:
REINFORCEMENT LAYOUT AT PANEL

PROJECT REFERENCE NUMBER:
E17258

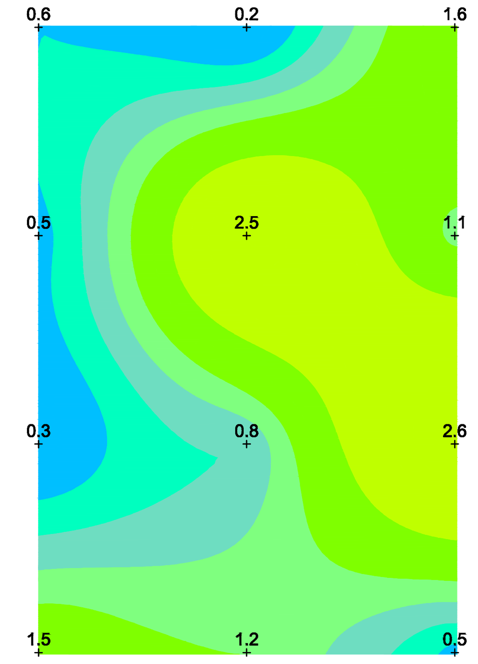
SHEET NUMBER: X-303

REVISION: 00

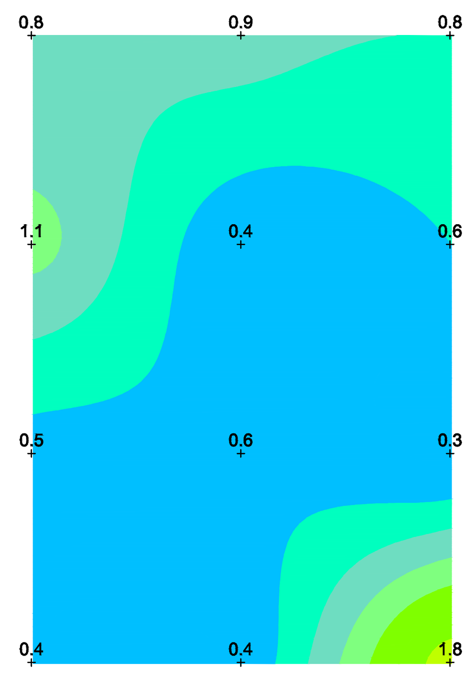




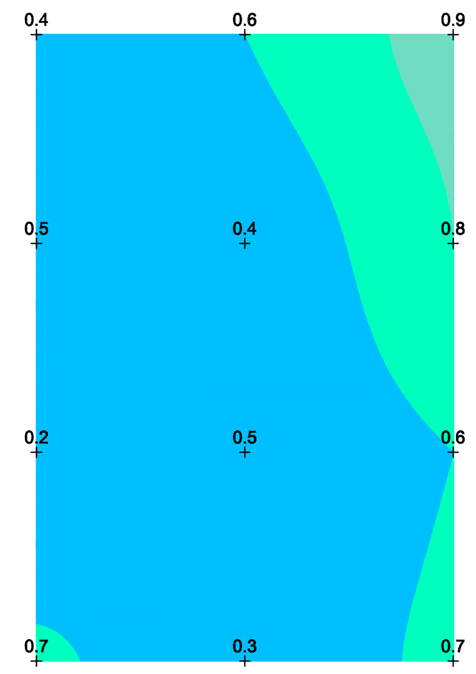
LOCATION 1
CORROSION RATE (i-COS)
CONTOUR MAP
3
NTS



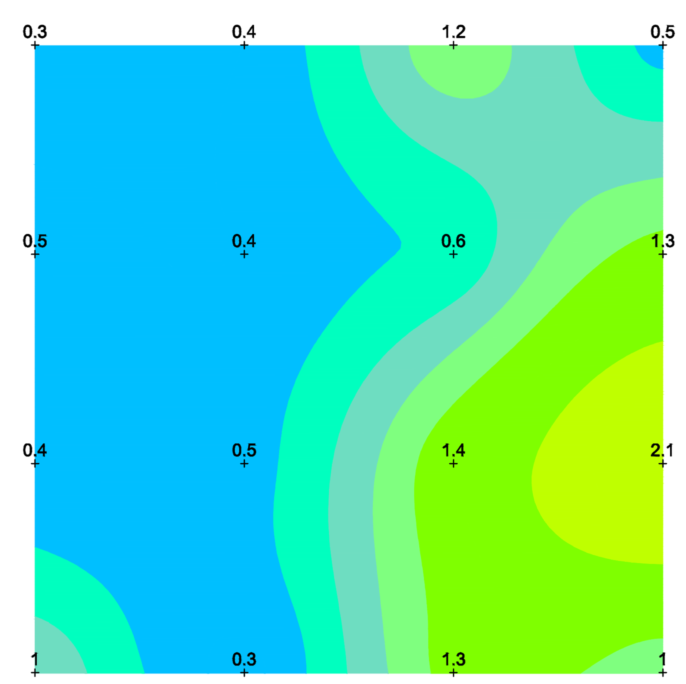
LOCATION 2
CORROSION RATE (i-COS)
CONTOUR MAP
4
NTS



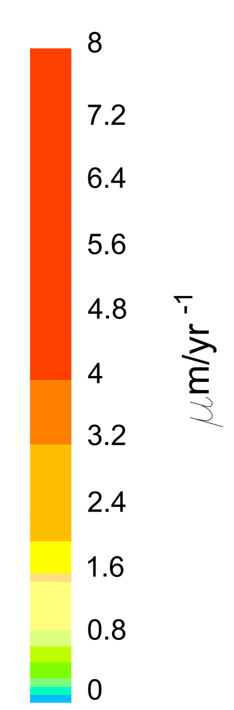
LOCATION 3
CORROSION RATE (i-COS)
CONTOUR MAP
5
NTS



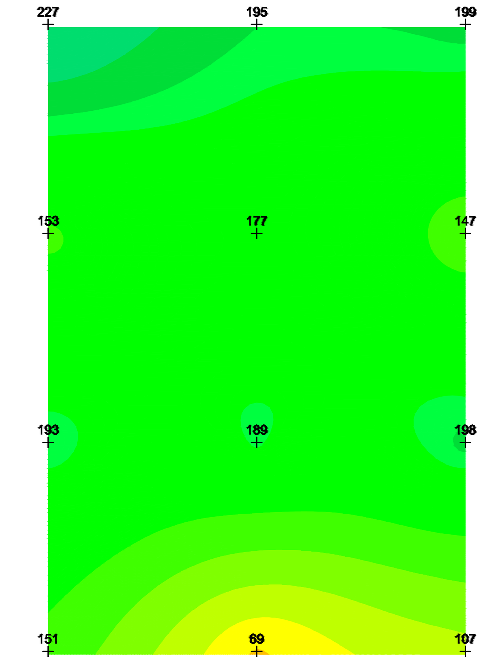
LOCATION 4
CORROSION RATE (i-COS)
CONTOUR MAP
6
NTS



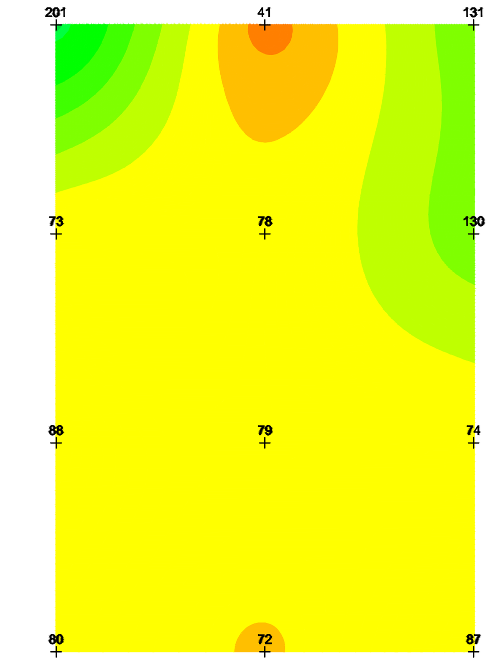
LOCATION 5
CORROSION RATE (i-COS)
CONTOUR MAP
7
NTS



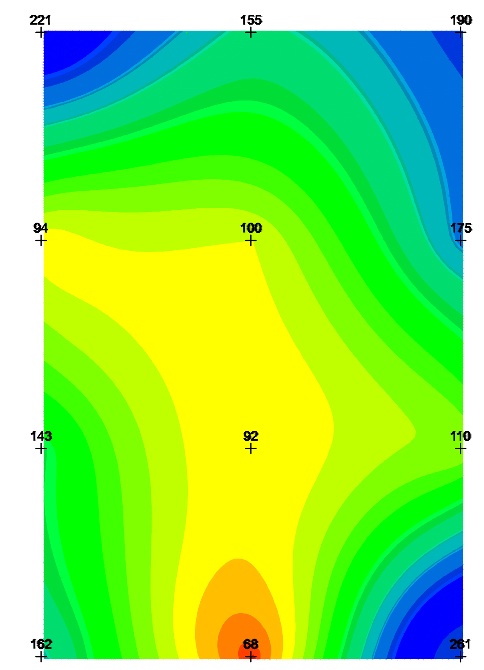
1 SOUTHWEST ELEVATION - CORROSION RATES (i-COS)
SCALE: 1/8" = 1'-0"



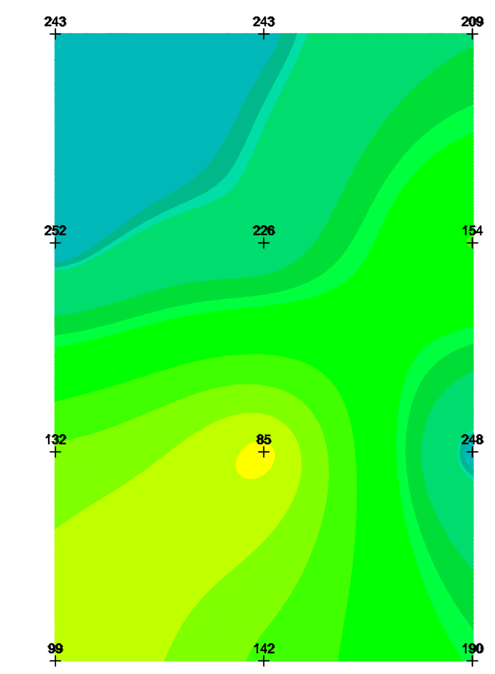
LOCATION 1
RESISTIVITY RATE (i-ROM)
CONTOUR MAP
8
NTS



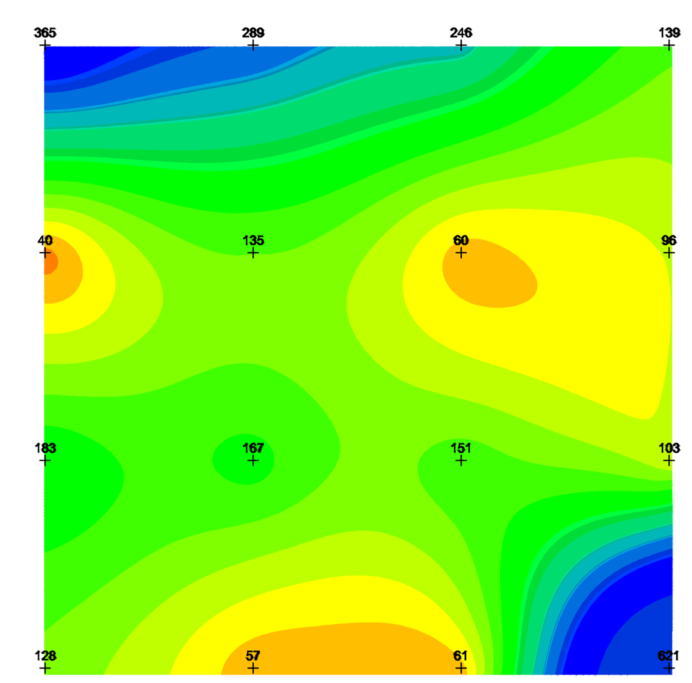
LOCATION 2
RESISTIVITY RATE (i-ROM)
CONTOUR MAP
9
NTS



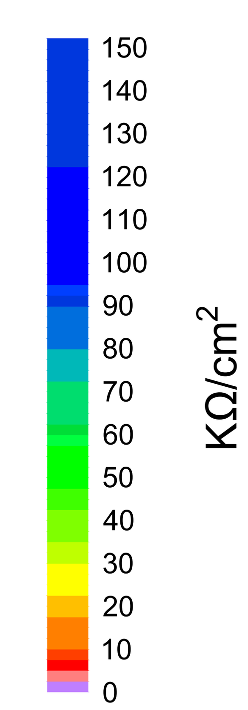
LOCATION 3
RESISTIVITY RATE (i-ROM)
CONTOUR MAP
10
NTS

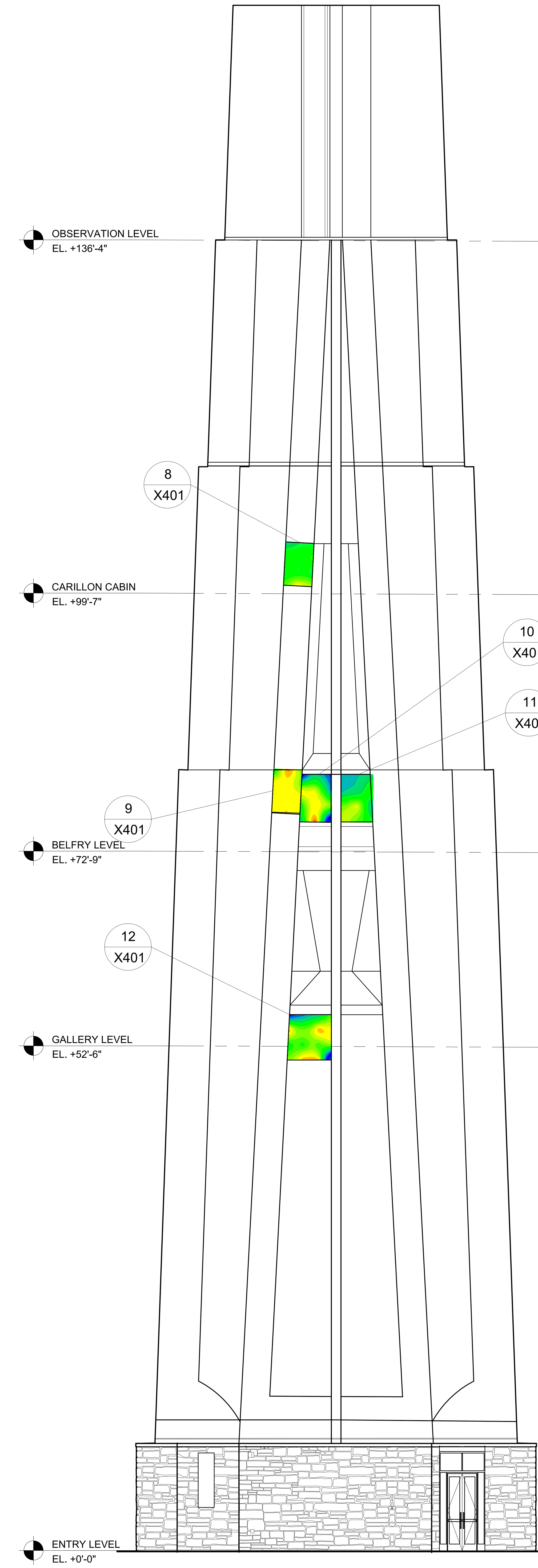


LOCATION 4
RESISTIVITY RATE (i-ROM)
CONTOUR MAP
11
NTS

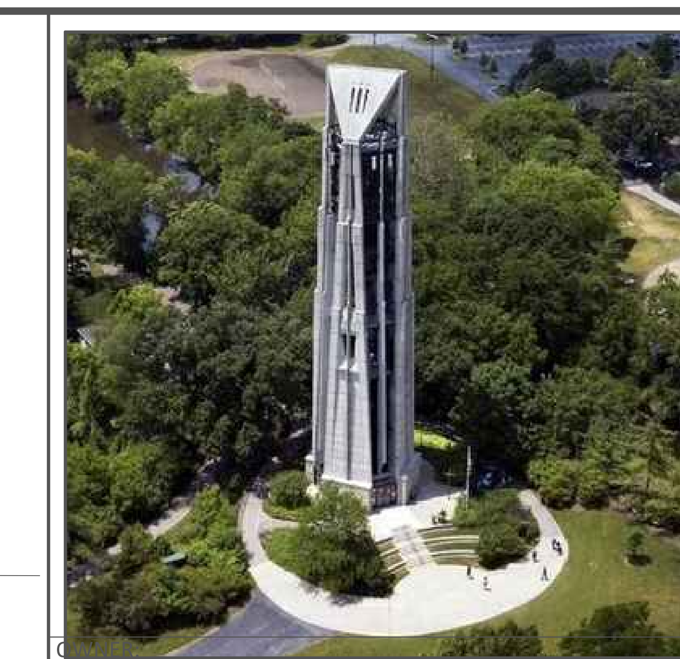


LOCATION 5
RESISTIVITY RATE (i-ROM)
CONTOUR MAP
12
NTS





2 SOUTHWEST ELEVATION - RESISTIVITY (i-ROM)
SCALE: 1/8" = 1'-0"



CITY OF NAPERVILLE
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DWG SCALE:
CAD DWG FILE: RE-E17258-NF-V1.0_Moser Tower.dwg
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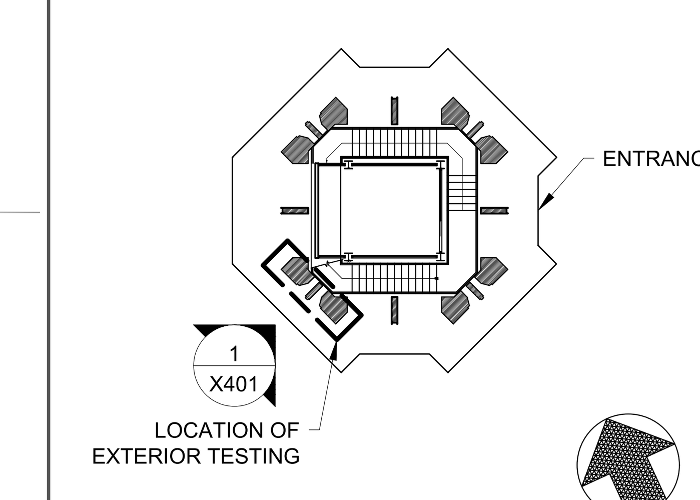
KEY MAP:



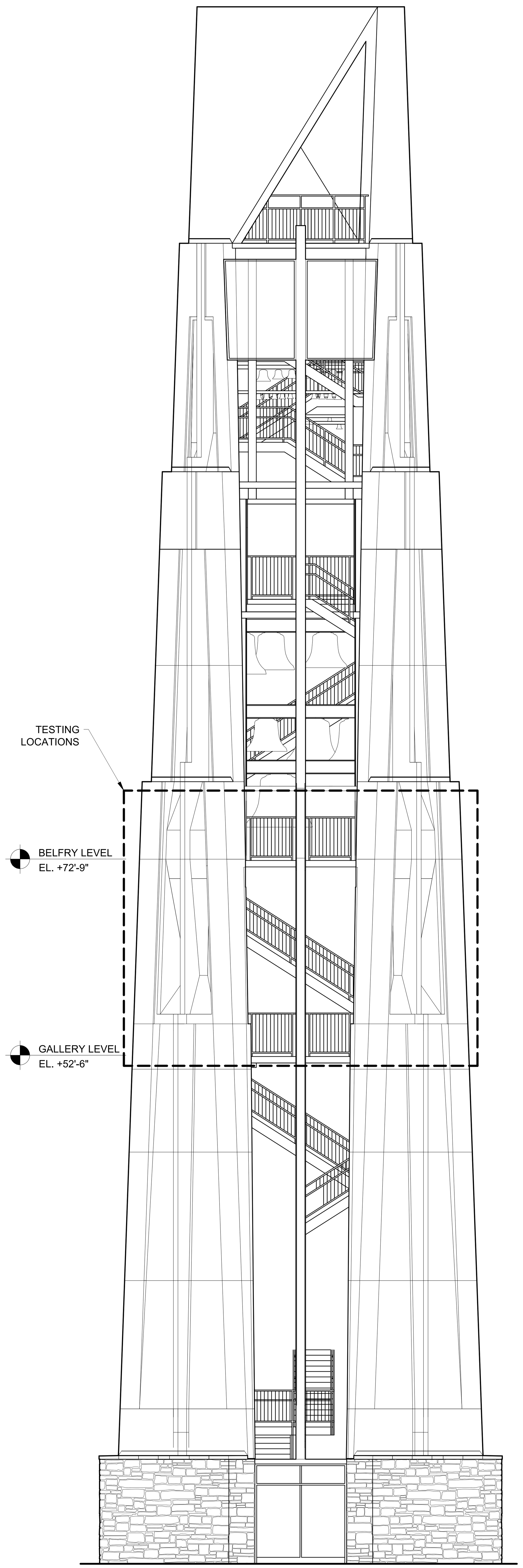
PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

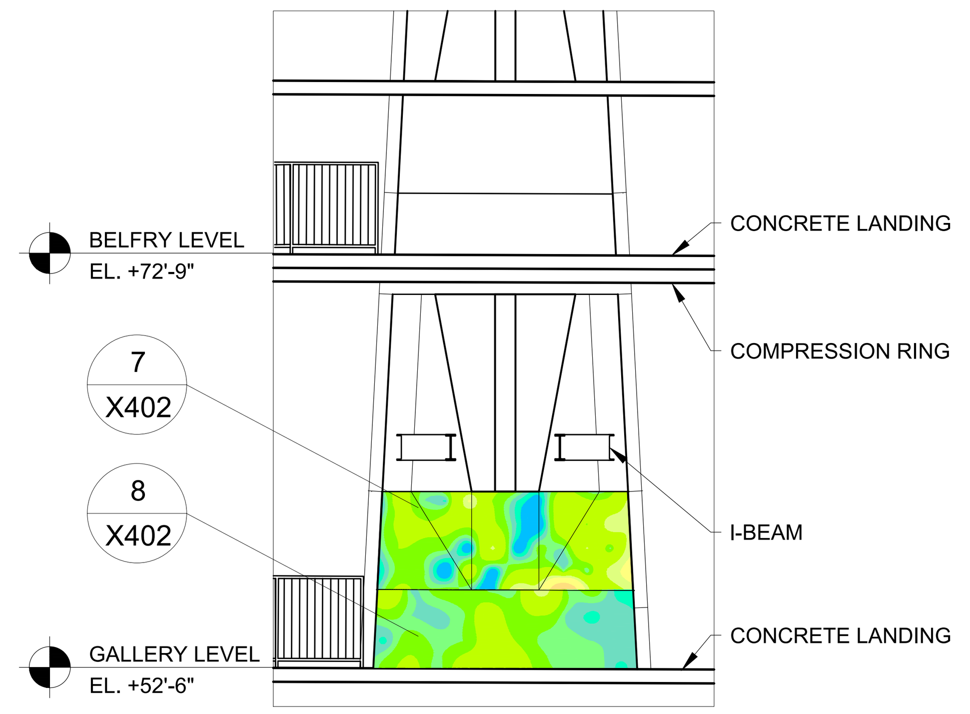
SHEET TITLE:
SOUTHWEST PRE-CAST CORROSION RATE (i-COS) AND RESISTIVITY (i-ROM) CONTOUR MAPS

PROJECT REFERENCE NUMBER:
E17258
SHEET NUMBER: X-401
REVISION: 00

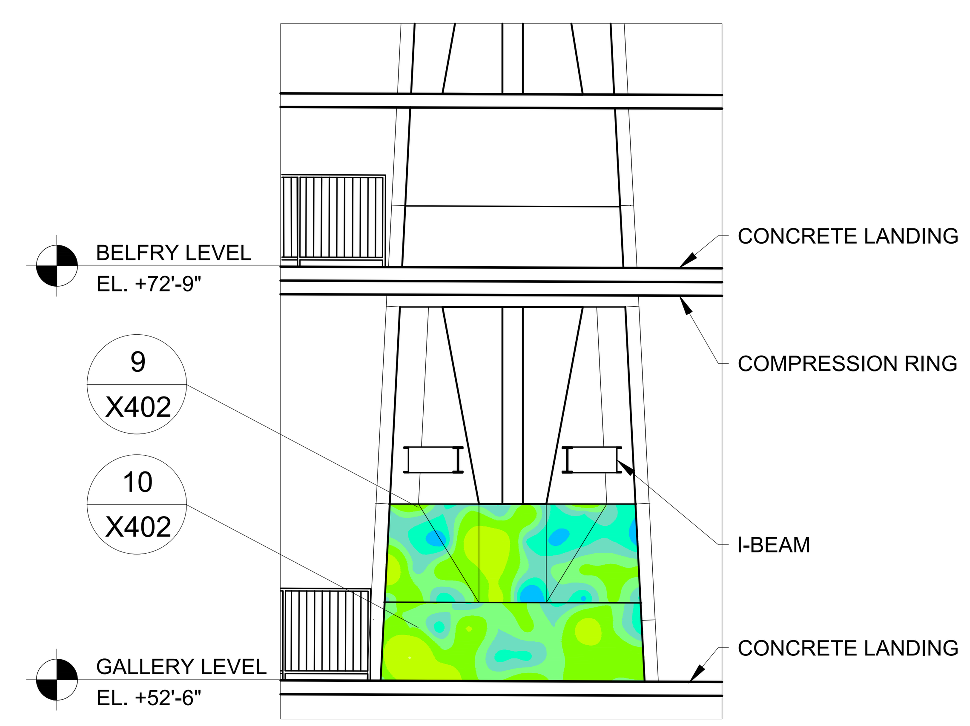
ANSI D SIZE DRAWING 22 X 34



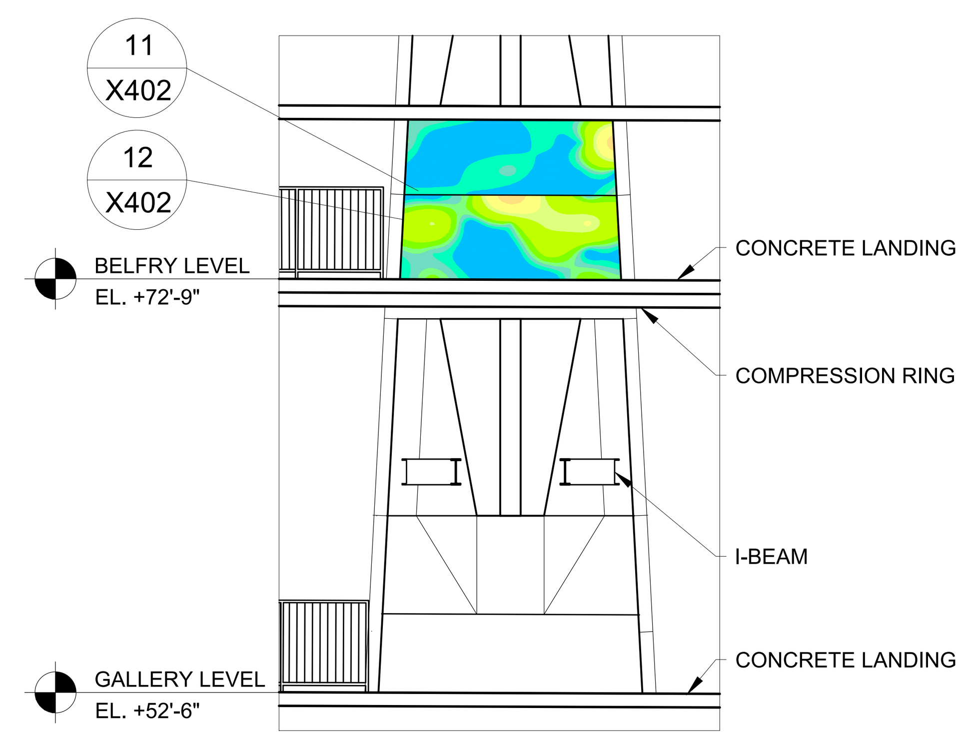
1 TESTING LOCATIONS - ELEVATION
SCALE: 1/8" = 1'-0"



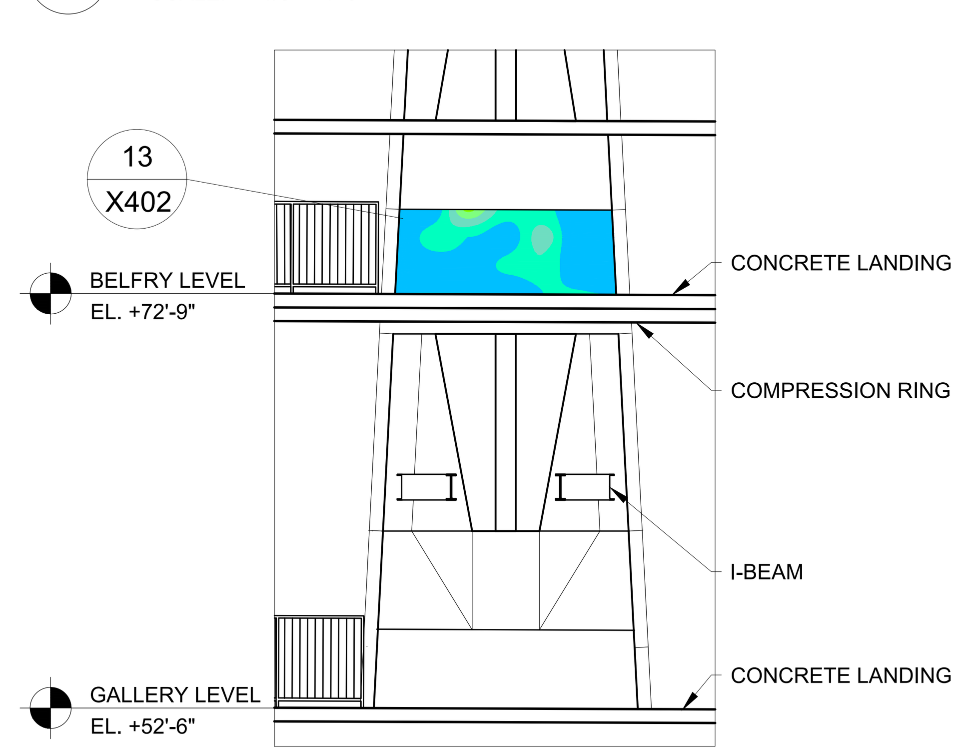
3 NORTHWEST
SCALE: 1/8" = 1'-0"



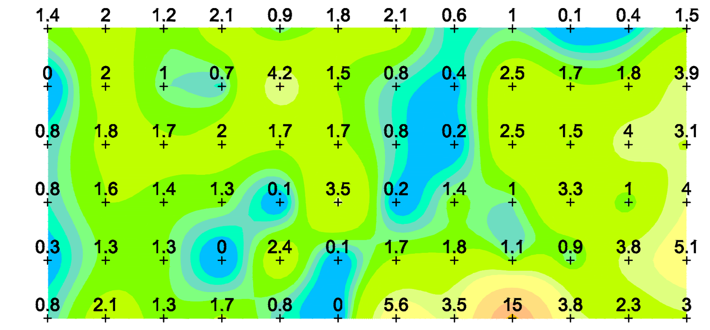
4 SOUTHEAST
SCALE: 1/8" = 1'-0"



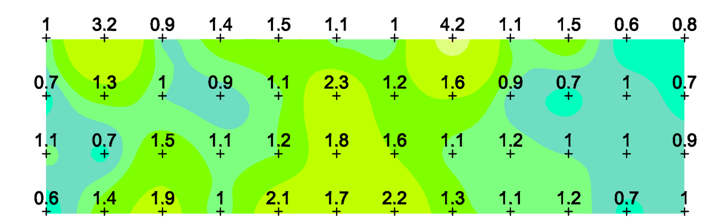
5 NORTHEAST
SCALE: 1/8" = 1'-0"



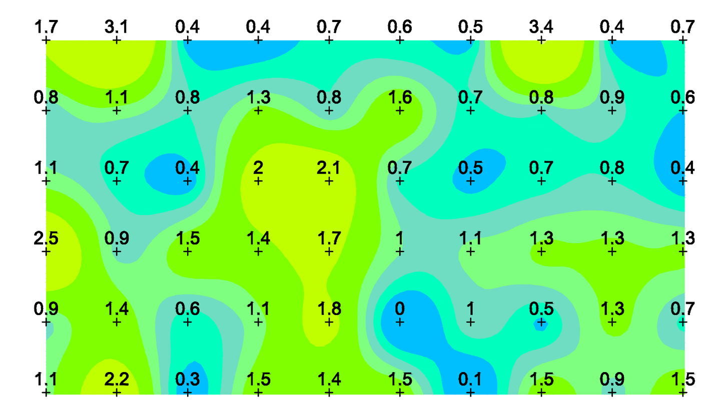
6 SOUTHWEST
SCALE: 1/8" = 1'-0"



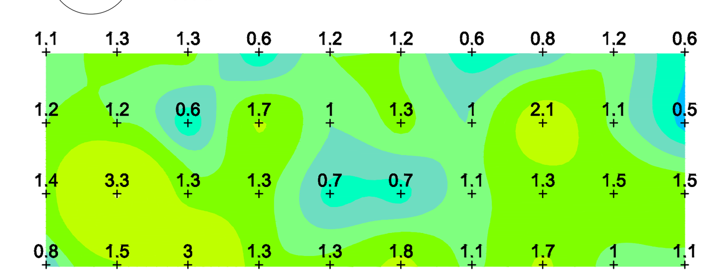
7 LOCATION 6
CORROSION RATE (i-COS) CONTOUR MAP
NTS



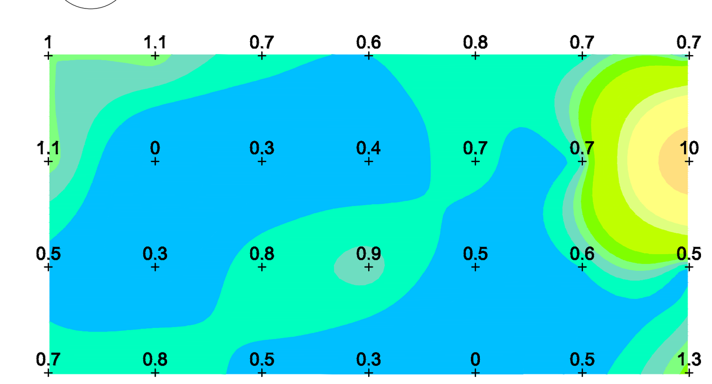
8 LOCATION 7
CORROSION RATE (i-COS) CONTOUR MAP
NTS



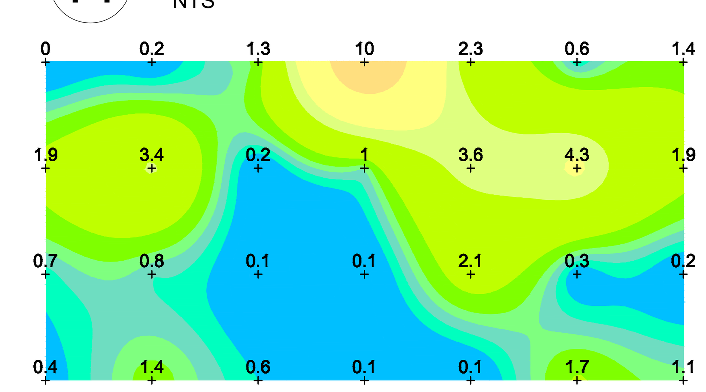
9 LOCATION 8
CORROSION RATE (i-COS) CONTOUR MAP
NTS



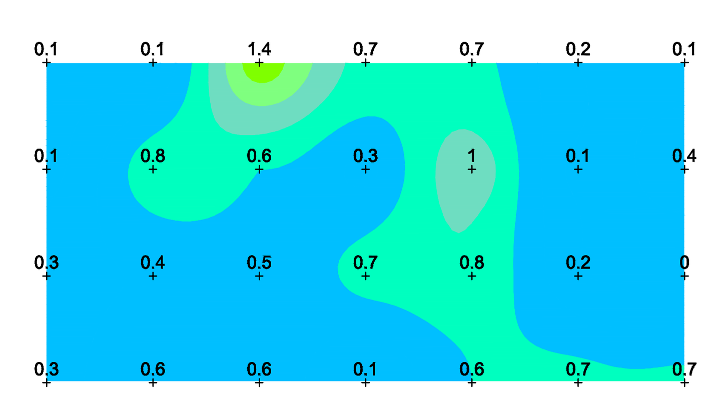
10 LOCATION 9
CORROSION RATE (i-COS) CONTOUR MAP
NTS



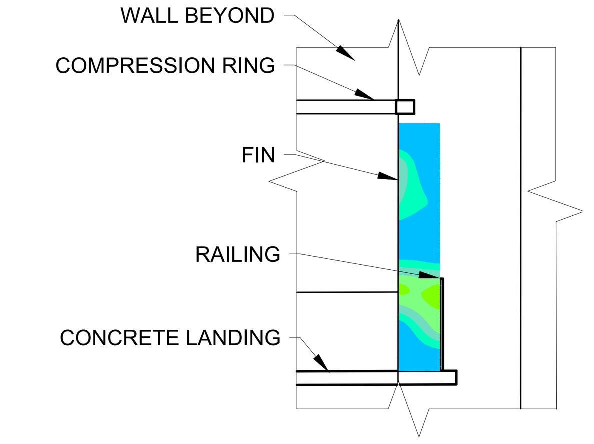
11 LOCATION 10
CORROSION RATE (i-COS) CONTOUR MAP
NTS



12 LOCATION 11
CORROSION RATE (i-COS) CONTOUR MAP
NTS



13 LOCATION 12
CORROSION RATE (i-COS) CONTOUR MAP
NTS



14 TESTING AT FIN
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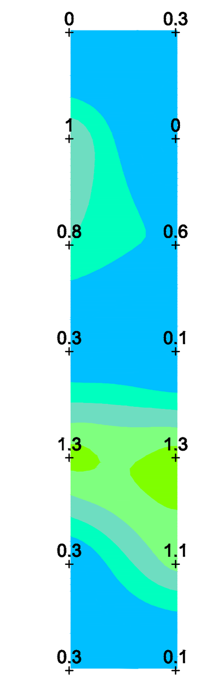
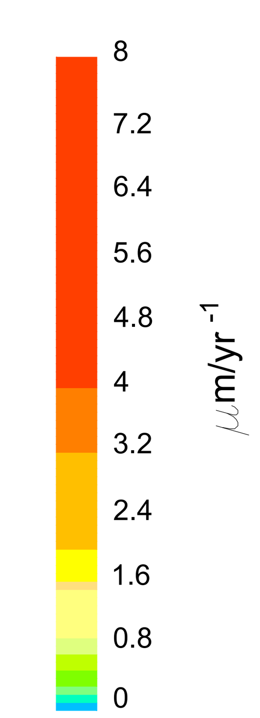
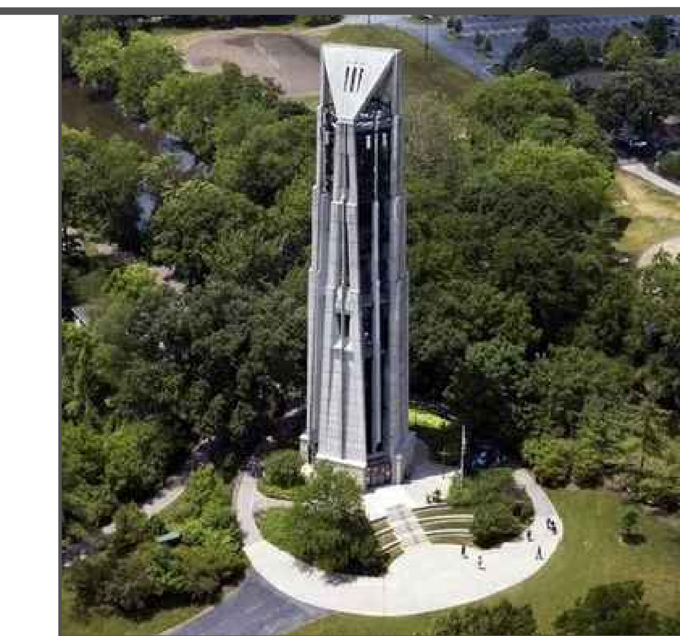


15 CORROSION RATE (i-COS) CONTOUR MAP
NTS





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DWG SCALE:
CAD DWG FILE: RE-E17258-NF-V1.0_Moser Tower.dwg
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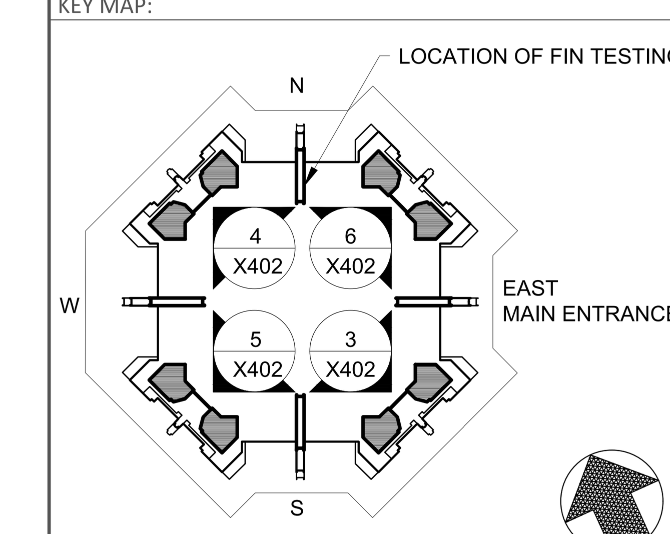
KEY MAP:



PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

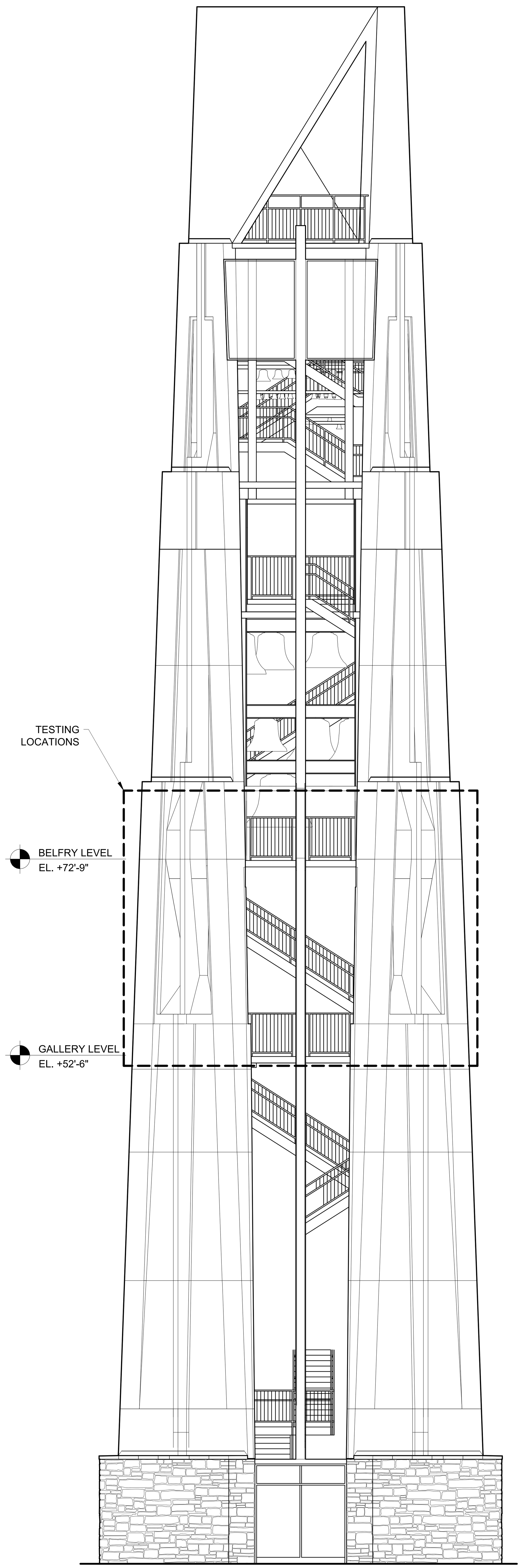
SHEET TITLE:
INTERIOR PRE-CAST PANELS
CORROSION RATE (i-COS) CONTOUR MAPS

PROJECT REFERENCE NUMBER:
E17258

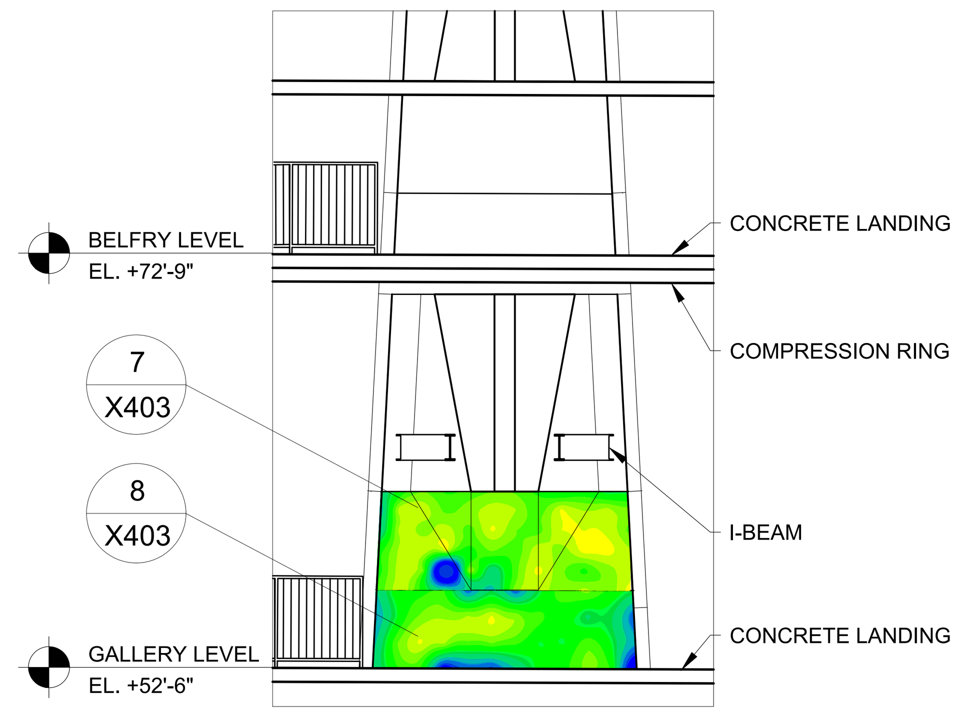
SHEET NUMBER:
X-402

REVISION:
00

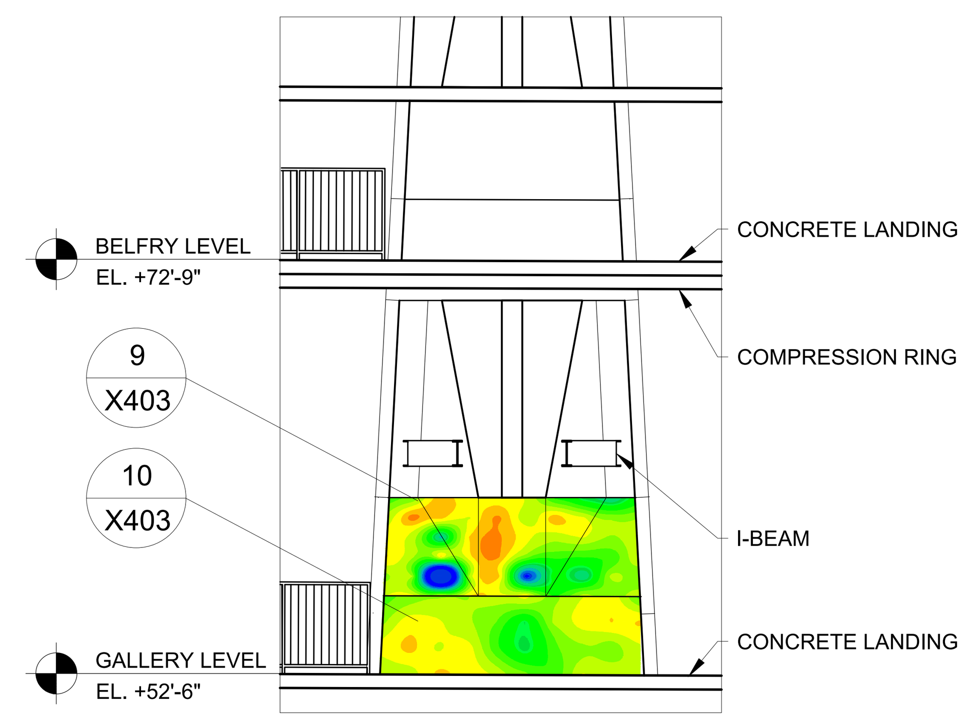
ANSI D SIZE DRAWING 22 X 34



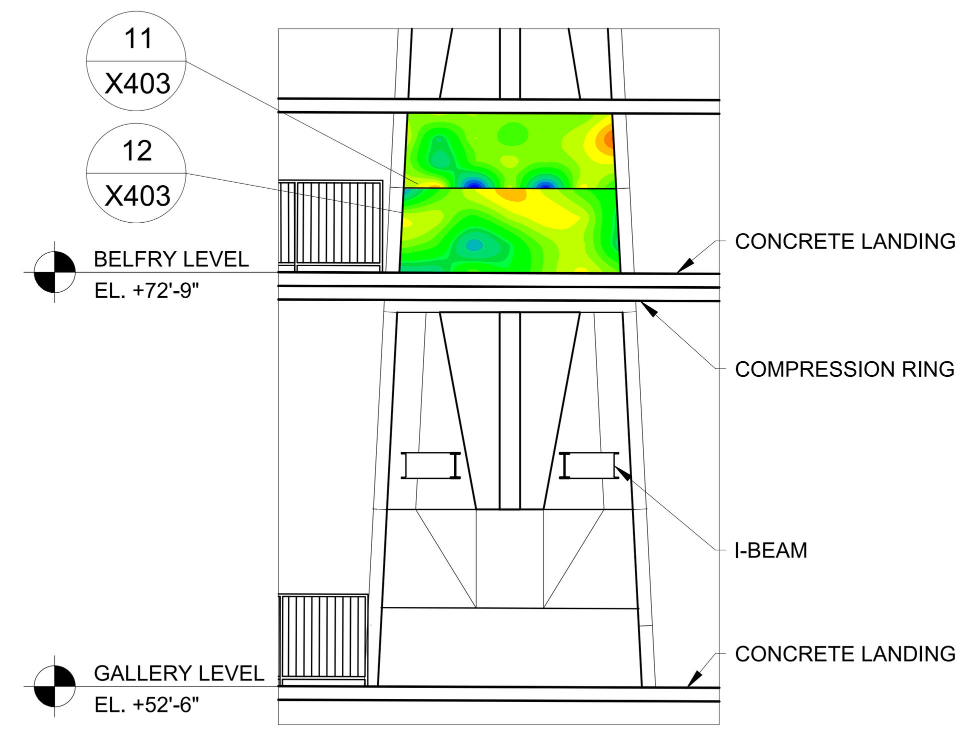
1 TESTING LOCATIONS - ELEVATION
SCALE: 1/8" = 1'-0"



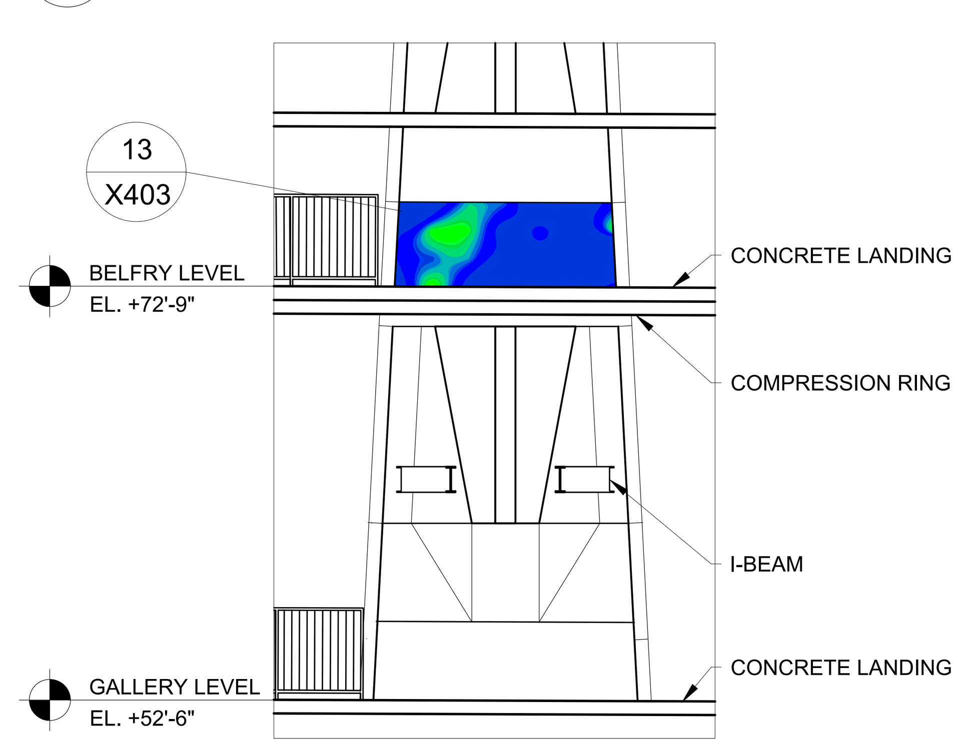
3 NORTHWEST
SCALE: 1/8" = 1'-0"



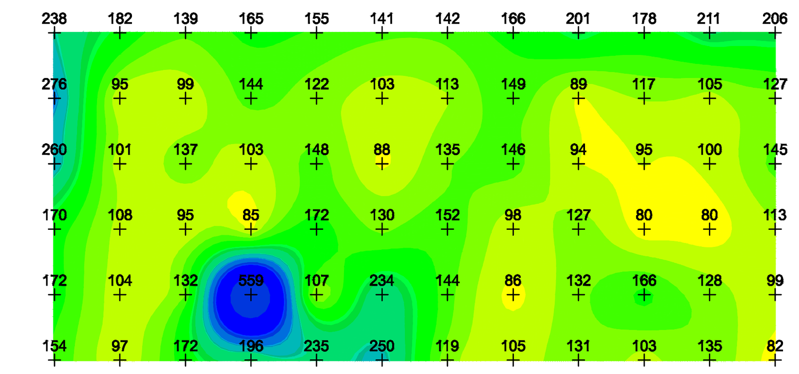
4 SOUTHEAST
SCALE: 1/8" = 1'-0"



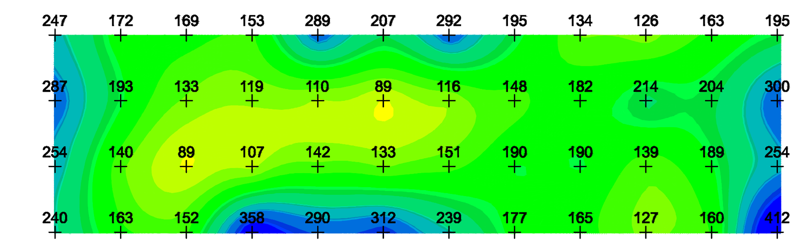
5 NORTHEAST
SCALE: 1/8" = 1'-0"



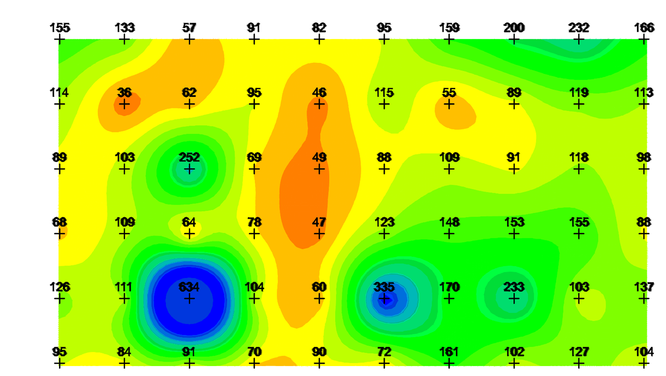
6 SOUTHWEST
SCALE: 1/8" = 1'-0"



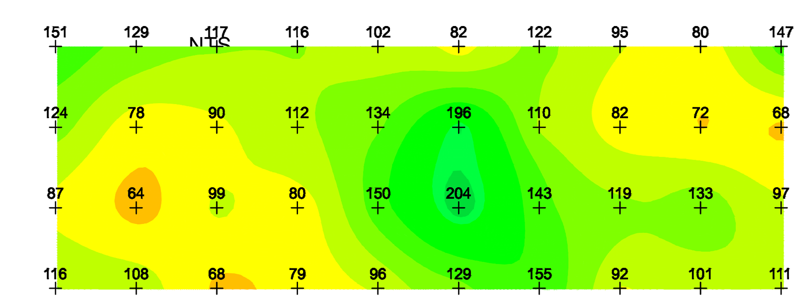
7 RESISTIVITY (i-ROM) CONTOUR MAP
NTS



8 RESISTIVITY (i-ROM) CONTOUR MAP
NTS



9 RESISTIVITY (i-ROM) CONTOUR MAP
NTS



10 RESISTIVITY (i-ROM) CONTOUR MAP
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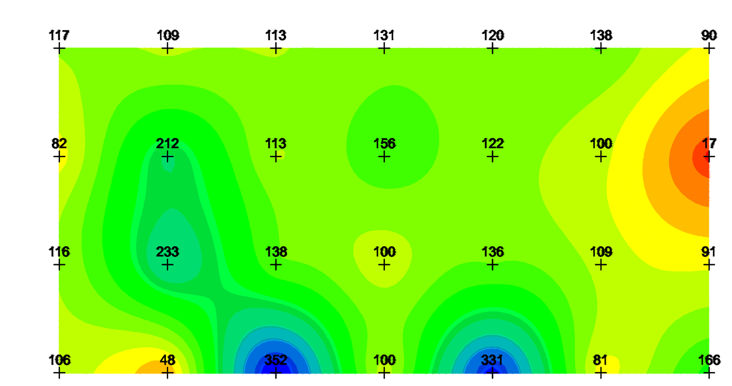

11 RESISTIVITY (i-ROM) CONTOUR MAP
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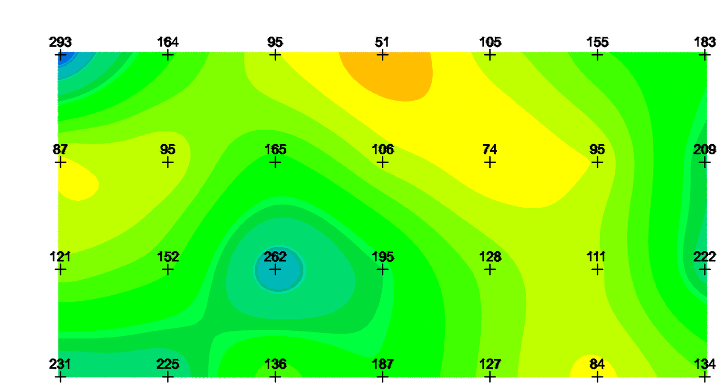
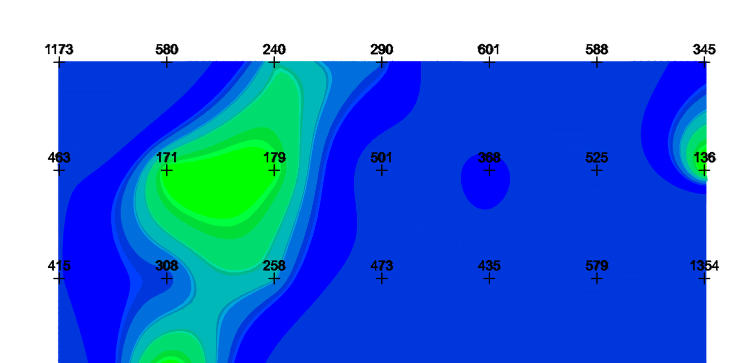


12 RESISTIVITY (i-ROM) CONTOUR MAP
NTS



13 RESISTIVITY (i-ROM) CONTOUR MAP
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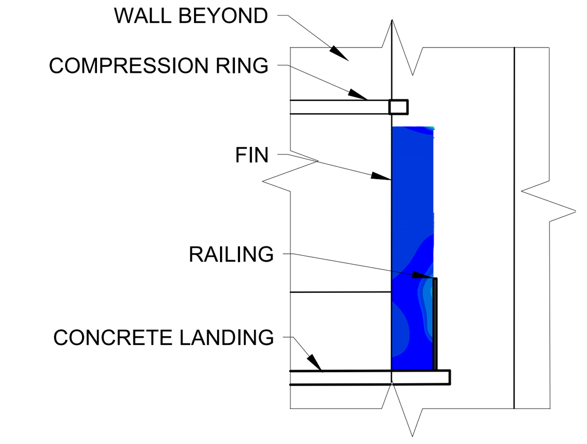
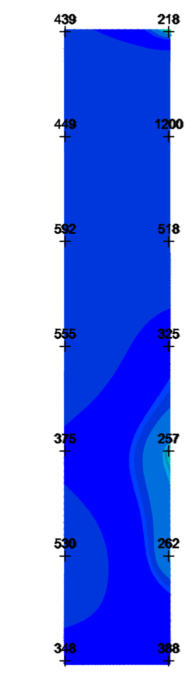
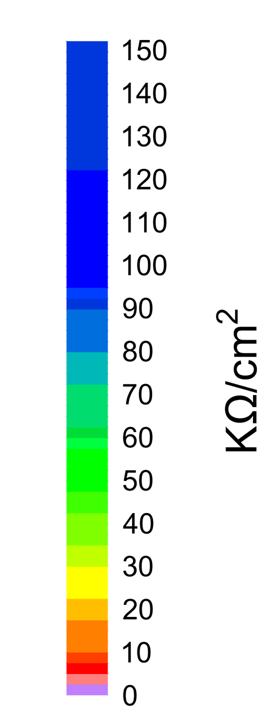
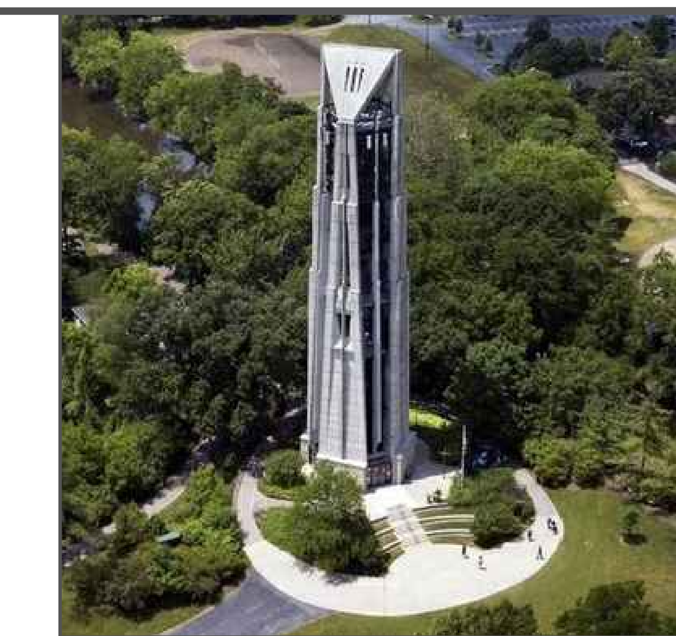


14 PROBE AT FIN
SCALE: 1/8" = 1'-0"



15 RESISTIVITY (i-ROM) CONTOUR MAP
NTS





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MATERIAL CONSULTANT:
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4 Jefferson Plaza, Suite 500
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CAD DWG FILE: RE-E17258-NF-V1.0_Moser Tower.dwg
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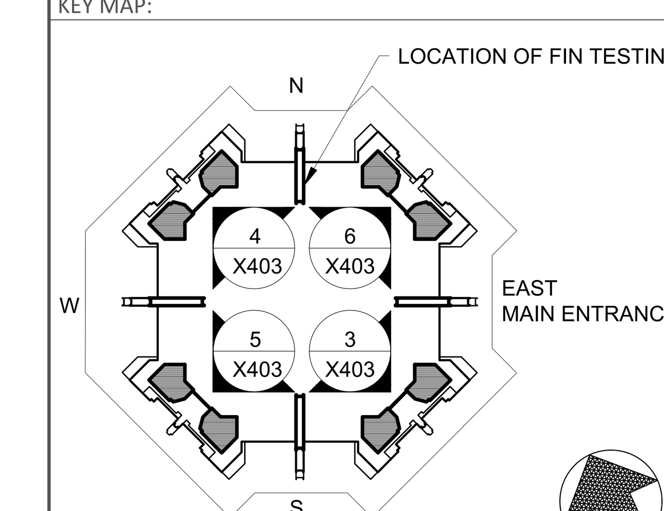
KEY MAP:



PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

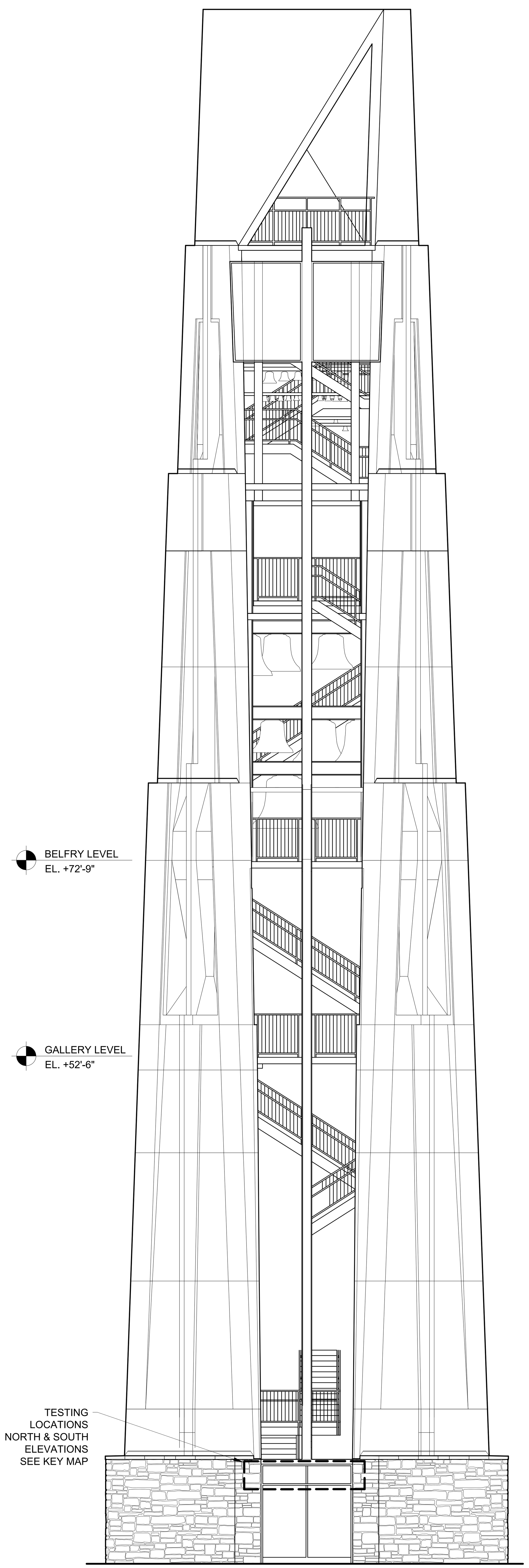
CORROSION ASSESSMENT

SHEET TITLE:
INTERIOR
RESISTIVITY (i-ROM)
CONTOUR MAPS

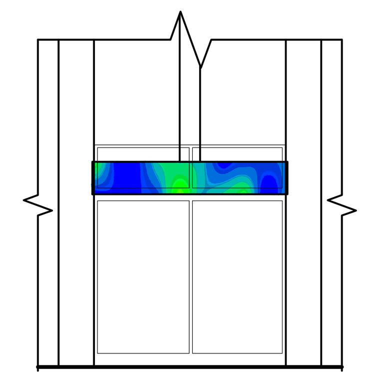
PROJECT REFERENCE NUMBER:
E17258

SHEET NUMBER:
X-403

REVISION:
00



1 TESTING LOCATIONS - NORTH, SOUTH ELEVATIONS
SCALE: 1/8" = 1'-0"



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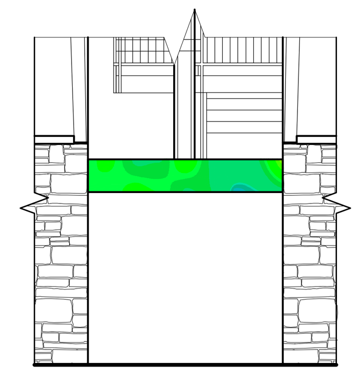

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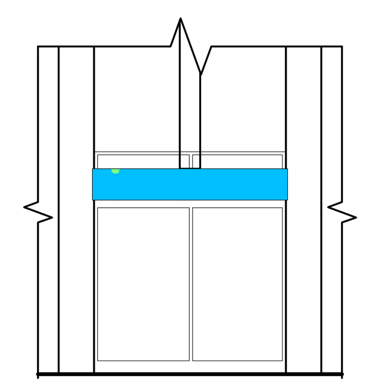

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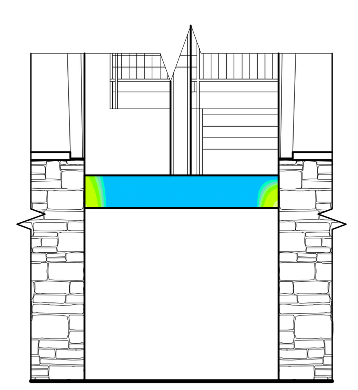

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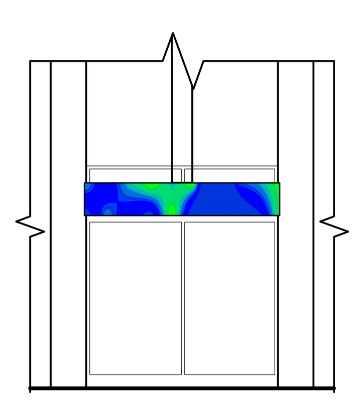
SCALE: 1/8" = 1'-0"



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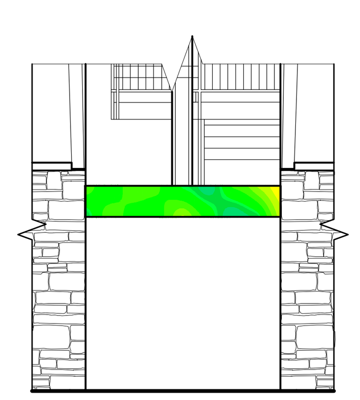

7 SOUTH LOCATION
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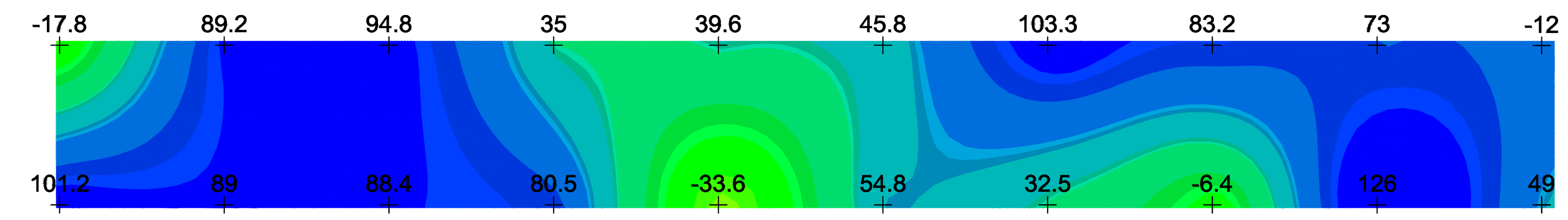
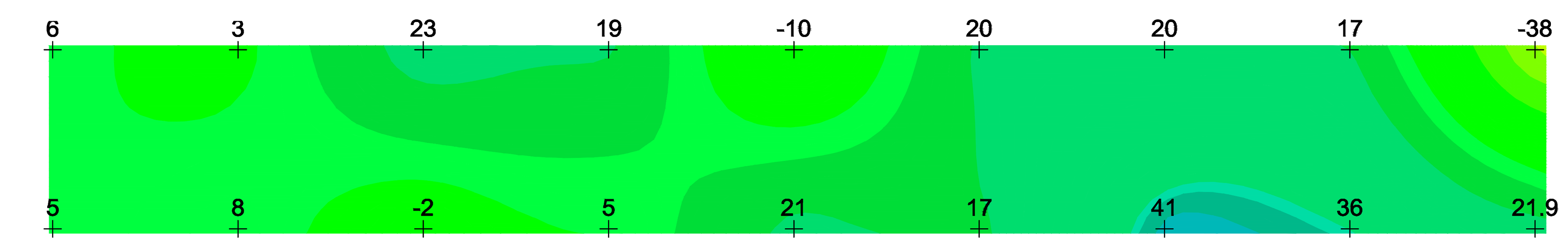
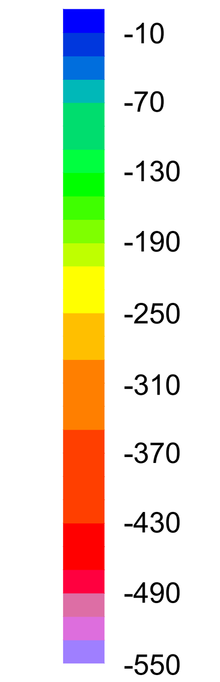
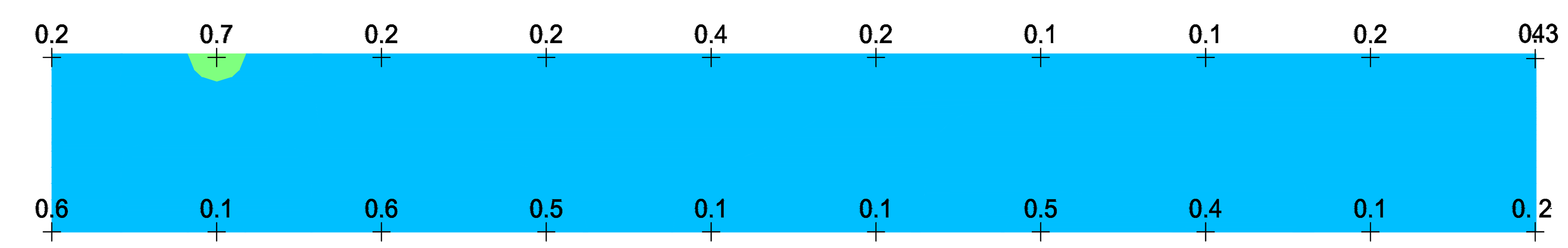


8 NORTH LOCATION CORROSION POTENTIALS (i-VOS) CONTOUR MAP
NTS

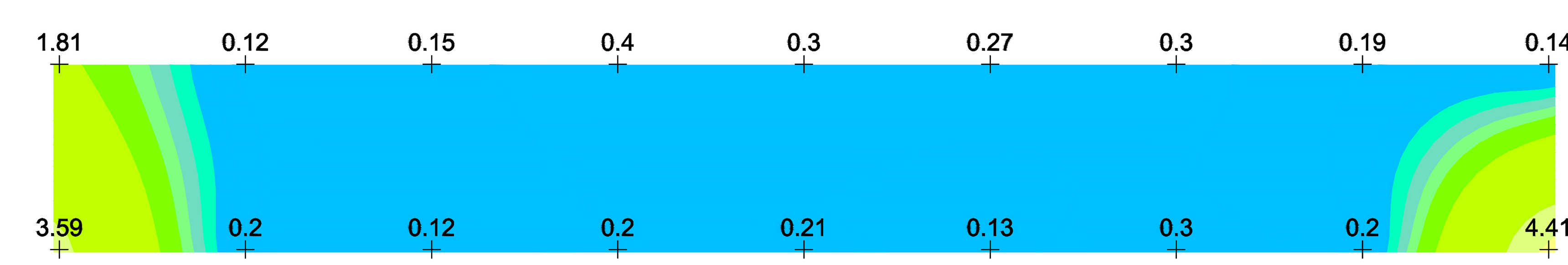


9 SOUTH LOCATION CORROSION POTENTIALS (i-VOS) CONTOUR MAP
NTS

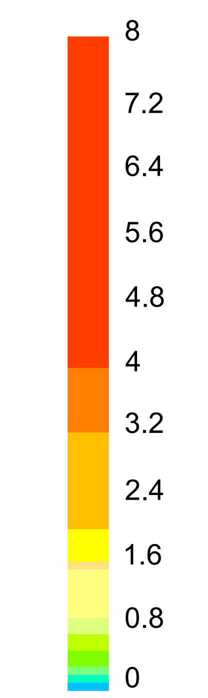


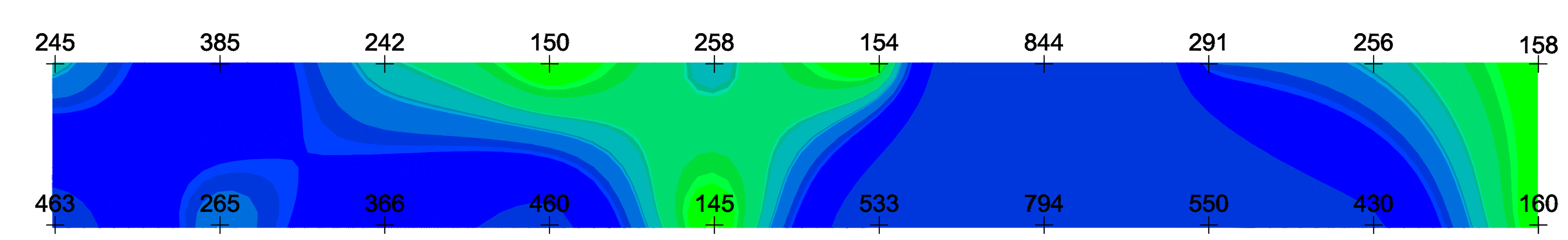


10 NORTH LOCATION CORROSION RATES (i-COS) CONTOUR MAP
NTS



11 SOUTH LOCATION CORROSION RATES (i-COS) CONTOUR MAP
NTS





12 NORTH LOCATION RESISTIVITY (i-ROM) CONTOUR MAP
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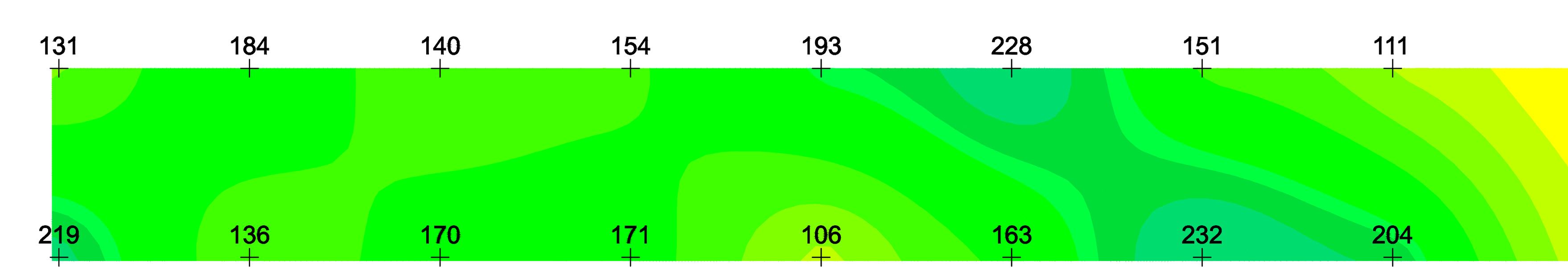
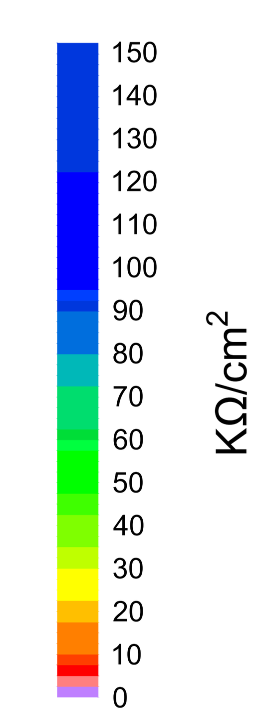
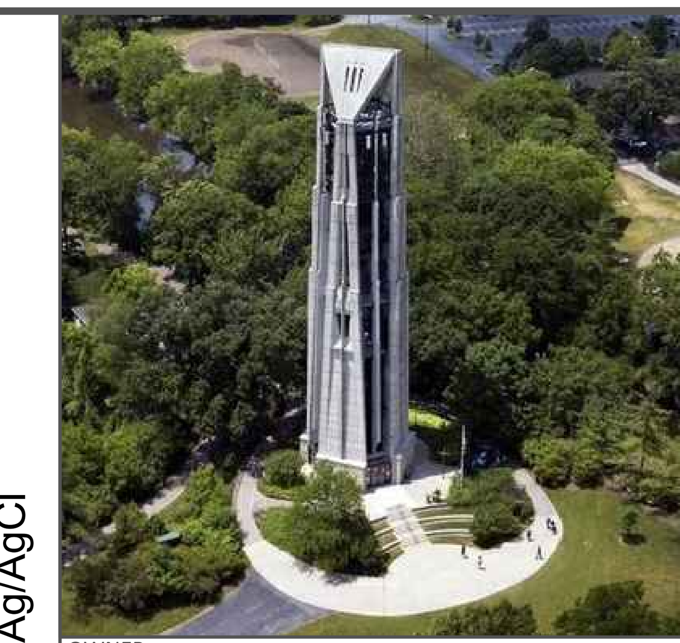


13 SOUTH LOCATION RESISTIVITY (i-ROM) CONTOUR MAP
NTS





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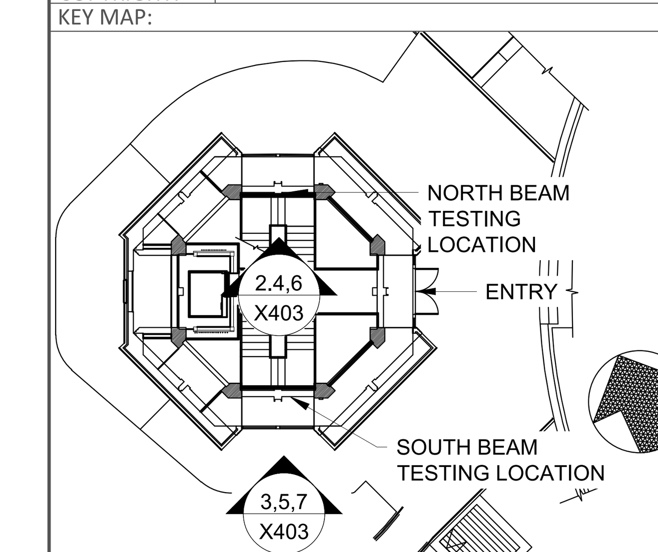
KEY MAP:



PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

SHEET TITLE:
NORTH / BEAMS
CORROSION POTENTIALS (i-VOS)
CORROSION RATES (i-COS)
& RESISTIVITY (i-ROM)
CONTOUR MAPS

PROJECT REFERENCE NUMBER:
E17258

SHEET NUMBER: X-404
REVISION: 00

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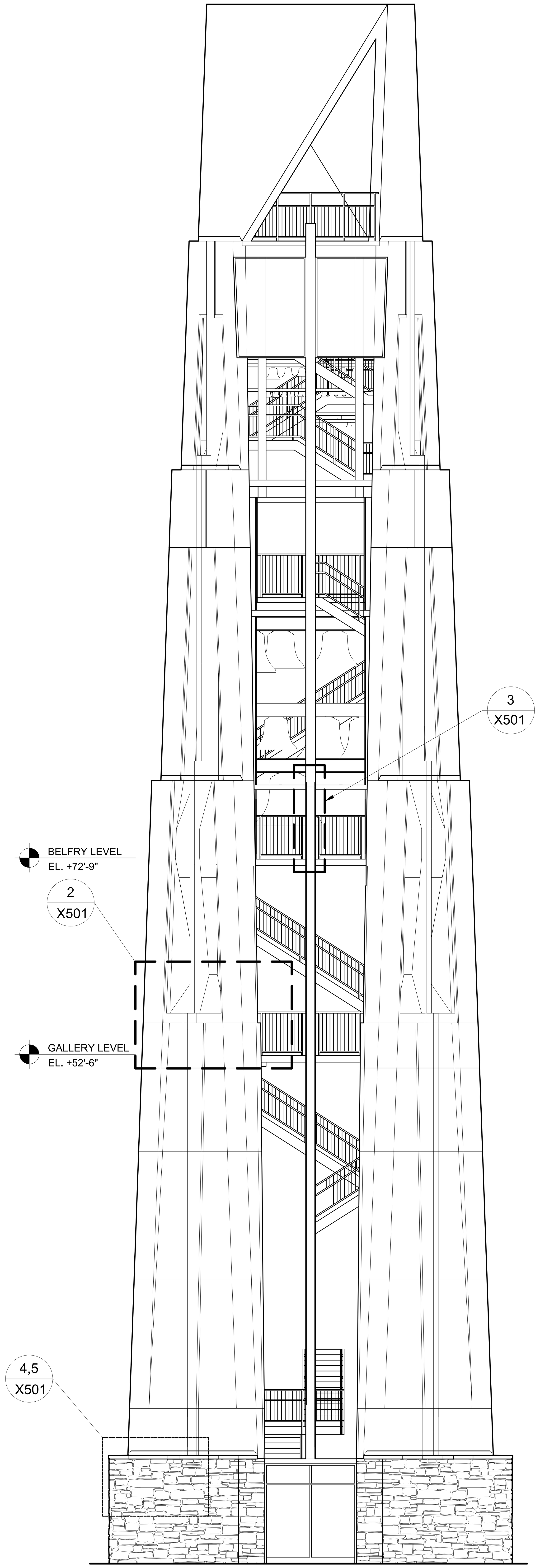
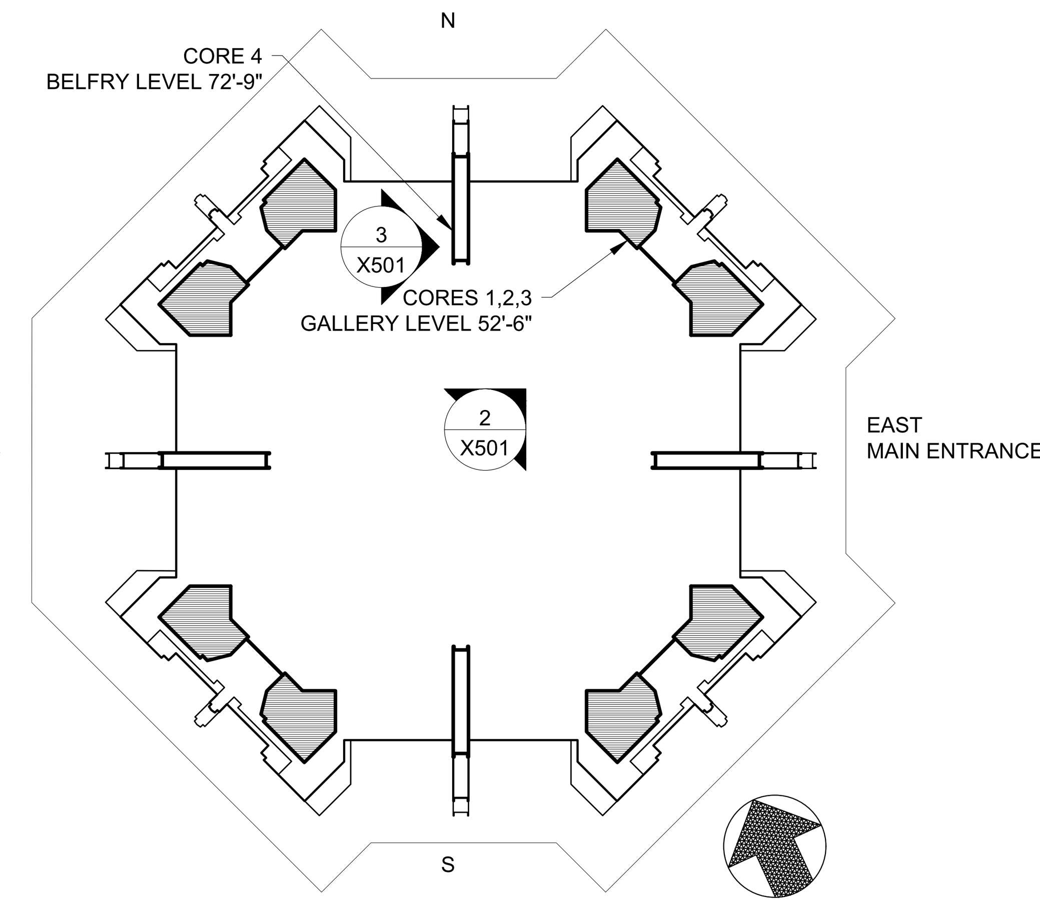
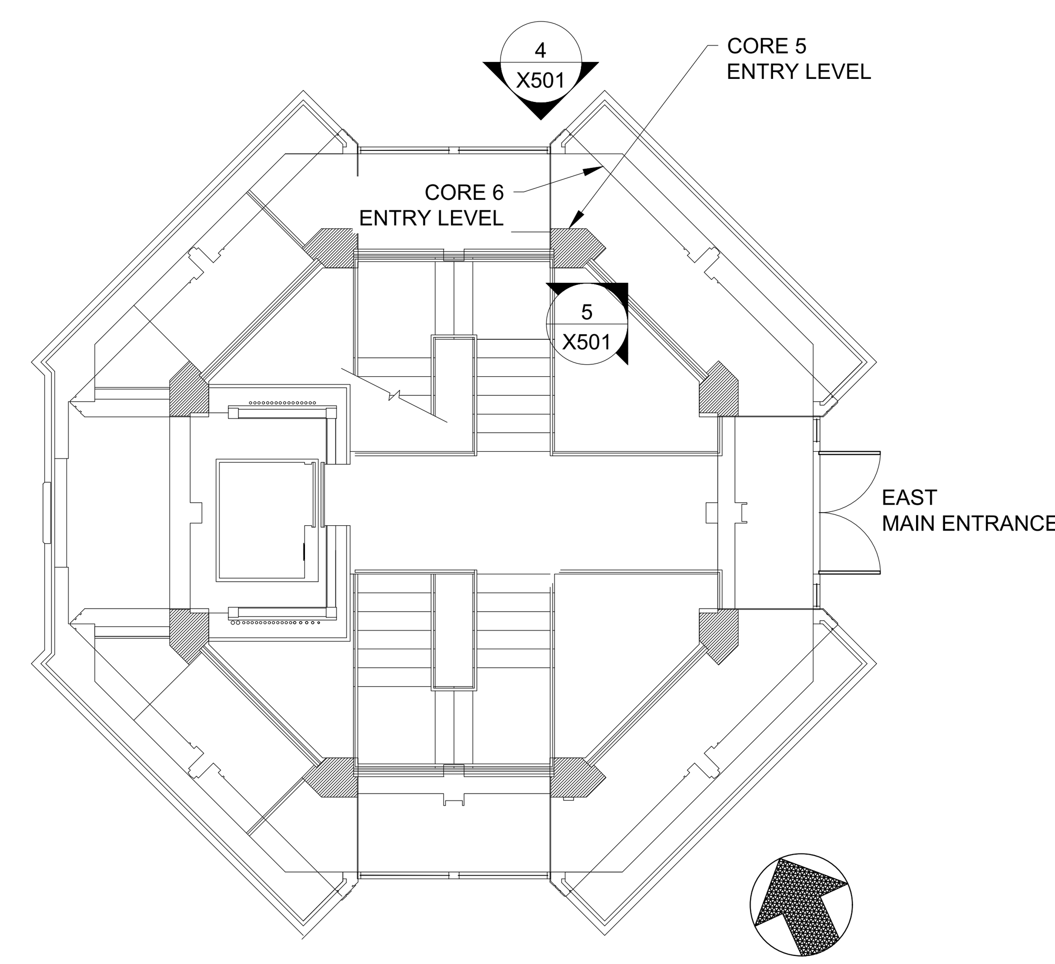


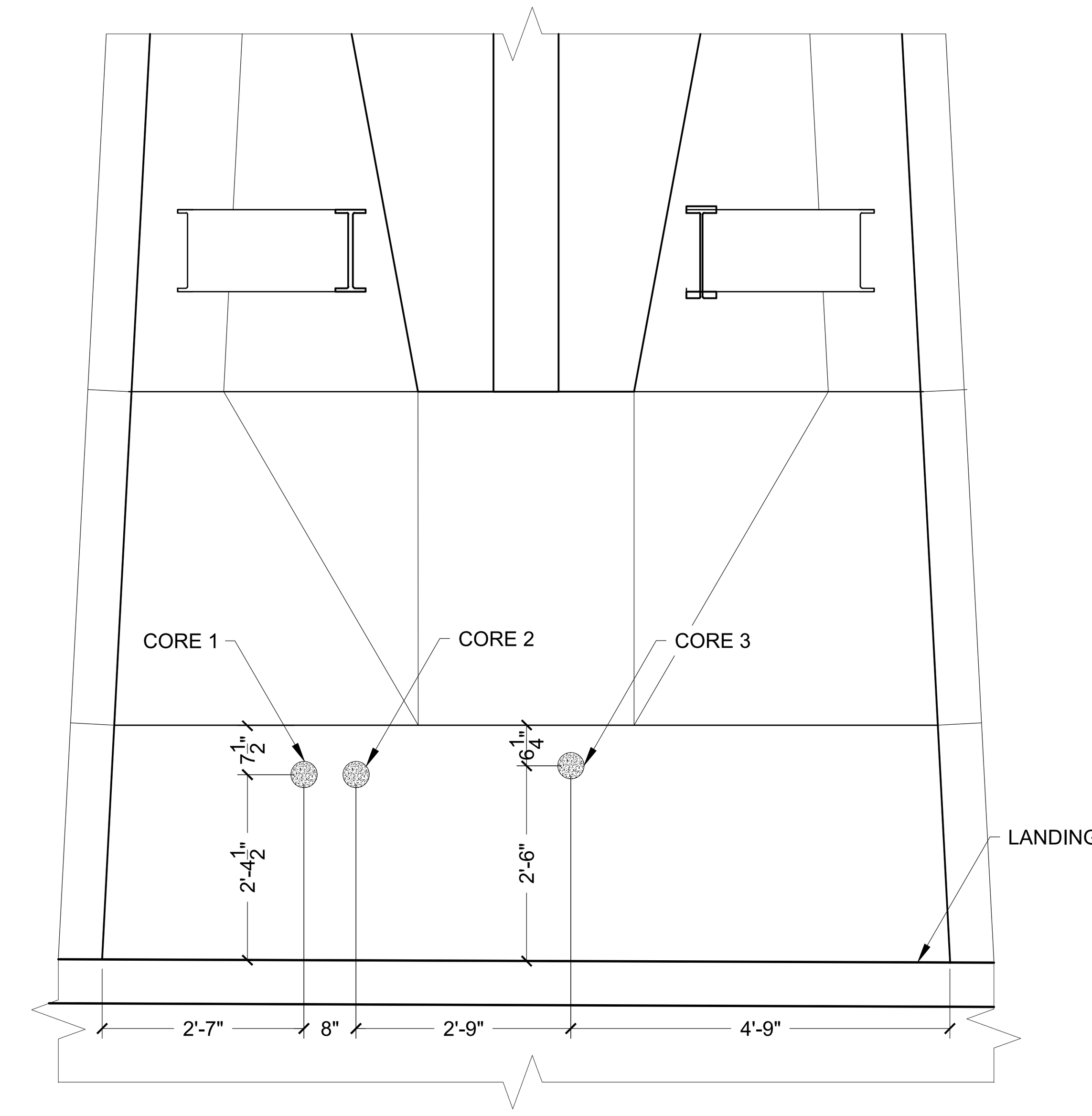
1 NORTH ELEVATION
SCALE: 1/8" = 1'-0"



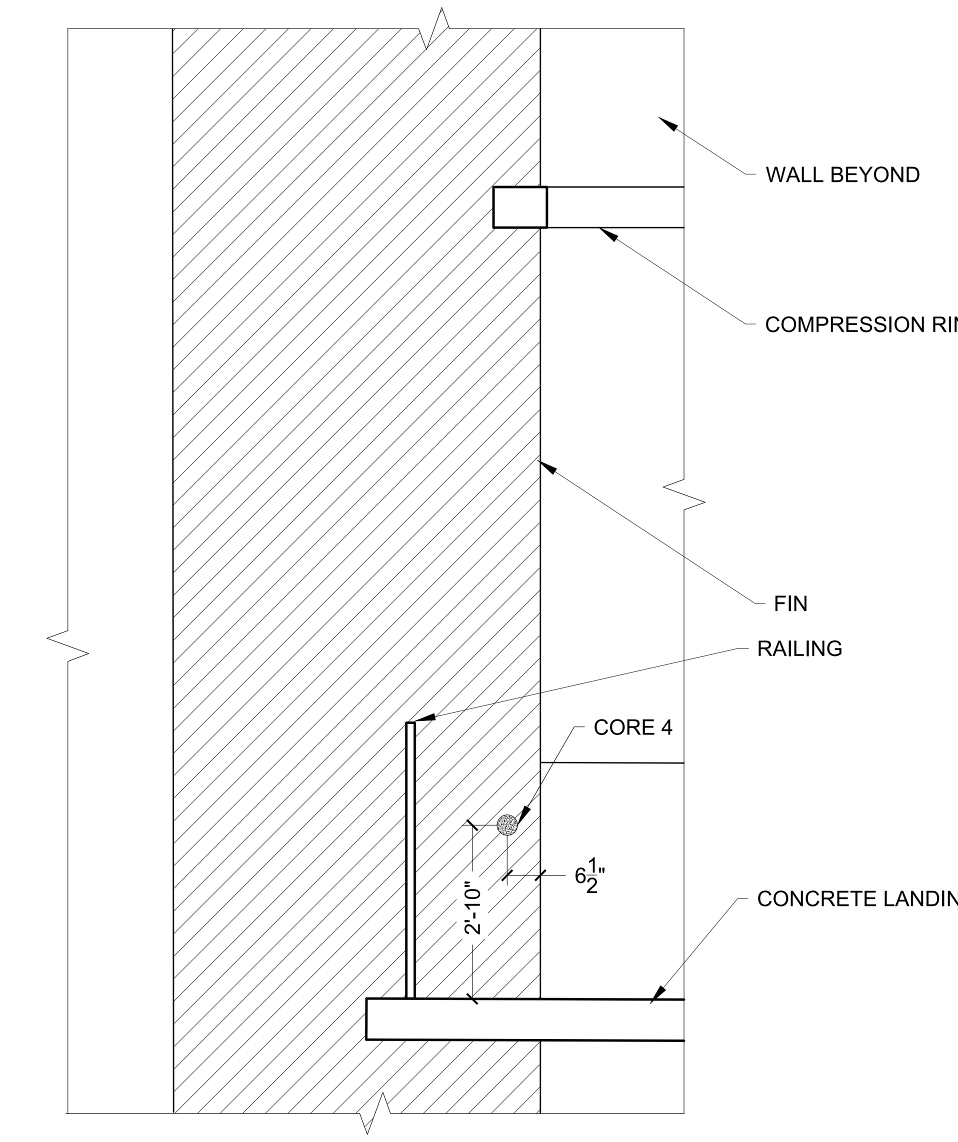
CORE 1 - 4 LOCATION PLAN
SCALE: 1/8" = 1'-0"



CORE 5 - 6 LOCATION PLAN
SCALE: 1/8" = 1'-0"



2 CORE LOCATIONS 1 - 3 GALLERY LEVEL
1/2" = 1'-0" SCALE:



3 CORE LOCATION 4 LEVEL 72.9
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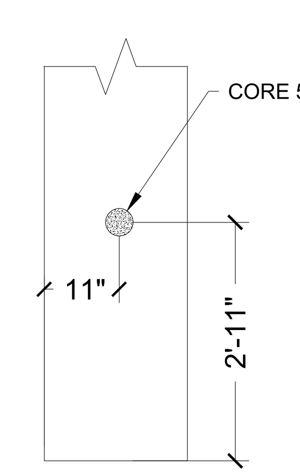
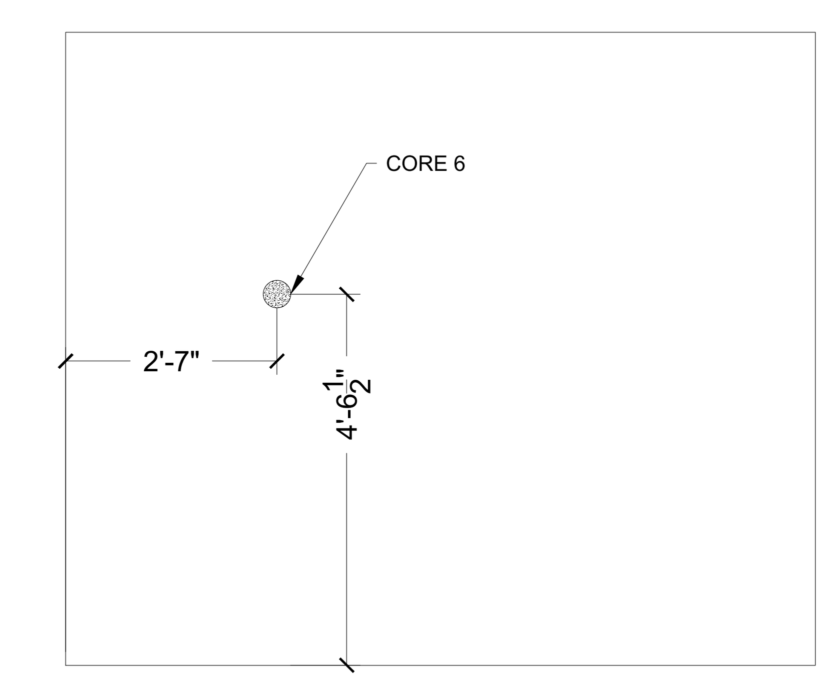


4 CORE LOCATION 5 ENTRY LEVEL
SCALE: 1/2" = 1'-0"



5 CORE LOCATION 6 ENTRY LEVEL
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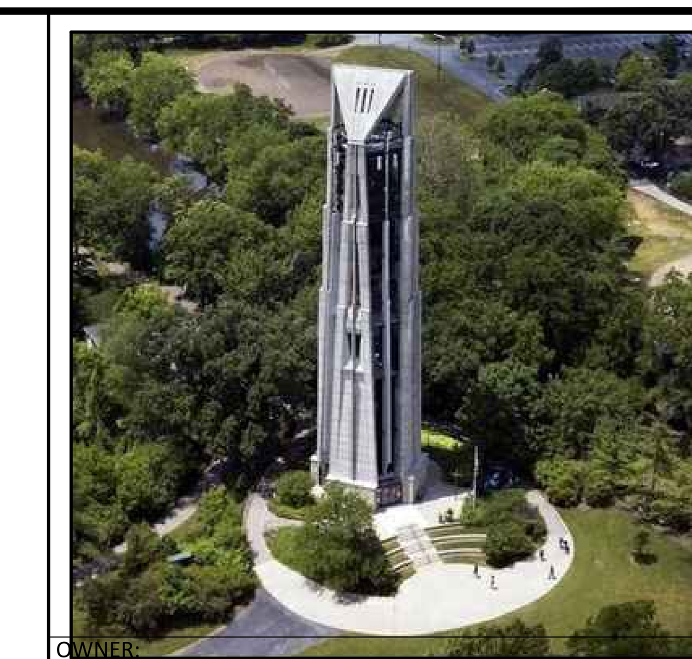

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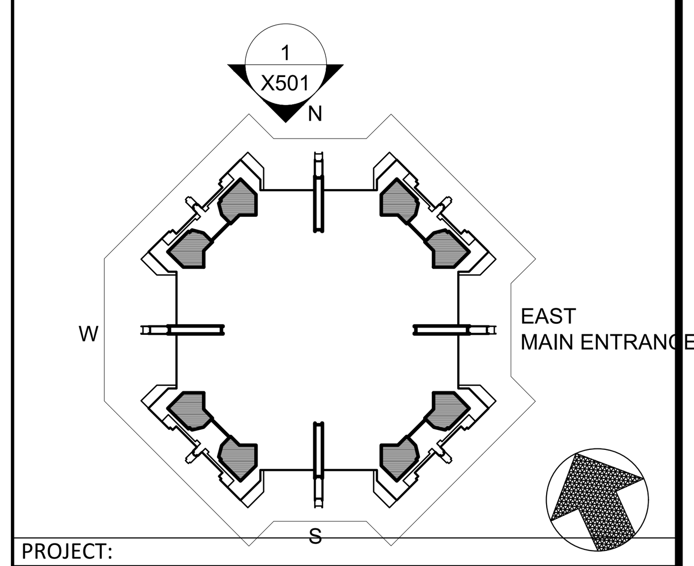

PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

SHEET TITLE:
CORE LOCATIONS

PROJECT REFERENCE NUMBER:
E17258
SHEET NUMBER: X-501
REVISION: 00

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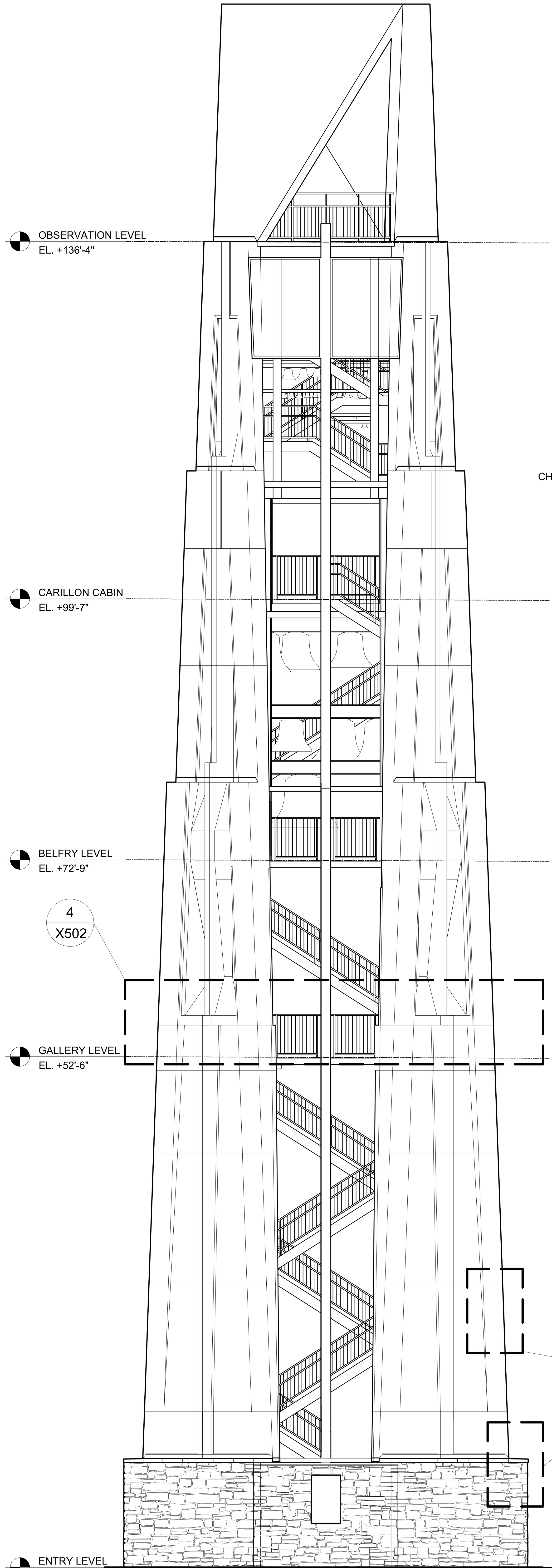
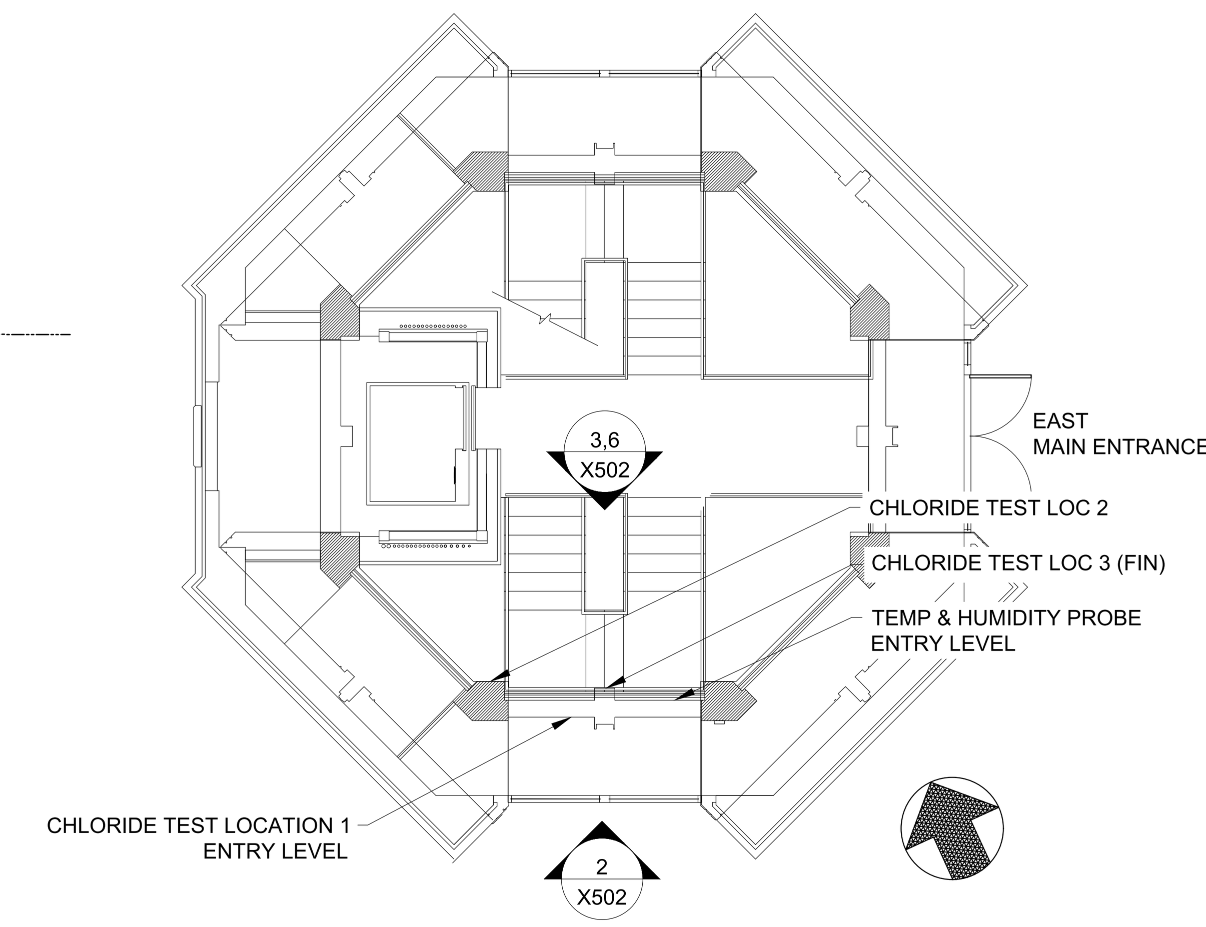
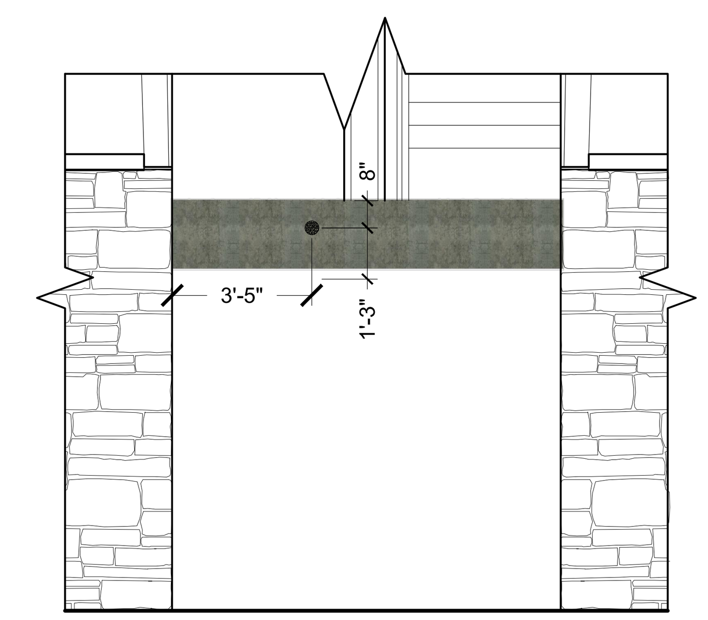


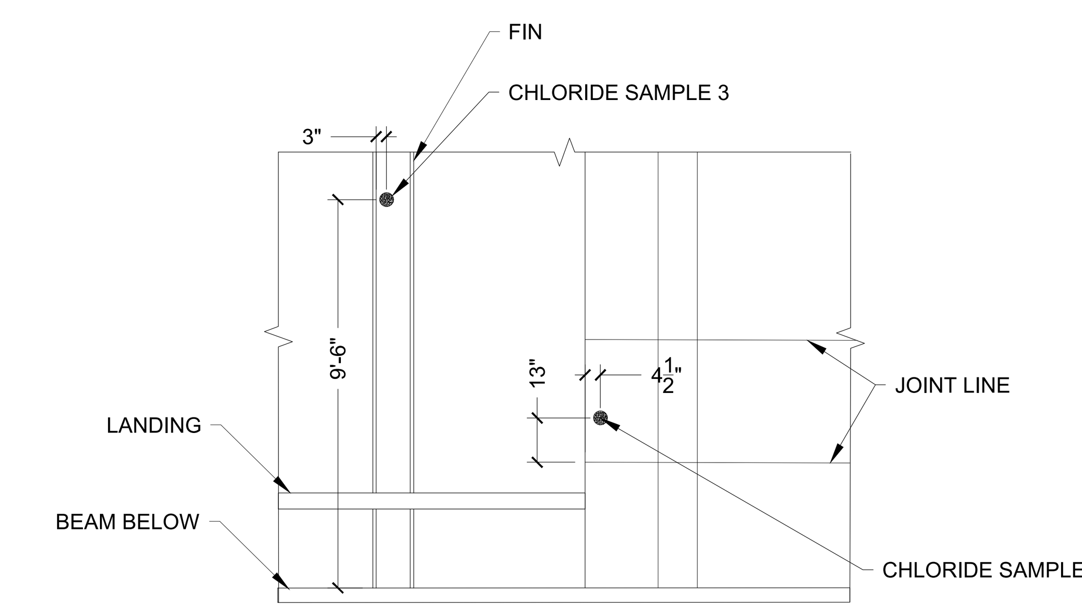
1 WEST ELEVATION
SCALE: 1/8" = 1'-0"



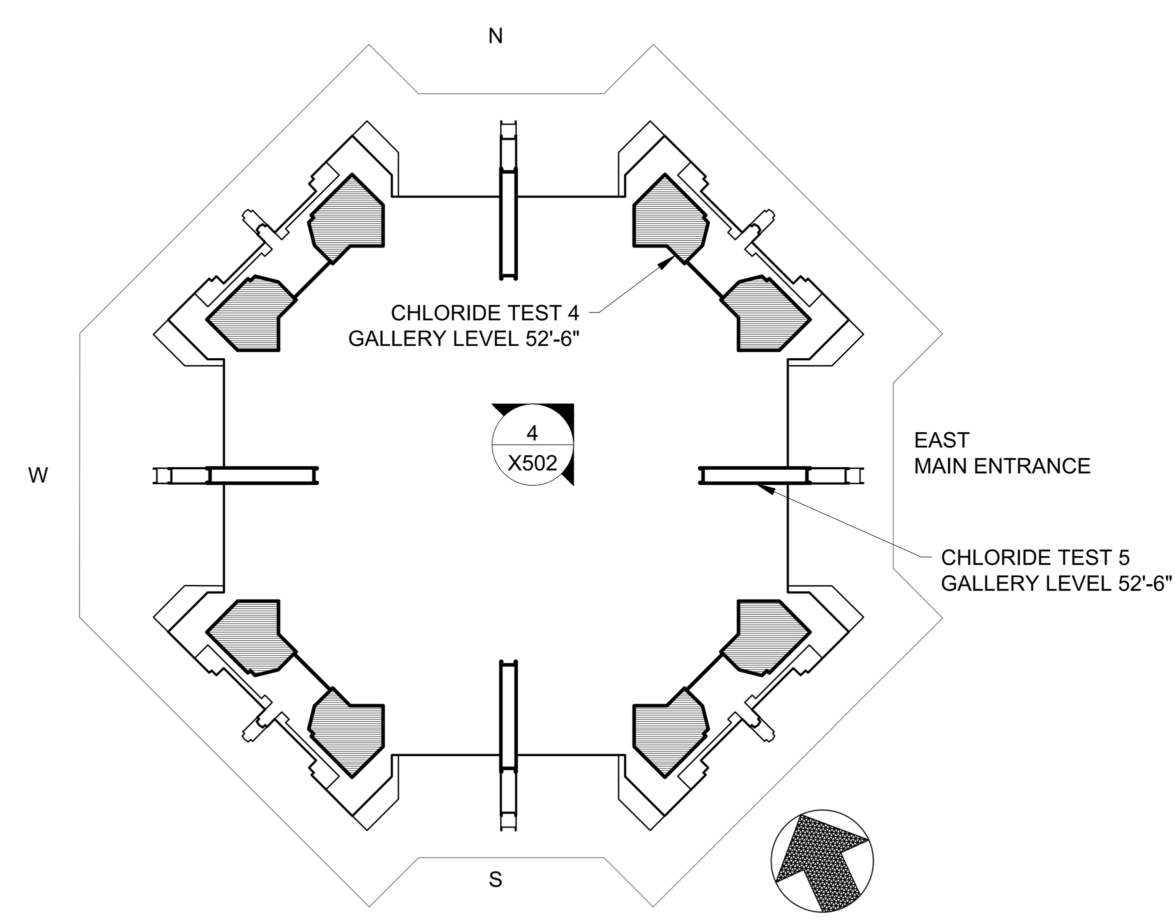
CHLORIDE TESTING 1 - 3 LOCATION PLAN
SCALE: 1/8" = 1'-0"



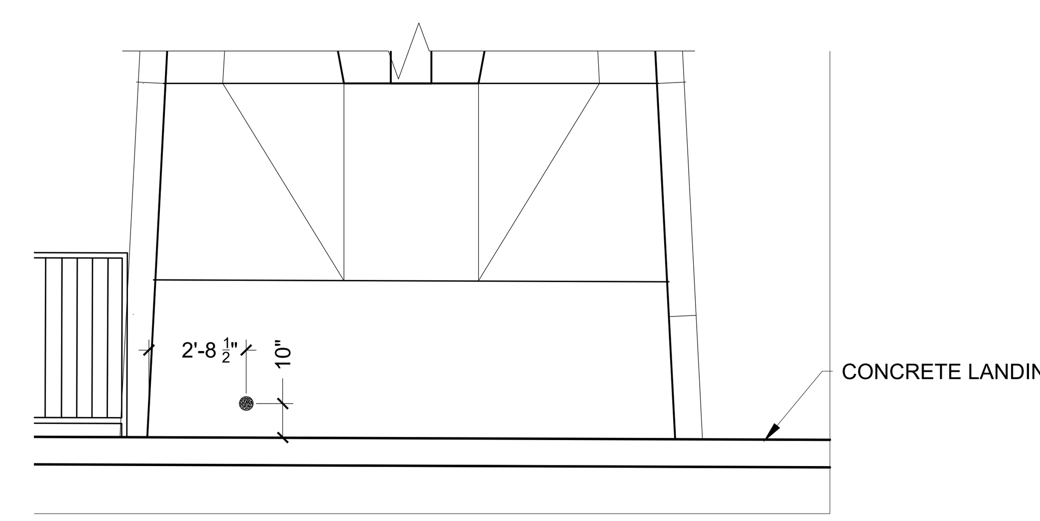
2 CHLORIDE SAMPLE (LOC 1)
SOUTH BEAM EXTERIOR
SCALE: 1/4" = 1'-0"



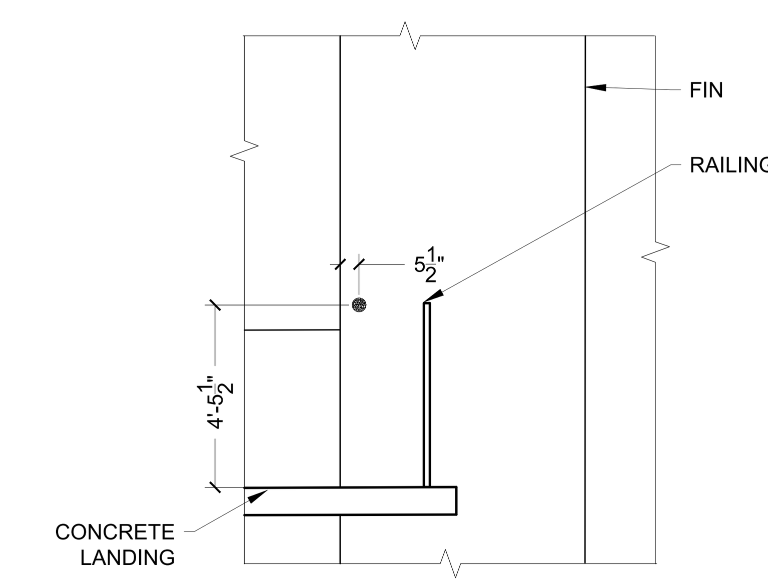
3 CHLORIDE SAMPLES (LOC 2 & 3)
SOUTH FACING LANDING
SCALE: 1/4" = 1'-0"



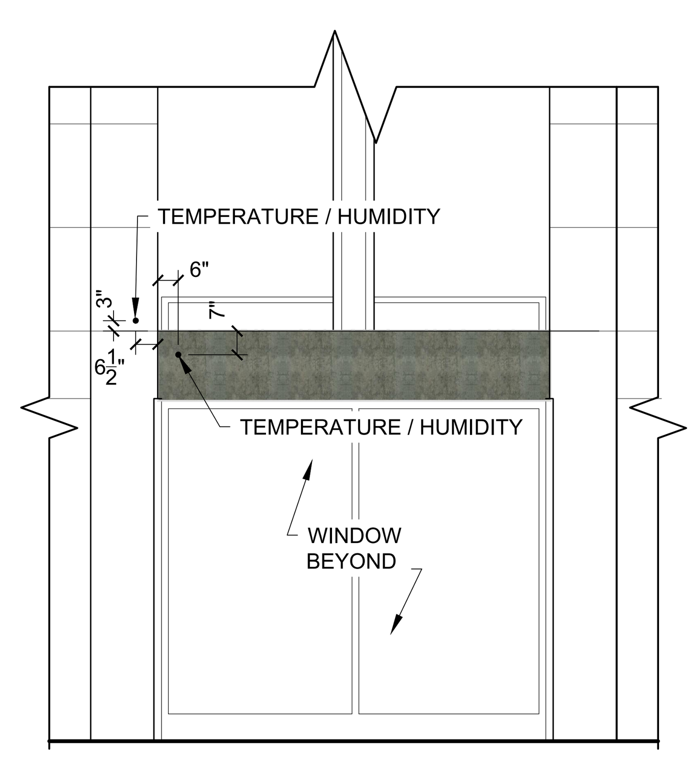
CHLORIDE TESTING 4 - 5 LOCATION PLAN
SCALE: 1/8" = 1'-0"



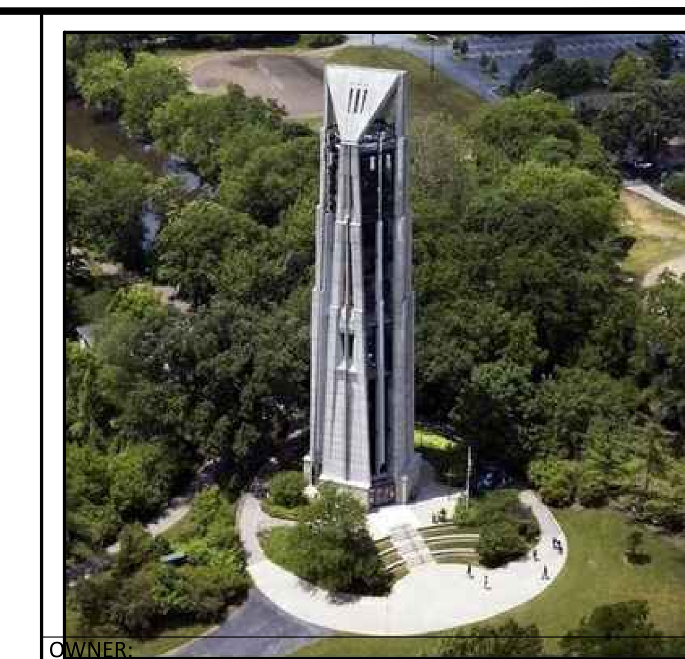
4 CHLORIDE SAMPLE (LOCATION 4)
GALLERY LEVEL
SCALE: 1/4" = 1'-0"



5 CHLORIDE SAMPLE (LOCATION 5)
GALLERY LEVEL NORTH FACADE, EAST FACING
SCALE: 1/4" = 1'-0"



6 EMBEDDED TEMPERATURE & HUMIDITY PROBES (LOC 1 & 2)
SOUTH BEAM INTERIOR
SCALE: 1/4" = 1'-0"



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CAD DWG FILE: RE-E17258-NF-V1.0_Moser Tower.dwg
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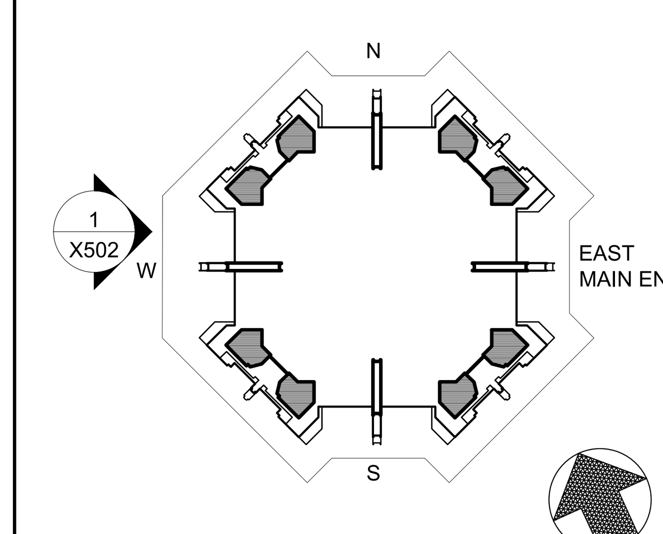
KEY MAP:



PROJECT:
MOSER TOWER
NAPERVILLE, ILLINOIS

CORROSION ASSESSMENT

SHEET TITLE:
CHLORIDE TESTING
TEMPERATURE & HUMIDITY
TESTING

PROJECT REFERENCE NUMBER:
E17258

SHEET NUMBER: X-502
REVISION: 00

APPENDIX B: CHLORIDE TESTING

ASTM C1152
ACID-SOLUBLE CHLORIDE ANALYSIS

Project No.	Date	Document Ref. No.
I17258	04/13/2018	

Project Name:	Moser Tower Durability Analysis
Site Address:	443 Aurora Ave
City/State/Zip:	Naperville, IL 60540
Client:	Engineering Resource Associates

Lab Tech:	Clarissa Roe
Structure Type:	Building
Year Built:	2000

Density: 3,800 lbs/yd^3 Cement Content: 546 lbs

No. of tests: 14

Reference ID	Interval	PPM	Lbs/yd^3	% wt concrete *1	% wt cement *2
South Beam- C1	1/8" - 1"	140	0.532	0.014	0.0974
South Beam- C1	1" - 2"	162	0.616	0.0162	0.1127
Pre-Cast Panel- C2	1/8" - 1"	177	0.673	0.0177	0.1232
Pre-Cast Panel- C2	1" - 2"	227	0.863	0.0227	0.158
Pre-Cast Fin- C3	1/8" - 1"	133	0.505	0.0133	0.0926
Pre-Cast Fin- C3	1" - 2"	252	0.958	0.0252	0.1754
Pre-Cast Panel- C4	1/8" - 1"	163	0.619	0.0163	0.1134
Pre-Cast Panel- C4	1" - 2"	265	1.007	0.0265	0.1844
Pre-Cast Fin- C5	1/8" - 1"	262	0.996	0.0262	0.1823
Pre-Cast Fin- C5	1" - 2"	439	1.668	0.0439	0.3055
Pre-Cast Fin- C5- Redo	1" - 2"	458	1.74	0.0458	0.3188
Grout Pocket- Pre-Cast F	1/8" - 1"	255	0.969	0.0255	0.1775
Grout Pocket- Pre-Cast P	1" - 2"	151	0.574	0.0151	0.1051
North- East Entry Level- (	1" - 2"	261	0.992	0.0261	0.1816

Notes:

*1 This air-dry density is assumed to be 3,800 lbs/yd^3 for the material. These values are estimated mathematically using the weights and the dimensions of the compressive strength samples. The density value is used in calculating the chloride content per cubic yard of material.

*2 For calculating percentage chloride by cement mass, the cement content is 546 lbs. (14.37%).

Comments:

Authorized by: Clarissa Roe 2018.08.23 14:46:22 -04'00'

FO-L70023-01
APPR. 11/01/16

APPENDIX C: PETROGRAPHIC ANALYSIS

May 23, 2018

Ms. Irene Matteini
eChem Consultants, LLC
4 Jefferson Plaza, Suite 50
Poughkeepsie, NY 12601

RE: Moser Tower, Naperville, IL
RJ Lee Group Project Number TCH804553

Dear Ms. Matteini:

A total of four (4) concrete cores were received by RJ Lee Group (RJLG) on April 18, 2018, for petrographic examination. The samples were extracted Moser Tower in Naperville, IL which was built in 2000 with reinforced concrete foundation and entrance wall and with pre-cast paneling used as cladding. Two samples are from the pre-cast panels and two are from the concrete elements. Table 1 lists the samples received and the RJLG assigned identification numbers. The purpose of the examination was to use petrographic methods to characterize the concrete, and to determine any degradation mechanism present.

Table 1. Sample Identification

eChem ID	RJLG ID	Description
#1	3150052	Pre-Cast Fin
#2	3150053	Pre-Cast Panel
#3	3150054	Column, Entrance Level
#4	3150055	Entrance Wall

The samples were analyzed following ASTM Method C 856 *Standard Practice for Petrographic Examination of Hardened Concrete,* and ASTM C 1723 *Standard Guide for Examination of Hardened Concrete Using Scanning Electron Microscopy (SEM)*. A cross sectioned slab of each core was polished for optical microscopy examination. A solution of phenolphthalein, a pH indicator, was applied to a freshly cut cross section to evaluate the depth of carbonation. This solution appears pink in contact with alkaline concrete with pH values in excess of 9, and colorless at lower pH which results when the paste has carbonated.

A polished thin section taken from an area of interest was prepared from each core using fluorescent dyed epoxy. The water-cement (w/c) ratio was estimated using a combination of techniques including, but not limited to, polarized and fluorescent light microscopy and SEM backscattered electron microscopy.

Summary

The pre-cast concrete was comprised of well hydrated Portland cement with natural gravel dolomitic limestone and natural siliceous and calcareous sand. The cores were generally in good condition. The Fin sample showed some light scaling and the Panel exhibited entrapped water channels at the paste aggregate interface. The samples did not appear to be adequately air entrained for freeze thaw resistance although no evidence of freeze thaw damage was observed.

The cast in place concrete elements were comprised of well to moderately hydrated GGBF slag and Portland cement blend with natural siliceous and calcareous sand. Both concrete cores were generally in good condition. The Column sample exhibited early age shrinkage micro-cracks with the longest at 1 ⅜". This sample was in good condition. The Entrance Wall sample appeared to be comprised of two pours with slight retarded hydration along the interface. The pours were similar with only slight changes in air void distribution and porosity noted.

The results of the petrographic examinations are presented in Appendix A at the end of this report, including photographs and representative images of the sample.

These results are submitted pursuant to RJ Lee Group's current terms and conditions of sale, including the company's standard warranty and limitation of liability provisions. No responsibility or liability is assumed for the manner in which the results are used or interpreted. This test report is not to be reproduced except in full, without written approval of the laboratory. Unless notified to return the samples covered in this report, RJ Lee Group will store them for a period of ninety (90) days before discarding.

Should you have any questions regarding this information, please do not hesitate to contact me.

Sincerely,

Patty Sue Kyslinger

Patty Sue Kyslinger
Concrete Petrographer

April Snyder

April Snyder
Senior Concrete Petrographer
CM Laboratory Manager

Appendix A

Petrographic Observations and Images

TCH804553 Moser Tower, Naperville, IL

RJ LEE GROUP
DELIVERING SCIENTIFIC RESOLUTION

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List of Figures

Client ID: Pre-Cast Fin		RJLG ID: 3150052	
Sample description: Pre-cast concrete			
Diameter	1 ¾"	Length	3 ⅜"

Summary of Petrography and Analytical Results					
Overall Condition	Estimated Air (%)	Estimated Paste (%)	Estimated w/c ratio	Cement Type & Hydration	Reinforcement Cover
Good	2-3	27	0.55 ± 0.05	Well hydrated Portland cement	Imprint of ½" ∅ steel bar observed on bottom of core

Properties	
Coarse Aggregate Type	Natural gravel comprised of dolomitic limestone with traces of sandstone and chert
Maximum Size	¼"
Gradation	Lack of mid-sized coarse aggregate
Shape	Rounded to sub-rounded
Distribution	Even
Bond to Paste	Good
Fine aggregate type	Natural calcareous and siliceous sand with trace amount of chert
Air Void Type & Distribution	Unevenly distributed small to medium sized spherical voids with the largest at $^{3}/_{16}$" long
Carbonation	Uneven up to $^{3}/_{16}$" deep
Paste Color & Hardness	The paste was hard and Very Light Gray (Munsell N8) in color.
Cracks/Microcracks	No cracking was observed.

Observations

- The top surface was lightly scaled with exposed sand and a few coarse aggregate exposed less than 1 mm.
- The bottom was snapped with an impression of a steel bar.
- Innocuous alkali silica reaction (ASR) of micro-crystalline quartz within a chert fine aggregate was detected. No cracking in the aggregate or into the paste had occurred.
- Variable porosity and abundant calcium hydroxide in the paste throughout the thin section.
- Minor-trace amounts of secondary ettringite formation within the paste and former cement grains was present in thin section.
- The cement grains appear to have two-toned relic structure indicative of the potential for high temperatures during curing which could lead to delayed ettringite formation (DEF). No evidence of DEF was detected.

Table 1. Petrographic Results for Fin (3150052).

Top View

Bottom View

Side View

Side View

Figure 1. Fin (3150052). Photographs of core in as-received condition.

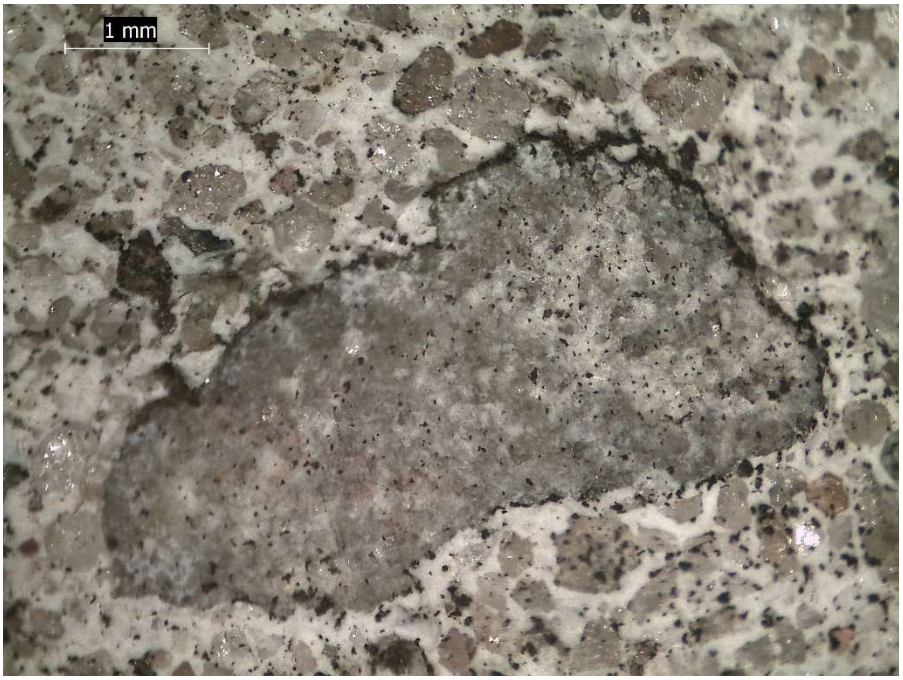


Figure 2. Fin (3150052). Stereo-optical micrograph showing exposed coarse and fine aggregates on the surface.

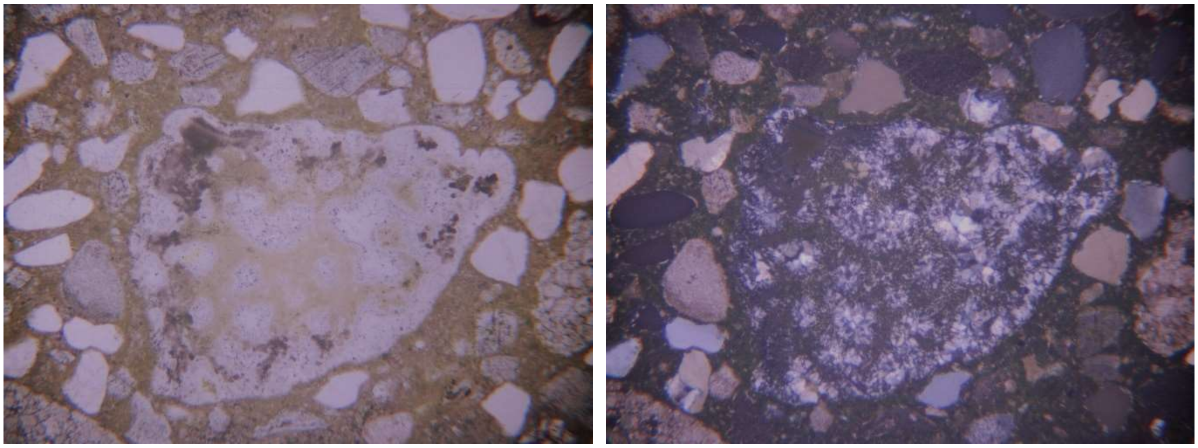
Figure 3. Fin (3150052). Stereo-optical micrograph showing the impression of the steel bar on the bottom of the core.

Figure 4. Fin (3150052). Photograph of freshly cut surface with phenolphthalein indicator applied.

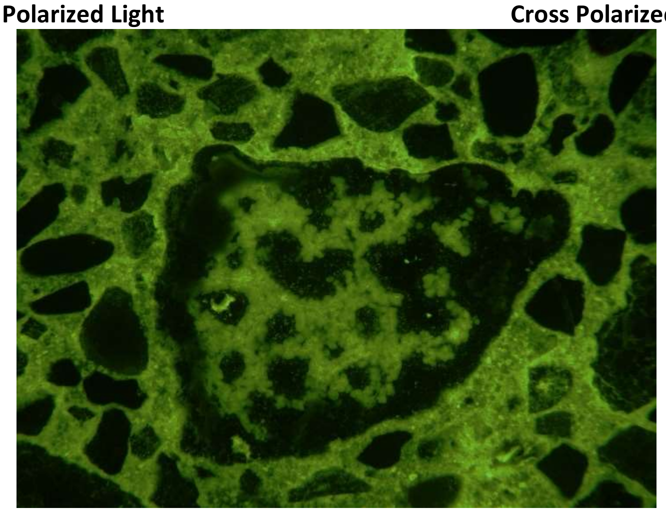
Figure 5. Fin (3150052). Photograph of polished cross sectioned slab.

Figure 6. Fin (3150052). Stereo-optical micrograph showing exposed coarse and fine aggregates on the surface.

Figure 7. Fin (3150052). Photograph of cross sectioned slab showing location of thin section preparation.



Plane Polarized Light **Cross Polarized Light**



Fluorescent Light

Figure 8. Fin (3150052). Optical micro-graph in different light modes showing a fine aggregate that is reactive rim, but no cracking in the aggregate or into the paste. Field of view 2.6 mm wide.

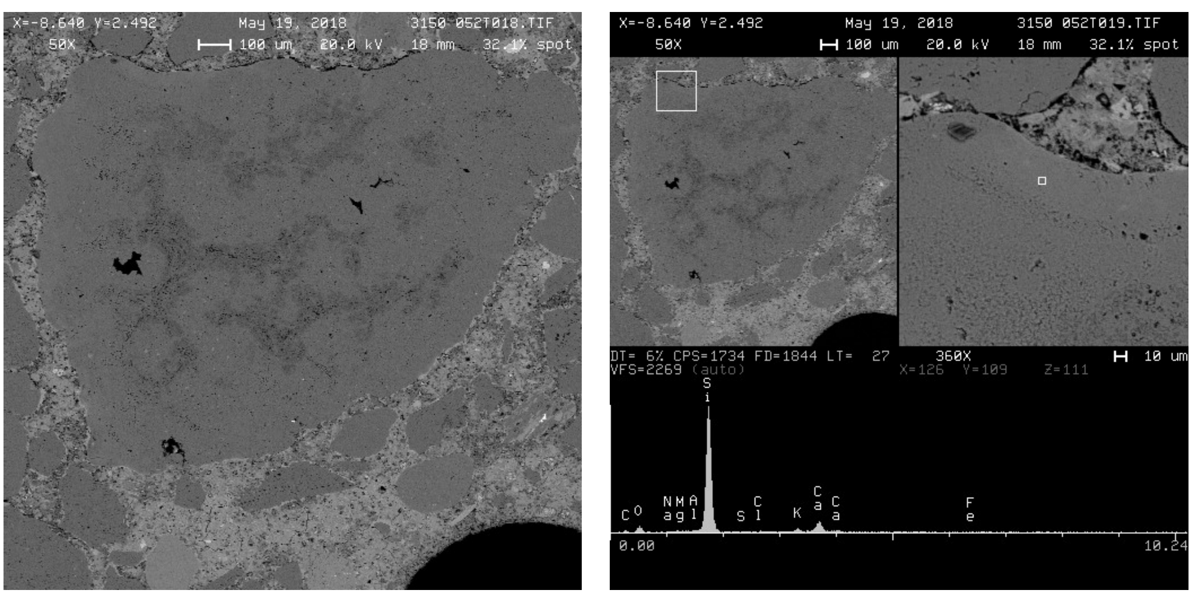


Figure 9. Fin (3150052). Backscattered electron (BSE) images with EDS spectrum showing the reactive rim on the fine aggregate and the ASR gel with traces of calcium and alkali (K).

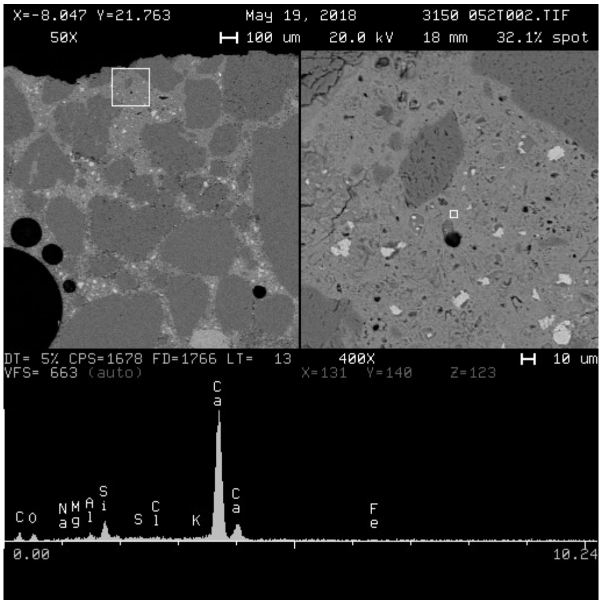


Figure 10. Fin (3150052). BSE images with EDS spectrum of carbonated paste.

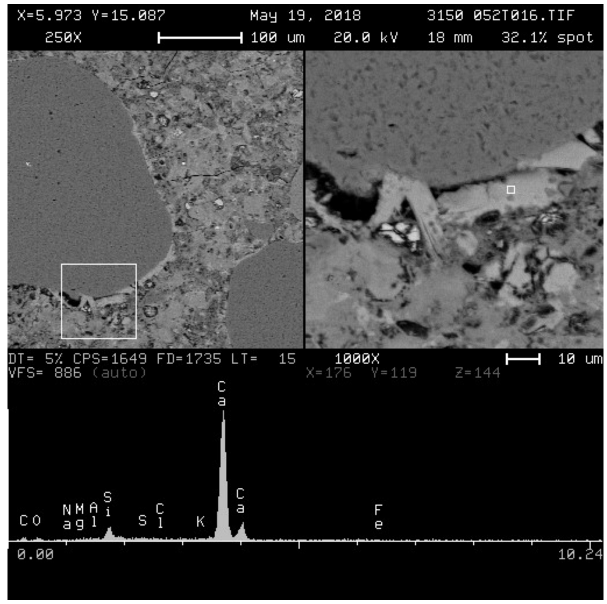


Figure 11. Fin (3150052). BSE images with EDS spectrum of calcium hydroxide along the paste/aggregate interface.

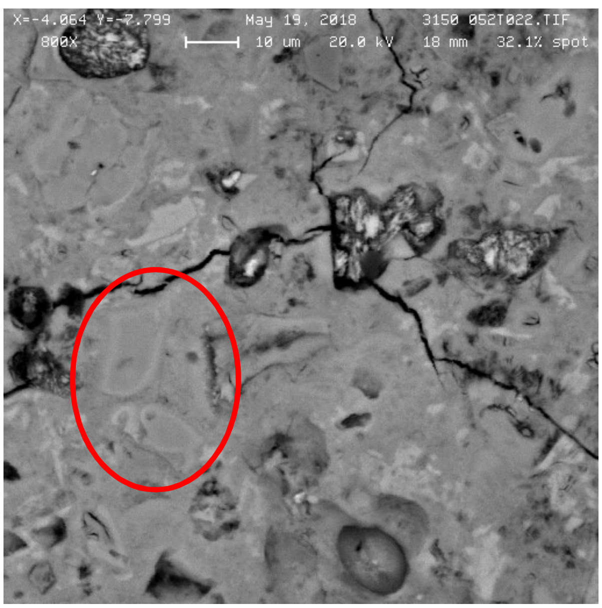


Figure 12. Fin (3150052). BSE image showing two toned rims of hydrated Portland cement grains.

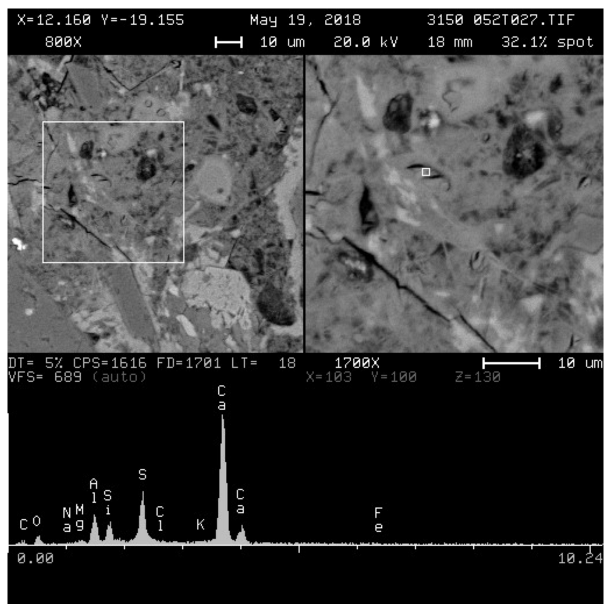


Figure 13. Fin (3150052). BSE images with EDS spectrum of ettringite in the porous paste.

Client ID: Pre-Cast Panel		RJLG ID: 3150053	
Sample description: Pre-cast concrete panel			
Diameter	3 ¾"	Length	4 ⅛"

Summary of Petrography and Analytical Results					
Overall Condition	Estimated Air (%)	Estimated Paste (%)	Estimated w/c ratio	Cement Type & Hydration	Reinforcement Cover
Good	5-7%	28	0.50 ± 0.05	Well hydrated Portland cement	ND

Properties	
Coarse Aggregate Type	Natural gravel comprised of dolomitic limestone with trace amounts of chert
Maximum Size	$^{5}/_{16}$"
Gradation	Even
Shape	Rounded to sub-rounded
Distribution	Good
Bond to Paste	Good
Fine aggregate type	Natural calcareous and siliceous sand
Air Void Type & Distribution	Unevenly distributed fine spherical air voids consistent with entrained air, and coarse voids up to ⅜" in diameter.
Carbonation	Uneven up to $^{5}/_{16}$"
Paste Color & Hardness	The paste was hard and consistent with Yellowish Gray (Munsell 5Y 8/1) in color.
Cracks/Microcracks	Not observed

Observations
• The top surface was a smooth cast surface with small air voids. Trace amount of map microcracking was present on the painted surface, but did not extend into the concrete. • The bottom was snapped through and around aggregates.
• Minor amount of very fine water channels or gaps along paste aggregate interface in top ⅛".
• Variable porosity and abundant calcium hydroxide in the paste.
• The cement grains appear to have two-toned relic structure indicative of the potential for high temperatures during curing which could lead to delayed ettringite formation (DEF). No evidence of DEF or any appreciable amount of secondary ettringite was detected.

Table 2. Petrographic Results for Pre-Cast Panel (3150053).

Top View

Bottom View

Side View

Side View

Figure 14. Pre-Cast Panel (3150053). Photographs of core in as-received condition.

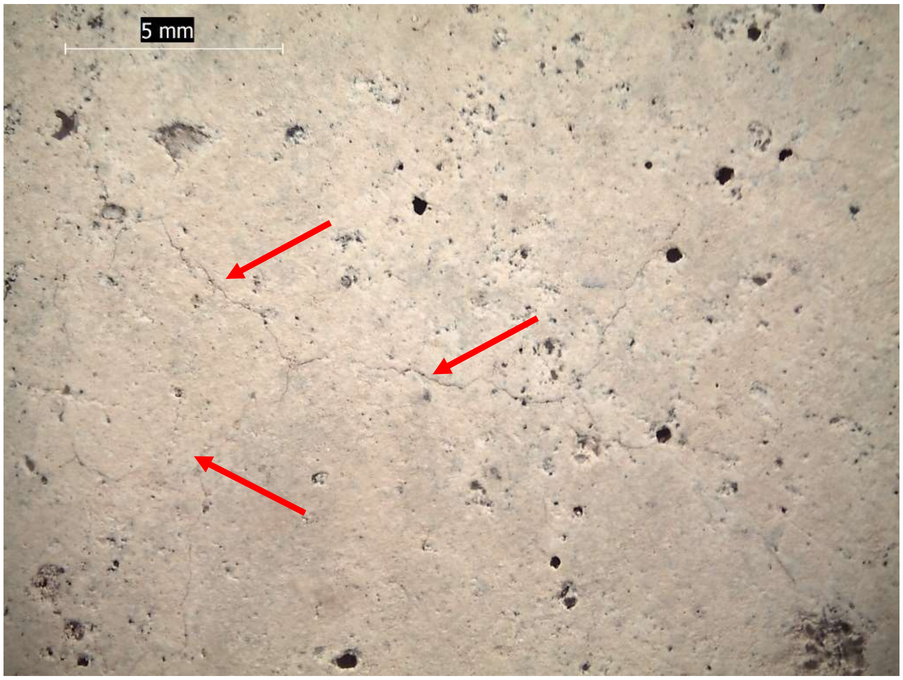


Figure 15. Pre-Cast Panel (3150053). Stereo-optical micrograph showing a minor amount of microcracks in the paint layer.

Figure 16. Pre-Cast Panel (3150053). Photograph of freshly cut surface with phenolphthalein indicator applied.

Figure 17. Pre-Cast Panel (3150053). Photograph of polished cross sectioned slab.

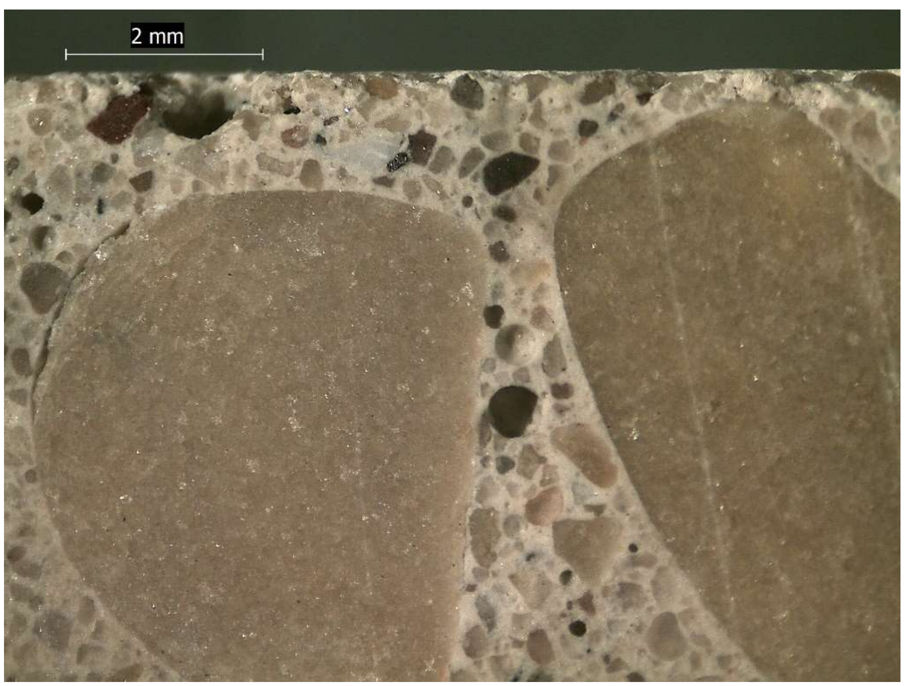


Figure 18. Pre-Cast Panel (3150053). Stereo-optical micrograph of the polished cross sectioned slab showing the top surface.

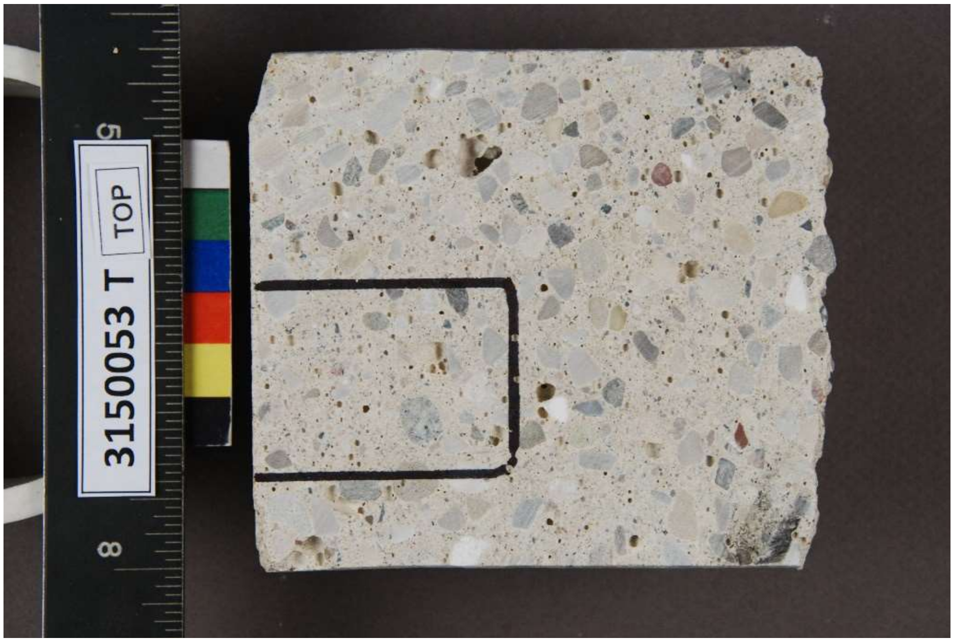
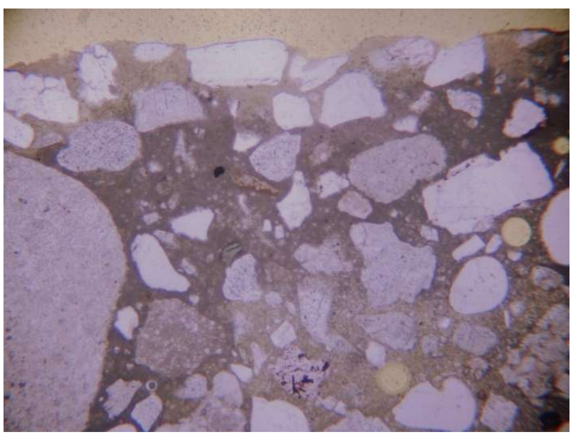
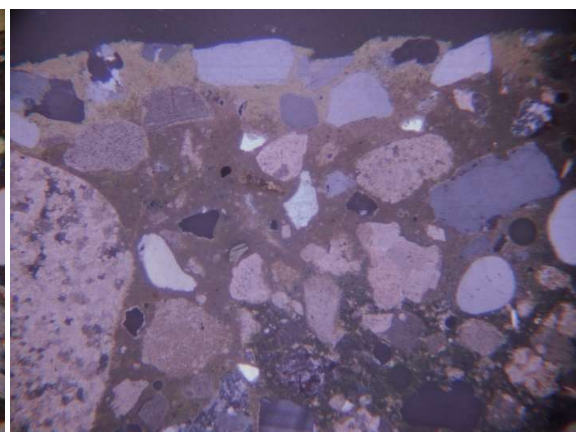


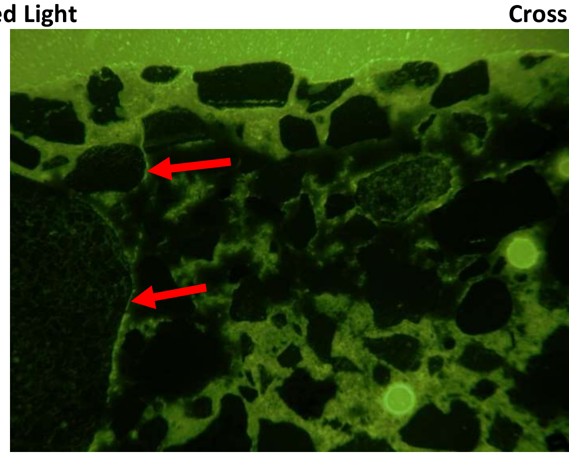
Figure 19. Pre-Cast Panel (3150053). Photograph of cross sectioned slab showing location of thin section preparation.



Plane Polarized Light



Cross Polarized Light



Fluorescent Light

Figure 20. Pre-Cast Panel (3150053). Optical micro-graph in different light modes showing carbonated paste and a water channel along aggregate (highlighted by red arrows). Field of view 2.6 mm wide.

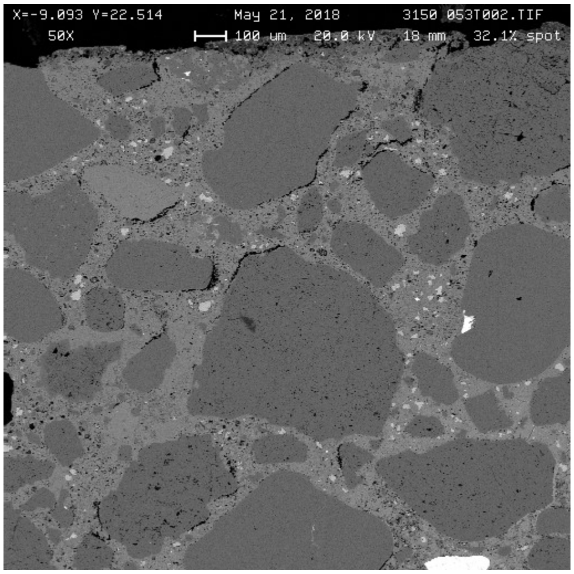


Figure 21. Pre-Cast Panel (3150053). BSE image of water channels or gaps along aggregate in the top ⅛".

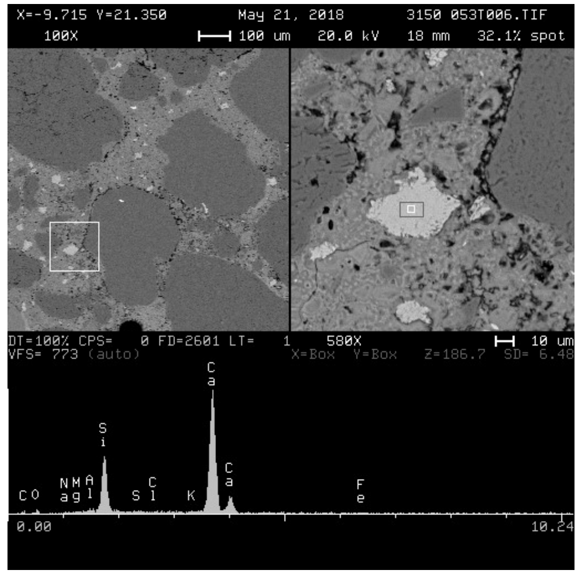


Figure 22. Pre-Cast Panel (3150053). BSE images with EDS spectrum of partially hydrated cement grain.

Figure 23. Pre-Cast Panel (3150053). BSE image showing two toned rims of hydrated Portland cement grains.

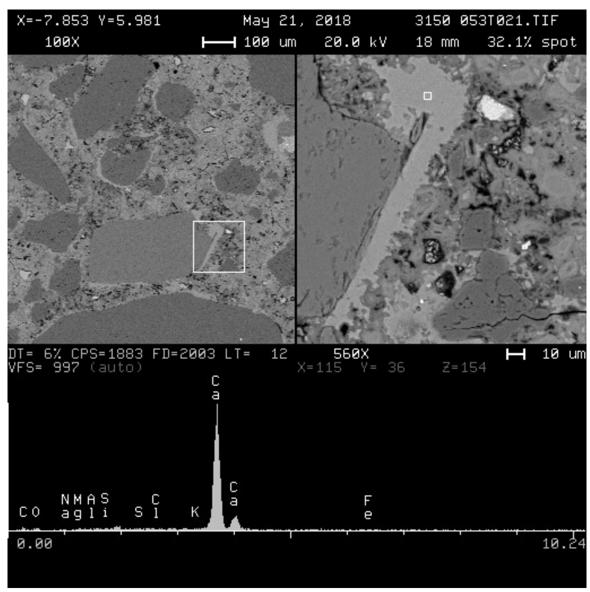


Figure 24. Pre-Cast Panel (3150053). BSE images with EDS spectrum of calcium hydroxide along the paste/aggregate interface.

Client ID: Column		RJLG ID: 3150054	
Sample description:	Concrete core		
Diameter	3 ⅝"	Length	2 ¼"

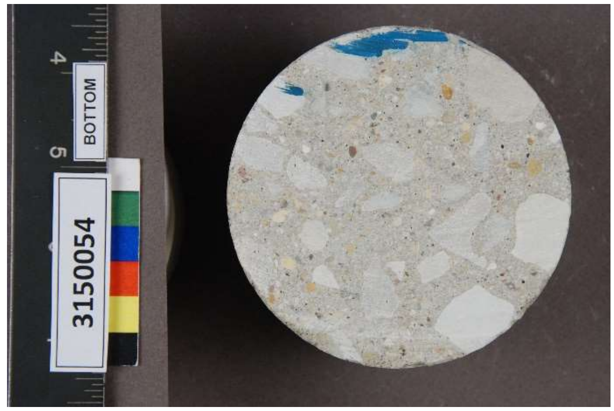
Summary of Petrography and Analytical Results					
Overall Condition	Estimated Air (%)	Estimated Paste (%)	Estimated w/cm ratio	Cement Type & Hydration	Reinforcement Cover
Good	3-5	28-30	0.40 ± 0.05	Well to moderately hydrated GGBF slag and well hydrated Portland cement	ND

Properties	
Coarse Aggregate Type	Crushed limestone with trace amount of sandstone
Maximum Size	½"
Gradation	Even
Shape	Sub-angular
Distribution	Good
Bond to Paste	Moderate
Fine aggregate type	Natural calcareous and siliceous sand
Air Void Type & Distribution	Unevenly distributed entrained air voids with trace amount of entrapped voids with the longest at ½".
Carbonation	Shallow carbonation up to $^{1}/_{16}$" deep with localized carbonation up to ½" along cracks.
Paste Color & Hardness	The paste was hard and Very Light Gray (Munsell N8) in color.
Cracks/Microcracks	• Shallow perpendicular microcracks along the surface carbonated along edge indicating early age up to $^{13}/_{16}$" deep. • One fine crack up to 1 ⅜" deep • Localized sub-parallel micro cracks from the chipped edge up to ¼" deep. • Trace amount of autogenous shrinkage microcracks throughout.

Observations
• The top surface was a smooth finished surface with chipping around the edges. • The bottom surface was cut.
• Few localized gaps at paste/aggregate interfaces consistent with trapped bleed water or poor compaction.
• Trace to minor amount of fine calcium hydroxide in the paste.
• Trace amounts of monosulfate and ettringite deposits in the paste.

Table 3. Petrographic Results for Column (3150054).

Top View



Bottom View

Side View

Side View

Figure 25. Column (3150054). Photographs of core in as-received condition.

Figure 26. Column (3150054). Photograph of freshly cut surface with phenolphthalein indicator applied.

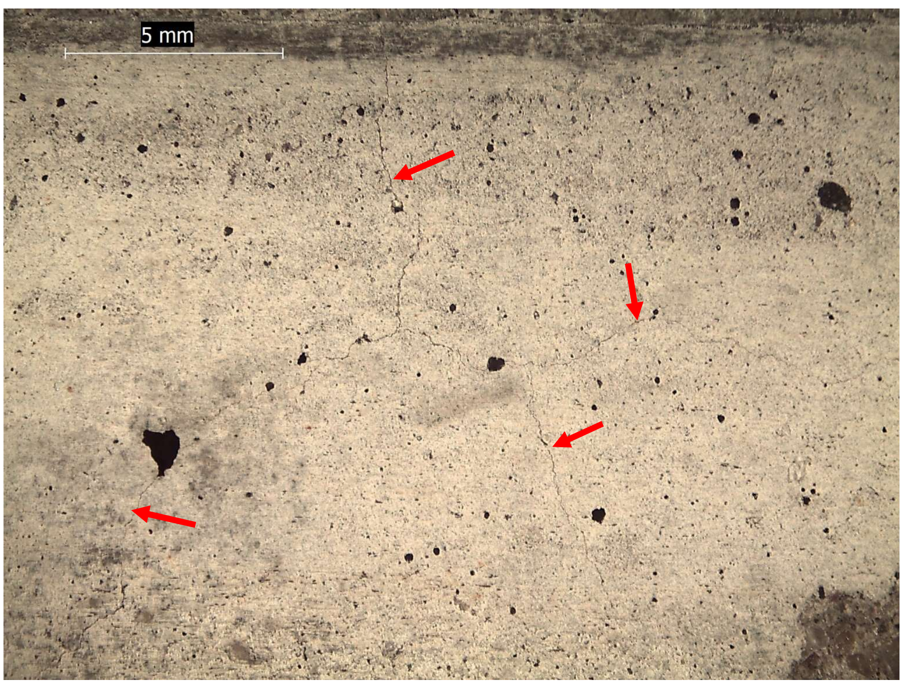
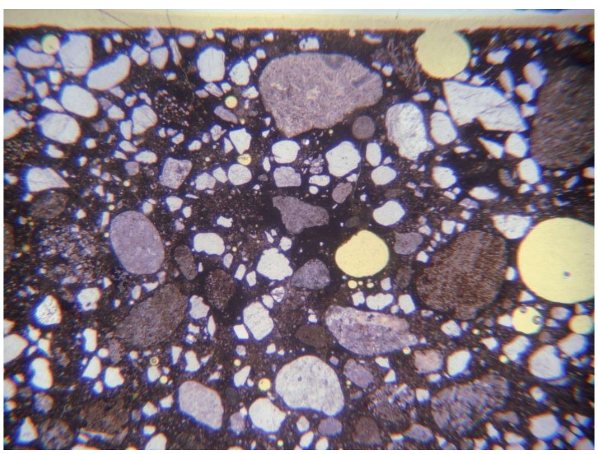


Figure 27. Column (3150054). Stereo-optical micrograph of the core side showing microcracks up to $^{13}/_{16}$" deep.

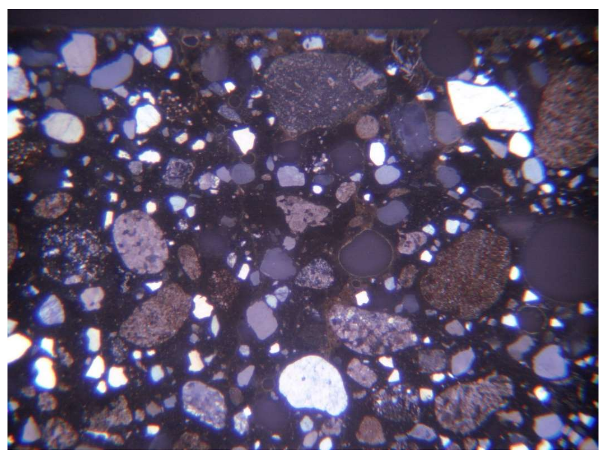
Figure 28. Column (3150054). Photograph of polished cross sectioned slab.

Figure 29. Column (3150054). Stereo-optical micrograph of the polished slab showing perpedicular microcracks consistent with drying shrinkage up to 3/16" deep.

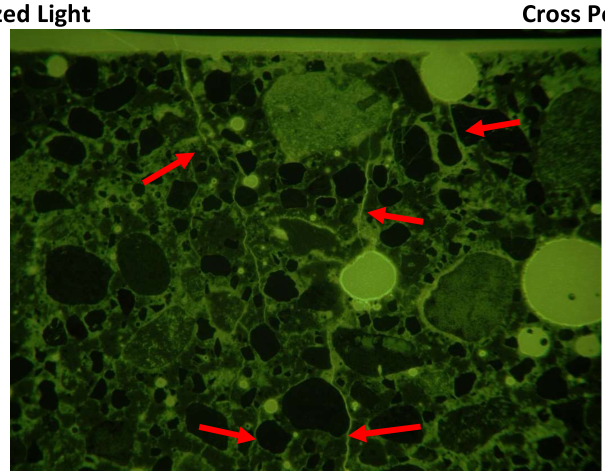
Figure 30. Column (3150054). Photograph of cross sectioned slab showing location of thin section preparation.



Plane Polarized Light



Cross Polarized Light



Fluorescent Light

Figure 31. Column (3150054). Optical micro-graphs in different light modes showing drying shrinkage micro cracks (highlighted by arrows) from the surface. Field of view 10.0 mm wide.

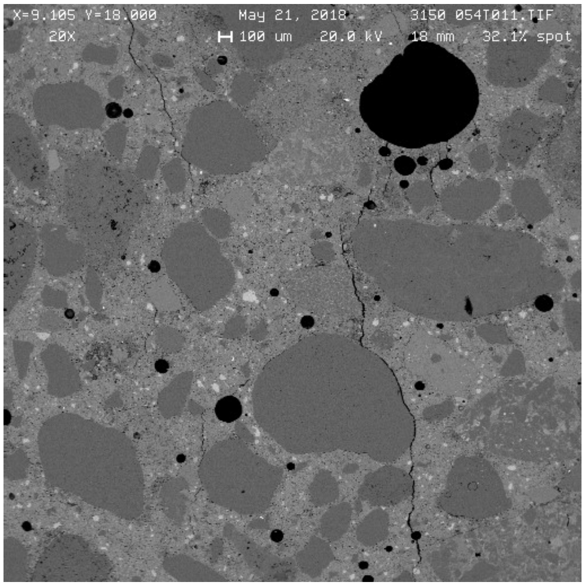


Figure 32. Column (3150054). BSE image of discontinuous microcracks near surface.

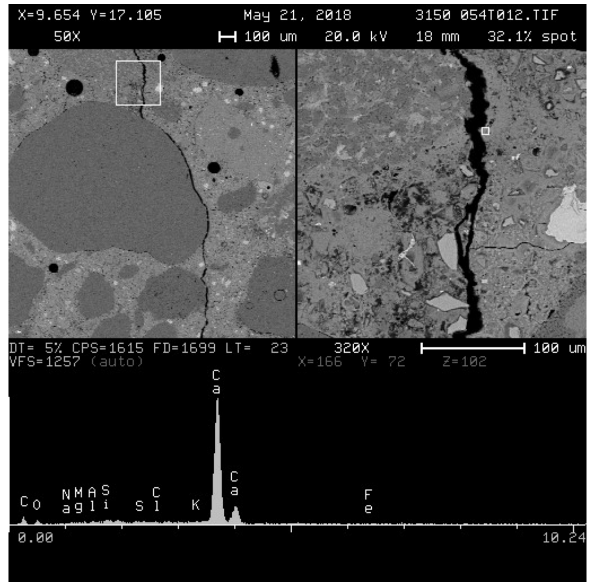


Figure 33. Column (3150054). BSE images with EDS spectrum of carbonated paste along the crack face to 3/16".

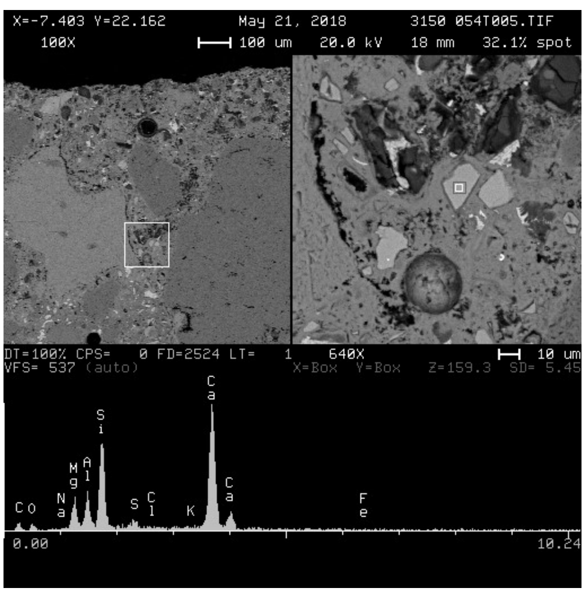


Figure 34. Column (3150054). BSE images with EDS spectrum of a partially hydrated GGBF slag grain.

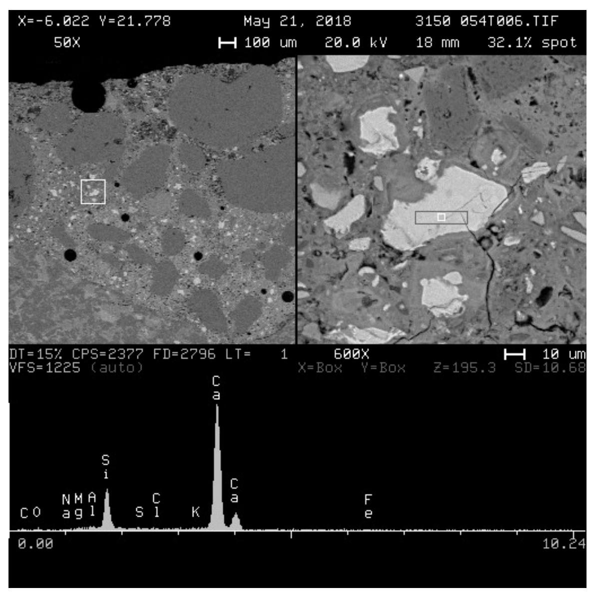


Figure 35. Column (3150054). BSE images with EDS spectrum of a partially hydrated Portland cement grain.

Client ID: Entrance Wall		RJLG ID: 3150055	
Sample description: Concrete core			
Diameter	3 $^{11}/_{16}$"	Length	2"

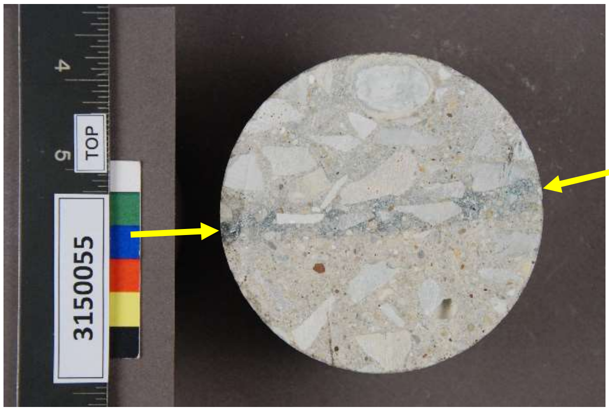
Summary of Petrography and Analytical Results					
Overall Condition	Estimated Air (%)	Estimated Paste (%)	Estimated w/cm ratio	Cement Type & Hydration	Reinforcement Cover
Good	3-5	27-28	0.40 ± 0.05	Well to moderately hydrated GGBF slag with well hydrated Portland cement	ND

Properties	
Coarse Aggregate Type	Crushed limestone and sandstone
Maximum Size	⅝"
Gradation	Even
Shape	Sub-angular
Distribution	Slightly uneven, although small area to evaluate
Bond to Paste	Good
Fine aggregate type	Natural siliceous and calcareous sand with trace amount of quartzite and opaques.
Air Void Type & Distribution	Uneven distribution of fine spherical voids consistent with entrained air with higher concentration on the right side, and a trace amount of entrapped air with the longest at ⅛".
Carbonation	Trace uneven carbonation less than $^{1}/_{32}$" deep.
Paste Color & Hardness	The paste was hard and graded in color from Yellowish Gray (Munsell 5Y 8/1) to Medium Gray (Munsell N5).
Cracks/Microcracks	• Trace to minor amount of perpendicular micro cracks up to 1 ¼" deep. • Trace to minor amount of autogenous shrinkage microcracks throughout paste.

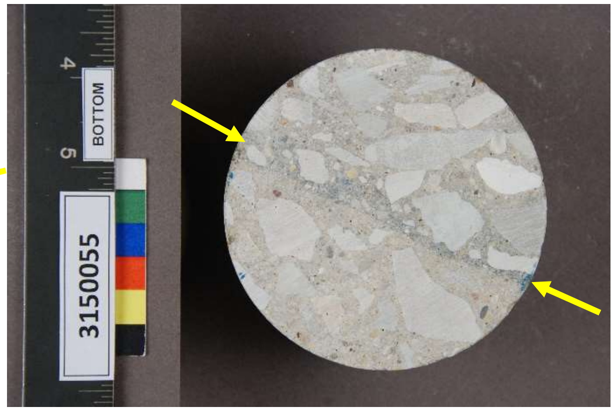
Observations

- Top and bottom of the core were cut.
- A cold joint was indicated bisecting the core. A narrow band of medium grey paste was present which was correlated with decreased hydration along the edge.
  - The medium gray paste belongs to a slighter denser paste with slightly reduced air content as compared to the concrete on the other side of it.
  - Slight retardation of hydration within medium gray paste approximately $^{1}/_{16}$" wide.
- Trace calcium hydroxide observed throughout section.
- Innocuous alkali silica reaction (ASR) within rim of a chert fine aggregate, but no cracking within the aggregate or into the paste.

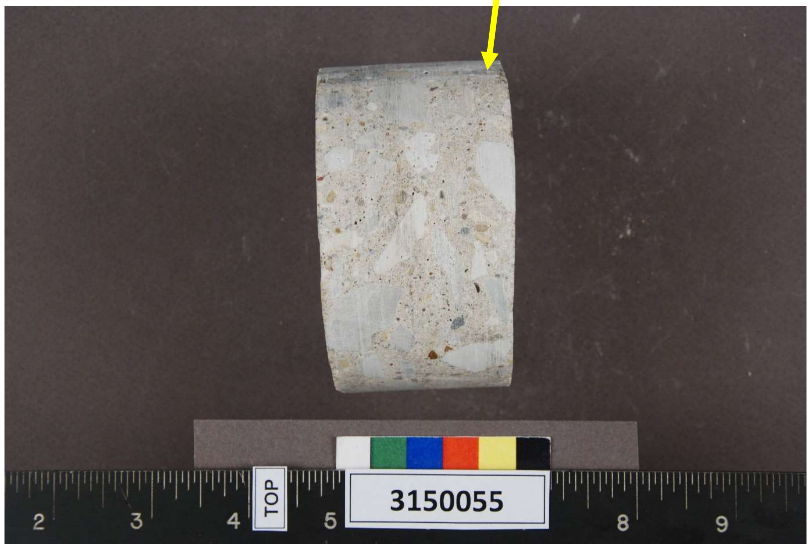
Table 4. Petrographic Results for Entrance Wall (3150055).



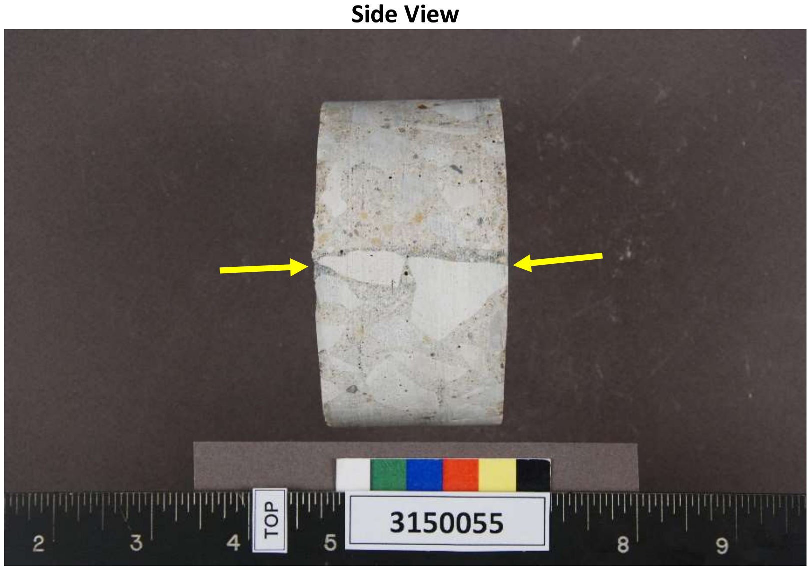
Top View



Bottom View



Side View



Side View

Figure 36. Entrance Wall (3150055). Photographs of core in as-received condition. A band of blue-gray paste (indicated by arrows) bisects the core.

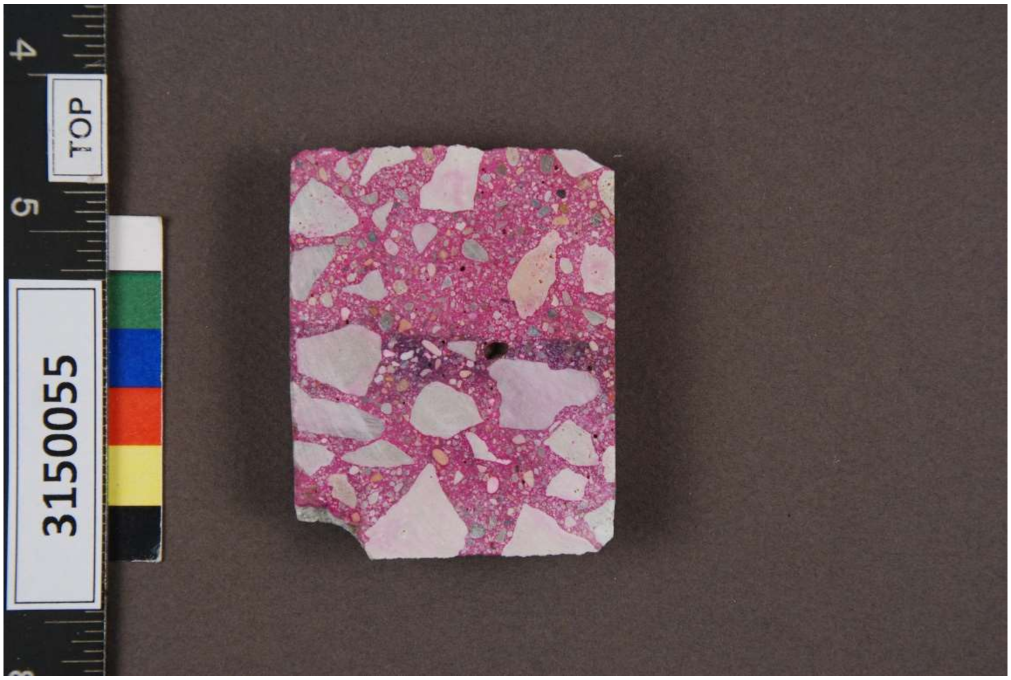


Figure 37. Entrance Wall (3150055). Photograph of freshly cut surface with phenolphthalein indicator applied.

Figure 38. Entrance Wall (3150055). Photograph of polished cross sectioned slab.

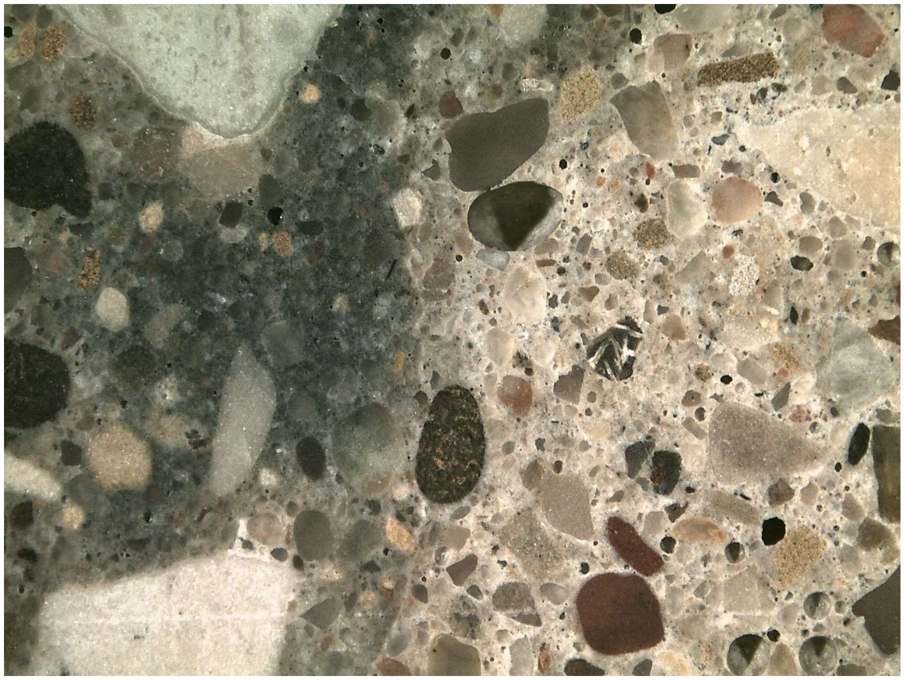


Figure 39. Entrance Wall (3150055). Stereo-optical micrograph showing the interface between concrete pours.

Figure 40. Entrance Wall (3150055). Photograph of cross sectioned slab showing location of thin section preparation.

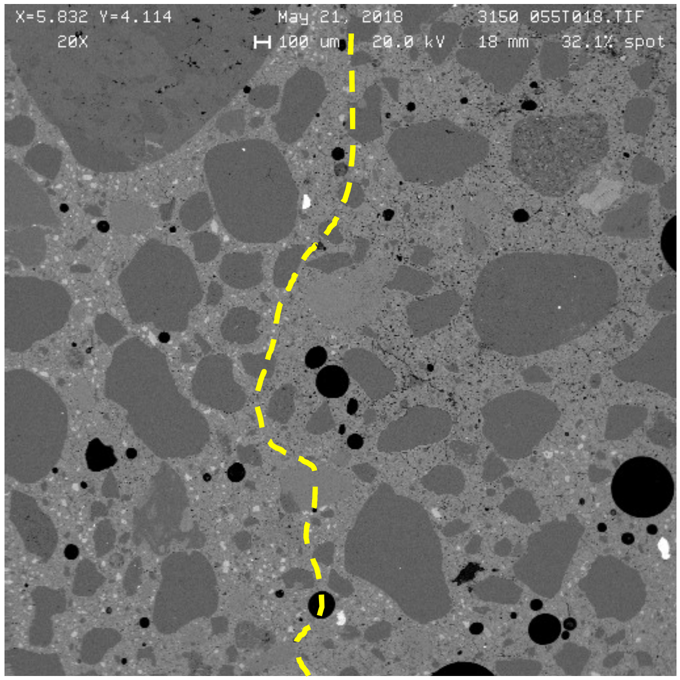


Figure 41. Entrance Wall (3150055). BSE image of the interface between the concrete pours.

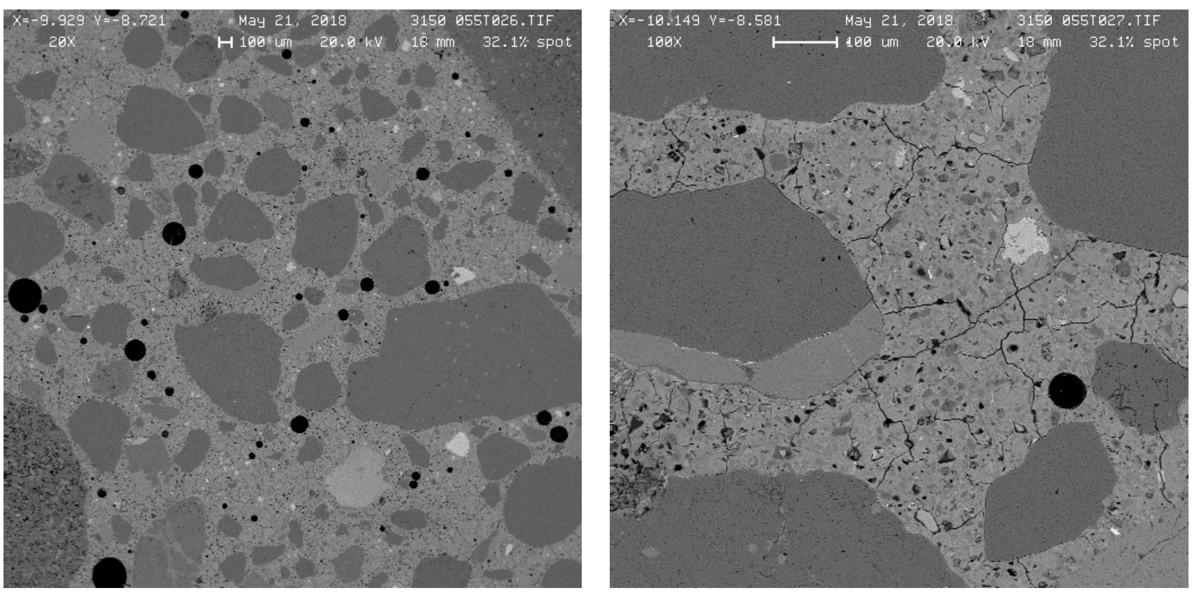


Figure 42. Entrance Wall (3150055). BSE images of the dense paste on one side of core.

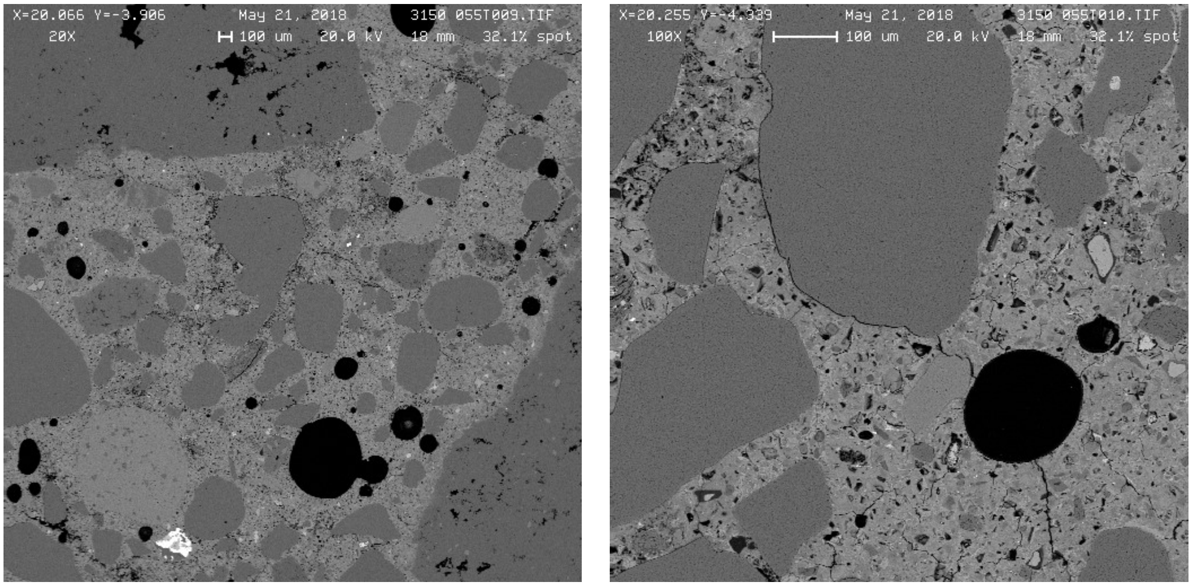


Figure 43. Entrance Wall (3150055). BSE images of slightly less dense paste on other side of core.

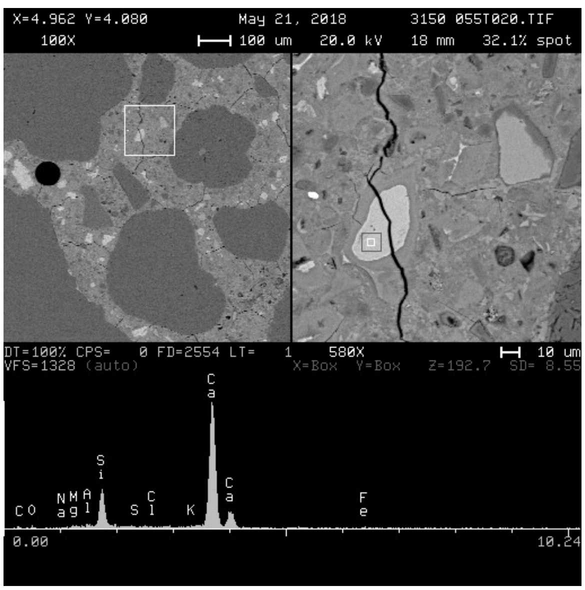


Figure 44. Entrance Wall (3150055). BSE images with EDS spectrum of a partially hydrated Portland cement grain.

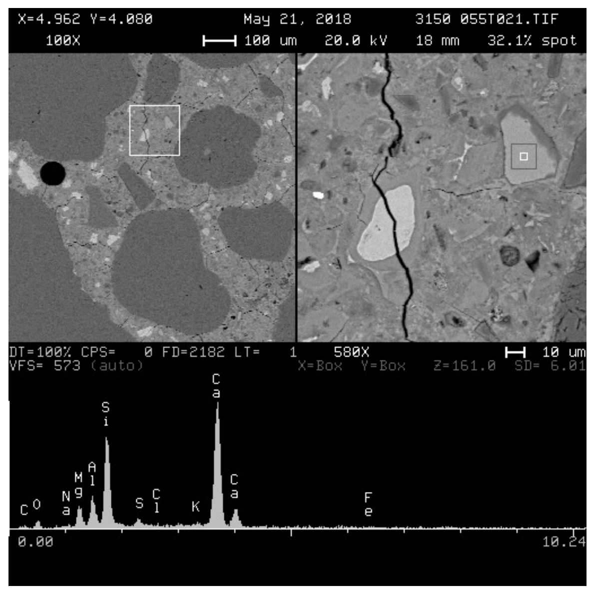


Figure 45. Entrance Wall (3150055). BSE images with EDS spectrum of a partially hydrated GGBF slag grain.

APPENDIX D: ASTM 1202

ASTM 1202
CHLORIDE RESISTANCE TEST REPORT

Project No.	Date	Document Ref. No.
I17258	05/03/2018	

Project Name:	Moser Tower Durability Analysis
Site Address:	443 Aurora Ave
City/State/Zip:	Naperville, IL 60540
Client:	Engineering Resource Associates

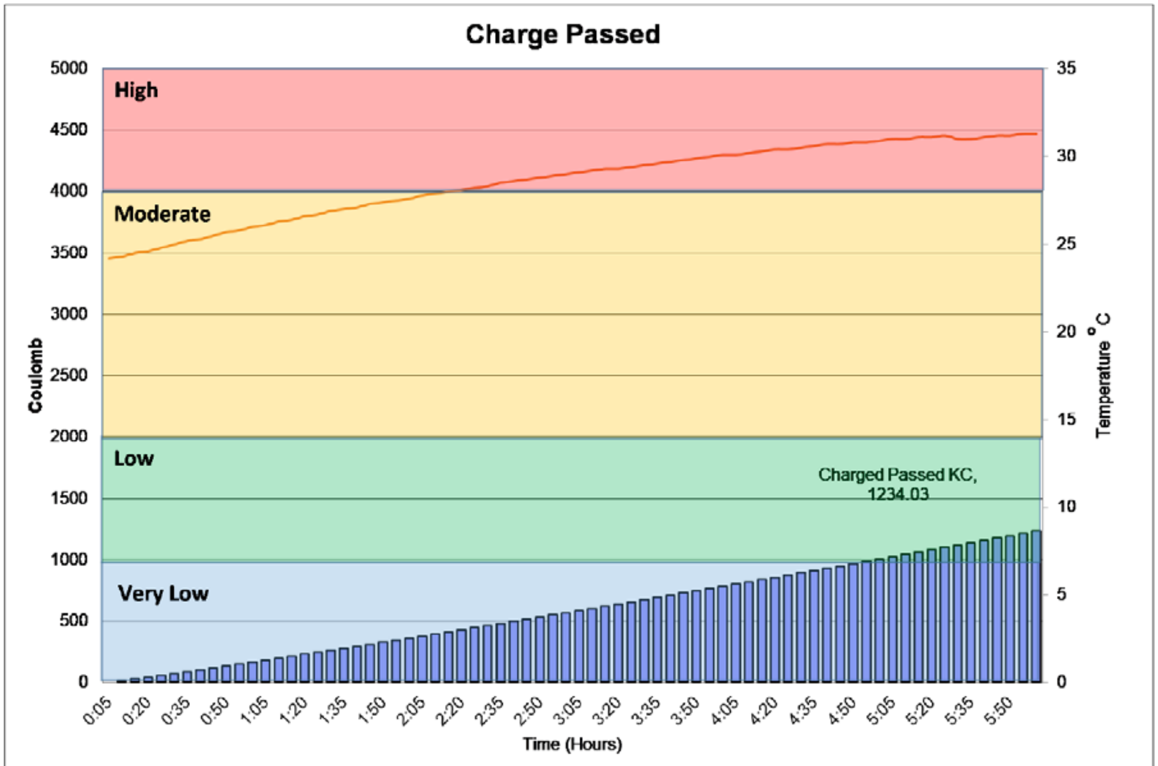
Lab Tech:	Clarissa Roe
Structure Type:	Building
Year Built:	2000

*See Procedure **PR-L01202** for Concrete's Resistivity to Chloride Ion Penetration details and how to use this form .

Core Reference:	1	Instrument No:	13404	Test Voltage:	60V	Sample Diameter:	100 mm	3.937	in.
		Channel No:	5	Test Time:	6 Hours	Sample Length:	50 mm	1.969	in.

Charge Passed (Coulombs)	Chloride Ion Penetrability
>4000	High
2000 - 4000	Moderate
1000 - 2000	Low
100 - 1000	Very Low
< 100	Negligible

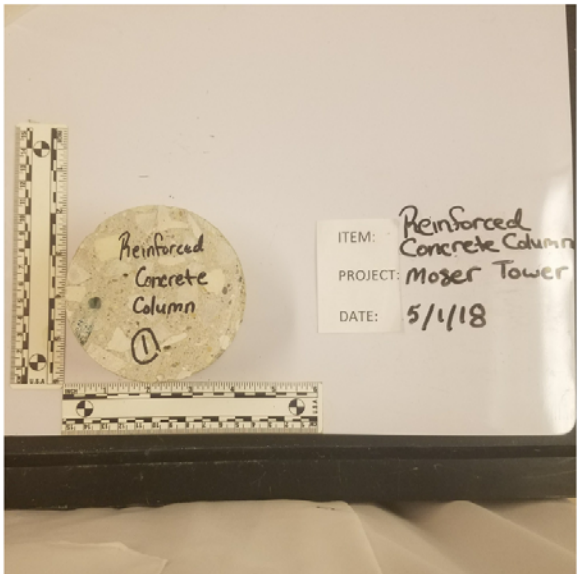
Charge Passed (adjusted):	1,234.03	Coulombs
Resistivity:	16,695	Ohms/cm^2
Penetrability Class:	Low	



Performance Chart

ASTM 1202
CHLORIDE RESISTANCE TEST REPORT

Project No.	Date	Document Ref. No.
I17258	05/03/2018	



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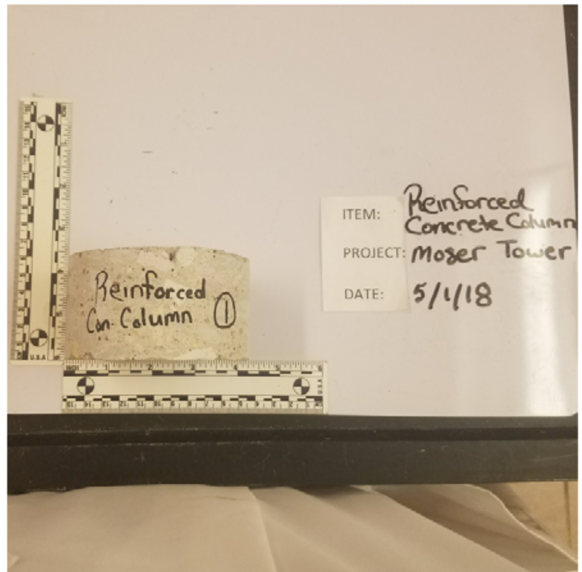


Core Side

Comments:

Authorized by: Clarissa Roe 2018.05.14 17:35:04 -04'00'

ASTM 1202
CHLORIDE RESISTANCE TEST REPORT

Project No.	Date	Document Ref. No.
I17258	06/04/2018	

Project Name:	Moser Tower Durability Analysis
Site Address:	443 Aurora Ave
City/State/Zip:	Naperville, IL 60540
Client:	Engineering Resource Associates

Lab Tech:	Clarissa Roe
Structure Type:	Building
Year Built:	2000

*See Procedure **PR-L01202** for Concrete's Resistivity to Chloride Ion Penetration details and how to use this form .

Core Reference: 3 | Instrument No: 13404 | Test Voltage: 60V | Sample Diameter: 92.075 mm | 3.625 in.

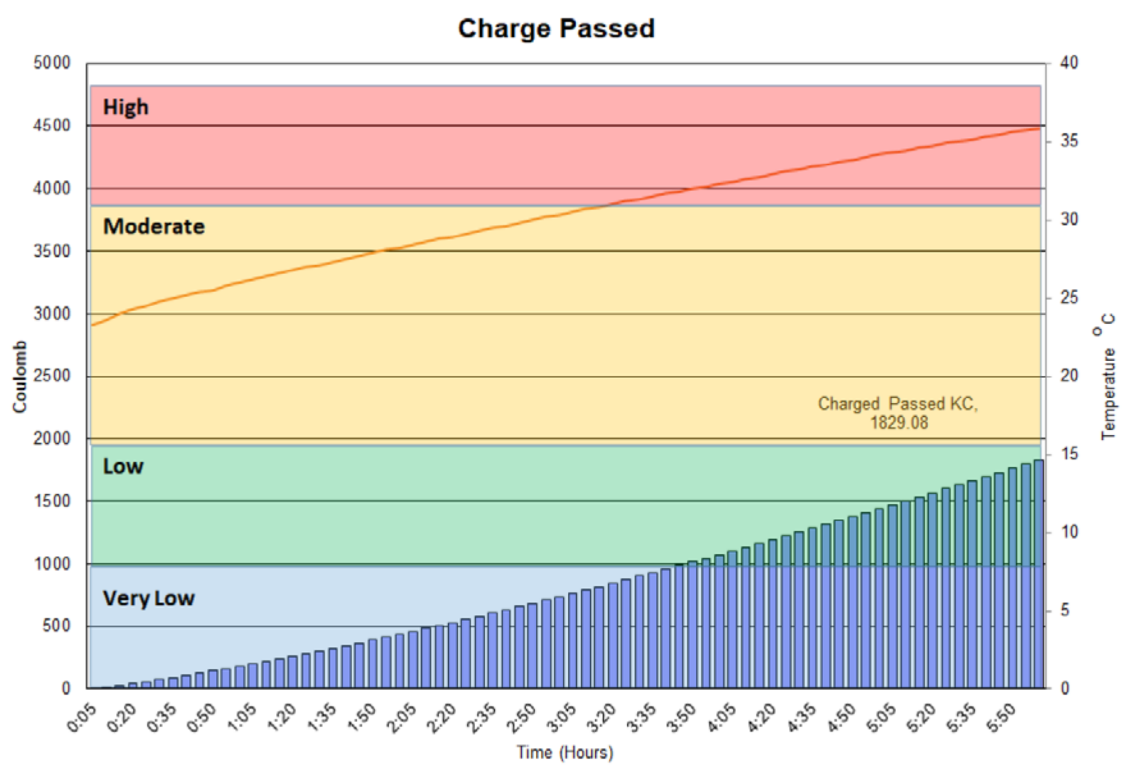
Channel No: 5 | Test Time: 6 Hours | Sample Length: 49.21 mm | 1.937 in.

Charge Passed (Coulombs)	Chloride Ion Penetrability
>4000	High
2000 - 4000	Moderate
1000 - 2000	Low
100 - 1000	Very Low
< 100	Negligible

Charge Passed (adjusted): 1,829.08 Coulombs

Resistivity: 11,264 Ohms/cm^2

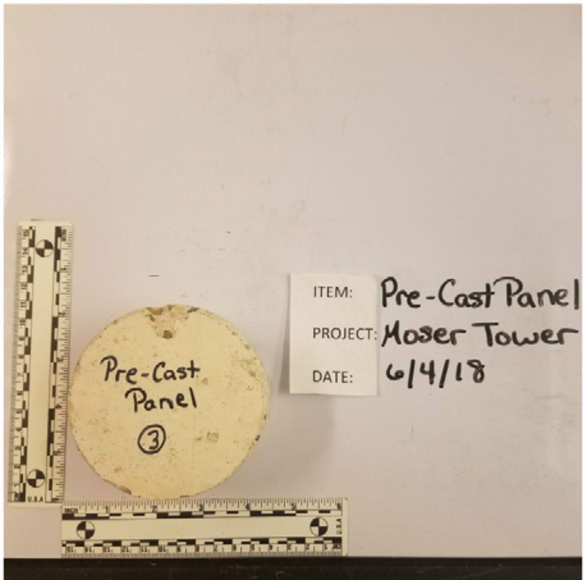
Penetrability Class: Low



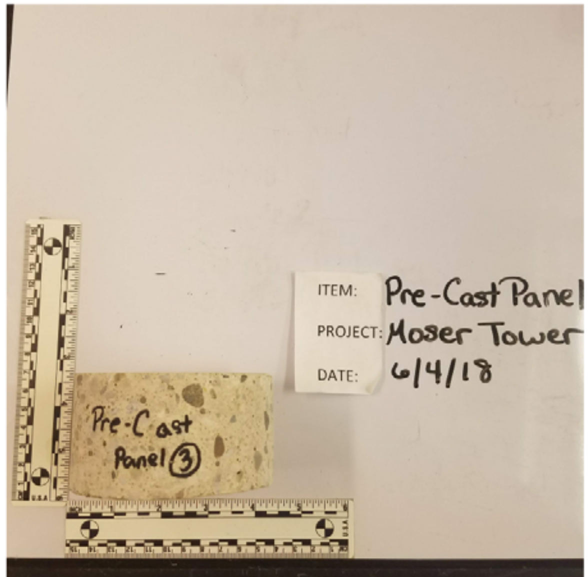
Performance Chart

ASTM 1202
CHLORIDE RESISTANCE TEST REPORT

Project No.	Date	Document Ref. No.
I17258	06/04/2018	



Core Top



Core Side

Comments:

Authorized by: Clarissa Roe 2018.06.07 09:24:13 -04'00'

APPENDIX E: BRUSH ARCHITECTS REPORT

Brush Architects

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Principal
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August 3, 2018

RE: MOSER TOWER / MILLENNIUM CARILLON
CONDITION ASSESSMENT REPORT
CITY OF NAPERVILLE
NAPERVILLE, IL

The City of Naperville retained Engineering Resource Associates, Inc. (ERA) Structural Engineers, ECHEM Consultants (ECHEM), and Brush Architects, LLC (BRUSH) to perform a comprehensive and holistic assessment of the condition of the Moser tower. Our team goal is to make recommendation for repairs and upgrades to ensure the long-term viability and serviceability of the tower and its components. ERA, as the lead consultant is focused on the structural viability of the Tower. ECHEM, as durability consultant is focused on the durability of the concrete and factors which impact the service life and longevity of the construction materials. BRUSH's task as the Architects within the team is to address the architectural and waterproofing/drainage concerns of the structure, as well as coordinate the recommended repairs into architecturally aesthetic solutions within the original design intent. The combination of the team's assessment and recommendations will provide a durable, usable, serviceable and aesthetically complete project.

Brush Architects' system of building assessment uses restoration principles of assessing original intent, maintenance issues, and mechanisms to which the building reacts in order to recommend solutions. Our component of the report establishes methodology of assessment, findings from our on-site examination, comparison of the existing conditions to the provided drawings, and recommendations based upon our findings, and the resulting prioritized repairs. Pending the acceptance of the report and an accepted work scope, the team will develop repair drawings and technical specifications which will allow for a professional and competitive bid process and construction services. Our goal is to maintain the structural, architectural, and material integrity of the Tower in keeping with original intent, project finances, improved conditions, long-term serviceability, and future use.

Executive Summary

The tower is in serviceable condition and is well within the realm of repair. Some suggestions as related to potential life safety are the highest priority, with many suggestions moving toward near to long-term maintenance items.

The architectural component of the investigation is summarized as follows:

1. The pattern and concentration of concrete patch failures at EL. 116′ and the underlying behavior of the structure present a life safety concern that requires further structural analysis and immediate repair. The patches, indicated on the building elevations, Appendix 02, graphically demonstrate the building dynamic of concern. Once the structural considerations are resolved, BRUSH will work with the team to design the architecturally appropriate solutions.

2. After the resolution of corroding steel components either by replacement of repair, they should be protected by durable, high-performance coatings and waterproofing adequate for the exposure and use.

3. The weather proofing of the building that by design is open to the elements is a futile effort, but the control of the impact of the weather is achievable. The control of the water paths and drainage of moisture from the building can be improved with appropriate membranes and improved water management.

4. The aesthetics of the concrete can be improved with the removal of the biological growth inherent on moist environments and consideration of anti-fungal/anti biological coatings for the concrete, all of which to be considered for effect on the concrete, the aesthetics of the concrete, and future re-application for maintenance concerns.

5. The ground floor and basement waterproofing, water management and drainage are inadequate. The resolution of these issues necessarily involves some reworking of the intersecting building components, both interior and exterior, for complete, functional and durable solutions.

Background

The Tower, designed by Vincent George Design Group, Inc. in 1999 was originally designed as a partially enclosed, weather protected space with glazing filling the openings in the lower part of the structure. In addition to the omission of the glazed enclosure, the existing construction varies from the original design in several notable aspects including the ground floor vestibule, layout and exterior cladding, roofing of the enclosure at the Belfry level and the steel stair design. The construction of the tower was interrupted due to lack of funding and then resumed several years later. During the interruption, the tower was reported to have stood open to the elements with a temporary wooden stair providing access to the interior. We also understand from anecdotal information that some of the precast concrete units remained in the precast fabricator's yard directly on the ground exposed to more extreme weather conditions than intended. The tower was finally completed in 2006 including the design changes by the architecture firm of Fujikawa Johnson Gobel Architects, Inc.

The Tower has experienced different modes of deterioration and failure over the years leading to some loss of use and ongoing maintenance issues. These include the delamination and failure of surface concrete at the joint at level 116’-8 5/8”, platform membrane failure and miscellaneous concrete cracks and spalls and the corrosion of atmospherically exposed steel. It is understood that the recent delamination and failure of the concrete initiated the Naperville Riverwalk Commission in retaining ERA for their study of the tower.

Methodology

The report has been organized by methodology, findings and observations, recommendations, photo documentation, and graphic diagrams to convey patterns of deterioration. The report is meant to be viewed in compliment to the reports by ERA and ECHEM.

Prior to beginning the project BRUSH walked through the tower on two occasions to review the general conditions and develop an understanding of the scope of the work. We then performed a hands-on and visual investigation of the structure on March 26-29, 2018 coordinated with on-site time with ECHEM and assisted by Golf Construction for access. Our hands-on inspection of the interior was conducted from the landings and stairs of the tower. The exterior inspection was performed from a suspended swing stage platform provided and operated by Golf Construction.

The following documents were provided to BRUSH for review as part of the investigation:

- ERA Preliminary Assessment Report
- Original Architectural Drawings
- Precast Shop Drawings (without the architect’s review comments)
- Precast Shop Tickets

The architectural component of the investigation includes the concrete structure, concrete textures and finishes, condition of the steel components as visible, condition of the exposed steel to concrete connections, stairs, composite deck landings, traffic coatings, sealants, glazing, and waterproofing. Each of these elements contribute to the overall durability and performance of the tower. The prioritization of the investigation focused on architectural components of the structure. Our observations address the exterior and interior surfaces of the structure as exposed for visual observation. By interior it is to be understood that it is the interior elevations of the concrete as viewed from the stairwell and landings. Exterior is the view of surfaces as seen from the exterior of the building. ECHEM and ERA may have observations that address the interior of a concrete unit, which is beyond the architectural defined work scope. The following are the architectural components and conditions that are the focus of this report.

1. Critical primary concrete structure and supporting steel
    a. Exterior of the tower limited to two drops accessed by suspended swing stage
    b. Interior faces and components as viewed from the interior stairs and landings.
    c. Visible connections of the concrete to the exposed steel
2. Secondary steel components, supporting stairs and landings
3. Architectural finishes and condition of the concrete and steel substructure
4. Visible Corrosion of metal components including their impact upon the concrete

5. Water management and moisture and drainage patterns of the tower
6. Comparison of existing conditions to the provided documents as viewed within the architectural definition of our work scope
7. Concrete aesthetics of pour patterns and finishes which will impact repair materials, finishes, and overall aesthetics

The project intentionally overlaps specialized skills of the team members with the goal to provide complimentary solutions each as necessary with a structural, architectural, and material based recommendations. The following is a list of our areas of responsibility as assessed by the architectural team.

Priority	Description of the Architectural role with the following components	Observations
01	Concrete Foundation Walls and base (poured-in-place) surfaces	Architectural mapping of textures and surfaces
02	Coated exposed Steel structure (stair and landing)	Condition of the metals and their protective coatings, areas of drainage or ponding
03	Precast Concrete Units (reinforced and post-tensioned)	surfaces, crack mapping, spall mapping as visible.
04	Precast Reinforcing	none
05	Precast Post-Tensioning System	none
06	Connections (precast embed to steel frame)	Exposed portions of connections visually reviewed for corrosion or moisture accumulation
07	Connections (precast embed to steel compression rings)	Exposed portions of connections visually reviewed for corrosion or moisture accumulation
08	Connection protection (sealant/flashing)	The condition of existing sealants and flashing, and areas where improvements are recommended.
09	Composite Deck Platforms	The top surfaces, underside, and edges where visually available.
10	Composite deck water management and waterproofing	The areas of current moisture retention, previous ponding, and discoloration where visually available.
11	Steel stair and railing connections and condition	The surfaces, condition of the metals and coatings, underside and edges where visually available.

12	Steel stair tread waterproofing and traffic coating	The surfaces, condition of the metals and coatings, underside and edges where visually available.
13	Basement water management and moisture controls	The condition of the central open Basement level where drainage patterns and debris accumulation were visually available. We did not excavate foundation waterproofing or explore sub-grade sub slab water drainage or protection conditions.
	Basement moisture controls	The condition of the basement roof slab with current and historic water infiltration patterns where visually available.
14	Stone veneer sealant and drainage	The surfaces and condition of the veneer and flashing and sealant where visually available.
15	Window wall drainage and sealant	The glazing and sealant surfaces where visually available.
16	Platform wear slab grading	The wear slab over the basement and adjoining the structure. Corroded and damaged railings and stairs were not included in the review.

Observations and Recommendations

The following observations are organized by material type will follow the order of location for the material type and then the observations and findings per the priorities as established above.

I. Cast-In-Place Concrete Foundation

Location: below grade foundation walls and roof slabs in the basement, and ground floor up to El. 11'-0".

Observations

The poured in place concrete portion of the structure that were visible for inspection, includes the basement foundation walls, ceilings, beams and columns as viewed from the basement, and the above grade base of the Tower which extends to 11"-0" above grade exposed from the interior **(Photos 3-12).** The exterior side of the above grade concrete is clad with a deep-set mortar jointed lannon-stone limestone coursing pattern veneer that is capped with Indiana limestone coping stones. The exposed interior side of the structure including foundation walls, columns and beams have a smooth formwork finish. The exposed concrete walls at the basement level have an exposed aggregate sand blasted finish **(Photo 6)**.

The concrete was observed to be in serviceable condition with little indication of significant deterioration. Minimal cracking and deterioration were observed except as noted below. Beginning at the basement level, we observed cracks in the concrete ceiling of the basement indicating long term water infiltration and mineral leaching **(Photos 11-16).** This condition occurs primarily under the main entryway, exterior slab and at the window line. Water infiltration through the ground level slab on the interior and exterior has caused corrosion of electrical conduits and junction boxes.

Repeated exposure to precipitation and the high moisture content of the building's environment has caused surface discoloration from both biologic growth and corrosion at steel embeds and connections.

Some hairline cracks were observed in the lower level exterior foundation wall, but no water infiltration. This area is exposed to the weather and is subject to rain and snow exposure. The interior ground floor concrete slab is connected by steel framing and composite deck and has a sloped topping slab with a failed traffic coating which is addressed later in this report.

The concrete in the enclosed Basement spaces including the exit hallway, mechanical room, and bathroom appears to be in good condition, except for numerous hairline cracks with evidence of long-term water infiltration ceiling and walls of the mechanical room **(Photos 14-17)**. Significant corrosion of electrical conduit and junction boxes, and the ceiling light fixture was observed. Since no liquid water was observed, it is not clear if these are still active leaks. We noted that the exterior slab on the East side of the Tower has been replaced and these leaks may have been repaired. However, based on our observations, cracks that bridge between the exterior slab and interior have not have been resolved. The waterproofing of these areas is addressed elsewhere in this report.

II. Precast Concrete Units

Location: The precast concrete units that make up the sculptural components of the tower begin at the El. 11'-0" above grade and bear on the cast in place concrete foundation.

Observations

The precast concrete consists of the factory formed units that make up the tower in the form of the piers and the vertical decorative fins between the piers. The precast units were factory formed in a controlled environment and cast into a form with a liner that provided the surface texture and allowed removal from the form. The architectural design of the piers consists of the interplay between the rougher layered slate textured finish and a smooth finish for the more detailed aspects. The strategic locations of the varied finishes accentuate the sculptural form of the tower and is an important architectural component for consideration throughout the maintenance and repair campaign.

While the main precast units appear to follow the original design, we noted that the precast fins have been omitted above El. 112'-0" and replaced with cell phone transmission antennas, changing the architectural appearance of the tower. The antennas are supported by a secondary steel structure specifically designed for the antennas but not designed in keeping with the tower aesthetics.

Starting at an elevation EL. 11'-0" above exterior grade, the precast units are stacked vertically on the cast-in-place concrete bases to form the primary structural piers of the tower. Their design provides vertical and lateral support of the structure, supplemented with internal structural steel compression rings **(Photo 18)**. Midway up the tower at El. 52'-2", there are two large W33x169 wide flange steel beams**,** attached to steel embeds in the precast concrete **(Photos 19-20)** that transfer the load of the stairs, carillon control room and bells above into the structure. The stairs below the Gallery are hung from these beams with lateral support provided by steel brackets and channel embeds in the precast **(Photos 21-23)**. There are 8 steel tube compression rings spaced intervals for the height of the structure. Their purpose is to provide stabilization of the precast panels and fins as well as vertical support for the stairs and landings. The connections of the compression rings to the precast units are addresses later in this report.

The architectural review for the precast concrete was the aesthetic and design components. We also identified areas of deterioration, cracks and spalls to determine patterns of behavior, so that the repair design addresses the causes of deterioration to establish priorities in repairs to replicate the existing surface characteristics.

A. Finishes

The finishes and surface texture of the precast units varies by orientation. The most decorative slate texture occurs on the outside faces and sides of the 4 main piers **(Photos 24-27).** Based on the shop drawings, the texture and reveals were produced using reusable plastic form liners. Some elements of the piers have a smooth finish to support the architectural design. The units have been cast so that the exposed surfaces are formed, and the unfinished surfaces concealed in the joints. The exposed horizontal ledges have rough, unformed surfaces.

B. Cracks and Spalls

Spalls -

Concrete spalls and insipient spalls were found to be most significant at level El. 116 related to the particular conditions at that elevation. Smaller spalls in the main piers are located at random locations, mostly adjacent to joints or steel embeds. Additional smaller spalls were observed where reinforcing bars are placed close to the face of the concrete. A systemic and discernable pattern of cracks and spalls was not observed in main precast piers. However, we noted several modes of failure in the precast concrete fins. These include spalls at the narrow decorative edges and at precast embeds.

Cracks -

Deterioration of the concrete due to combinations of deficiencies in the exterior surfacing in the form of cracks and spalls allows moisture to penetrate the surface and wet the internal surfaces and reinforcing.

1. Precast Units –

   a. Roof Panel Edge cracks - Crack patterns at the sharp edges roof panels are of concern. They have not yet connected to form spalls, but the regular patterning denotes that residual moisture remains in the concrete possibly in relation to the interior steel armature **(Photos 28-30).** There is no drip mold at the edge of the concrete which would force the water to fall from the edge. The sloped design directs water along the edge of the concrete unit. There are some small isolated spalls where the reinforcing is placed too close to the surface **(Photos 31-34)**. Some spalls at reinforcing appear to have been previously patched.

   b. Cracks and Water Infiltration –

   Water infiltration through surface cracks and mineral leaching was observed at two locations. The most severe condition was observed at the Northwest pier at the Observation deck level **(Photos 63-66)**, where there is significant water infiltration through cracks and the attendant mineral leaching. There is also a marked deterioration of the surface of the concrete where the ring beam bears on the precast concrete. This condition may be caused by water infiltration at the adjacent horizontal ledge which was not accessible during our investigation. The perimeter of the ring beam is fully sealed at this location which indicates water infiltration from the exterior face.

Recommendations

The exterior ledges should be examined for water infiltration and the sealant should be partially removed from the ring beam for further investigation. Once the source of the infiltration is sealed, the mineral deposits and loose concrete should be removed, and the concrete patched. A drainage opening should be installed below the ring beam to allow any future water infiltration to escape. The corroded surfaces of the ring beam should be recoated.

2. Precast Fins –

The narrow vertical precast fins between the main piers **(Photos 39-42)** are connected to the compression rings via steel embeds and welded clip angles **(Photo 43-44)**. The finish on these panels is a smooth finish with a sanded texture **(Photos 45-46)**. The color of the panels is a lighter than main piers. the One side of each panel has 4-6 small patches from the removed lift points, which do not match the parent concrete. The panels display several types of damage including cracks, spalls, and damage as described below.

a. Steel Embed Damage - The panels are attached to the ring beams via steel embeds at the top and bottom corners of the panels **(Photos 43-44)**.The proximity of the steel embeds to the surface of the concrete has caused local spalling and hairline cracks. Some of these conditions have been previously patched.

Recommendations

The failed panel corners due to stresses at steel embeds can be addressed by excising the failed ridges adjacent to embeds. This is not visible from the exterior and would have little impact on the aesthetics as experienced from the interior of the tower. Otherwise, each damaged location can be patched to match the existing, using the appropriate material and reinforcing. The steel embeds, and clip angles should be treated in conjunction with the concrete repairs as indicated later in this report.

b. Narrow Edge Damage - The edges of the fins have two narrow decorative ridges approximately 1.5″ x 2″ along each edge. The narrow proportion of these ridges make them vulnerable to damage either during installation or by ongoing thermal stress in the panels. There are numerous insipient spalls and patched spalls in the ridges **(Photo 39).** Some of the patches match the parent panel well. The original design and the shop drawings show the fins topping out at El. 142′, but the top section has been omitted and cell phone antennas put in their place **(Photos 47-48).**

Recommendations

The narrow ridges appear to be stable but will probably continue to spall over time due to their vulnerable geometry. These ridges should be sounded for insipient spalls and all spalls removed and patched with a set repair design including carefully applied reinforcing and patch material. Another solution to prevent ongoing failure may be to excising part of each ridge to make them less vulnerable to damage.

c. Hairline Stress Cracks – Some of the panels as indicated in the attached diagrams, have either isolated or a series of horizontal or diagonal hairline cracks **(Photos 35-36).**

Recommendations

The treatment of the stress cracks should be selected after the cause of the cracks are determined and the stresses relieved or resolved. We do not recommend that these cracks be routed and sealed as this will be aesthetically unacceptable and may cause unintended damage. The appropriate solutions will be determined by the team based on a holistic understanding of the concrete matrix, structural cause of failure and aesthetic considerations. The goal of the repairs is to prevent water infiltration into the cracks while not trapping moisture at the cracks.

d. Failed Joint Grout - The surface grout between the panels has failed at many locations and has delaminated from the joint. This allows additional water penetration into the joint, exacerbating the freeze-thaw damage.

Recommendations

These joints were not accessible for close-hand inspection during our investigation and further examination of the joints is required. The failure may be related to the overall stresses on the panels and the correct solution will involve an understanding of the stresses on these joints. However, it is likely that the failed grout should be ground out and re-grouted or sealant installed.

e. Lift Point Patches – Each panel has 5-7 lift point patches on one side from where they were lifted into place **(Photos 41-42, 46).** The lift cables were detached, and the openings patched with material that does not match the color or finish of the panels. We did not observe any failure of these patches and this is primarily an aesthetic issue.

Recommendations

A vapor transparent mineral coating may be applied to the patches to better match the host panel. This solution should be considered in conjunction with any other treatment of the panels that may be decided.

C. Patches

1. Large Failed patches at El. 116′-8″

There are a number of significant areas of concrete failure at precast joint EL. 116′-8″. At the one location observed at close hand during the investigation we requested that the contractor remove delaminated and hazardous sections of failed concrete and patches **(Photos 53-56).** Similar patches were observed at other locations at the same level as indicated on the attached diagrams. These patches appear to have been coated ostensibly to match the color of the parent concrete. The patches were installed without leaving a joint between units which may have contributed to their subsequent failure. The removed patches that were observed were not anchored per industry standards with stainless-steel rods for patches over 3″. Additionally, standards for structural, load bearing patches have different requirements to transfer the required loads without failure. Patch installation should take into consideration the finish of the host material, to match finish patterns and colors of the host concrete.

The removal of the failed patches exposed the mild steel reinforcing in good condition as well as the end of the dead-end plate of the post-tensioned bar for the precast units below **(Photo 55).** The systemic failure at this elevation appears to be related to the proximity of the post-tensioning hardware to the surface of the concrete due in large part to the narrower geometry of the units. This one area was quickly re-patched while the swing stage was mobilized.

Stainless-steel threaded rod reinforcing in chemical anchors was installed by the contractor. The color, aggregate and surface texture of the new patches were not designed to match the host concrete **(Photos 61-62).**

Failed patch with post-tension dead end with anchor stressing assembly. Note new stainless-steel patch reinforcing.

Recommendations

Based on the architectural survey of the pattern of deterioration at this specific elevation, it is our opinion that the concrete failure is primarily of a structural nature to be addressed by ERA. The repairs should address structural issues in conjunction with the durability and aesthetic considerations. We noted that the post tensioned dead-end assembly does not match the shop drawings which may have a bearing on the failure. Additionally, the reinforcing configuration and quantity around the PT dead-end should be reviewed for common practice standards.

Any patches whether they are load bearing or not, must match the host material in color, texture and form as determined through test mock-ups and color sample development. Tertiary issues include proximity of the PT dead-end to the surface of the concrete. Expansion and contraction of the embedded steel should be addressed during patching. All patch material should be colored and textured to match the color of the cleaned parent material. The areas to be patched should be sawcut at right angles with the appropriate depth and surface texture for the required bond.

2. Miscellaneous Patching

Other patches include those to cover aspects of the construction such as to fill lift points and to cover exposed clip angle attachments. The lift point patches are most exposed on one side of the narrow fins. Patches that cover clip angles are shallow and most have failed or developed hairline cracks **(Photos 57-60).** Some of the clip angle patches are sealed with sealant which can trap moisture, accelerating corrosion of the steel clip angle. Various patches were also observed adjacent to fin embed plates.

Recommendations

In general, all patch material should be formulated to match the parent concrete as closely as possible in color and texture to minimize the aesthetic impact of the patches especially on highly visible areas. Patches of the stone slate finish on the exterior facing surfaces of the precast should match the adjacent texture. Large patches that require form and pour installation should have a form liner that approximates the original. Smaller patches that can be hand patched should be hand tooled and to mimic the original the original texture. The design of patches, depending on whether purely aesthetic or structural should follow predetermined guidelines for cleaning and coating exposed reinforcing, orthogonal geometry, saw-cutting edges, minimum depth, surface magnitude, depth below exposed reinforcing, surface saturation, priming and curing. Where applicable, the patch should be extended to the nearest reveal or joint.

Patching of clip angles, which all occur on the interior faces are not durable and tend to trap water against the steel and are not advisable. These patches should be removed, the steel cleaned and coated with a high-performance coating per the recommendations elsewhere in this report. As mentioned, horizontal clip angle connections should be protected with a fluid applied roofing material as with other horizontal surfaces.

3. Examples of Previous Maintenance Efforts

The maintenance efforts have had varied success which we will discuss so that the next round of repairs considers our observations. The patches that have been installed do not appear to have been designed to match the parent concrete in color or materiality. Patches fall into several categories, including:

- Patches to repair concrete failures and spalls
- patches to cover aspects of the installation, such and lift points and embed plates
- patches of damage that may have occurred before installation

III. Structural Steel

Location: The structural steel is located at all elevations of the structure as indicated.

Observations

This section addresses the architectural coatings and corrosion that has developed or may develop from failed coatings, inadequate waterproofing and improper water management. Much of the corrosion observed stems from the fact that what was intended to be weather protected has been exposed to the weather and significant precipitation. In addition, a sufficient water management and drainage strategy was not factored into the areas that were originally designed to be interior. The protection of critical structural connections, both exposed and concealed are also addressed in this section.

An important observation is that that the steel structure below El. 70′ is not galvanized as it was originally designed to be within the weather protected enclosure. The steel above this level is appropriately galvanized. The exception to this rule is that the stairs are not galvanized and suffer systemic corrosion.

A. Steel Framing and Components

This section contains our observations and recommendations for the repair of all exposed steel in the tower, including structural steel, secondary steel, railings, stairs, and steel to precast connections. Each condition presents different mechanisms and levels of corrosion requiring solutions that are designed to address each condition. In all cases, the exposure to water and the failure of protective coatings present the risk to long term durability. Since the lower part of the tower, designed to be weather protected has been exposed to the weather since it was constructed, patterns of corrosion and associated failure have become evident and should be addressed accordingly.

1. Coated steel framing (below EL. 70′-0″)

The internal structural framing is comprised of wide flange steel members that begin at the Gallery level where two large beams are bolted to large embeds in the precast concrete. The steel framing above, bears on these beams and the stairs below are hung from these beams via steel angle hangers. Lateral stability for the base of the tower is provided by a steel tube braced frame concealed within the elevator shaft. We observed isolated surface corrosion of the exposed structural steel, mostly at connections, field welds and some horizontal flanges. The braced frame inside the elevator shaft was not observed and its condition and coating is unknown.

Recommendations

The recommendations for the protection of exposed structural steel includes cleaning and repainting corroded welds and surface corrosion. Areas of coating failure due to underlying corrosion, blistering and flaking should be mechanically cleaned down to base metal and immediately recoated with the appropriate zinc-rich primer and then recoated with a high-performance paint system to match the existing color. Steel that is newly exposed due to the removal of abandoned slab edges or corroded decking assemblies should be prepared and recoated accordingly. It is advisable to establish a baseline of protection for all the steel, it is recommended that all the steel or at least distinct sections or member be recoated at this time, including the appropriate surface preparation as recommended for the new high-performance coating system.

Steel members that require replacement during repairs such as corroded slab edge angles should be coated with a high-performance coating designed for exterior metals, hot-dipped galvanized. Based on an analysis of the galvanic assemblies and moisture exposure, stainless steel components may be considered. To the extent possible, galvanized assemblies should be shop fabricated, assembled, and shop primed and coated before installation. Corrosion found on the exposed galvanized steel where the galvanization has been damaged should be repaired per specifications to be written with the construction documents.

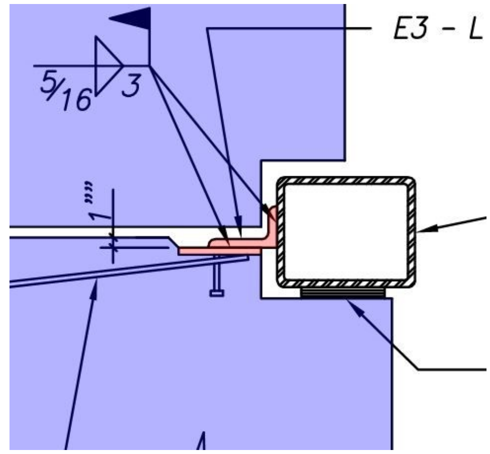
2. Galvanized steel framing (above EL. 70'-0")

The structural framing and compression rings above El. 70'-0" are hot-dipped galvanized per the design documents. The galvanized ring beams are coated to match the steel below and the internal galvanized framing is uncoated. The framing braces the tower, supports the stairs, the carillon cabin, the bells and the observation deck. We observed that the steel is in good condition with corrosion isolated to field welds and connections where the galvanized coating has been damaged. Bolts and field welds appear to have been sprayed with zinc-rich coating. Secondary framing supporting the carillon mechanism and bells is also galvanized steel.

At the Belfry level (EL. 72'-9") there is galvanized secondary framing and decking originally designed as the roof of the stair opening in what was to be the roof of the enclosed portion of the structure **(Photos 67-68).** This structure contributes to the lack of drainage at the Belfry level, is now obsolete and should be modified for improved drainage or removed.

3. Compression Ring Connections

There are 8 steel tube (TS 10x8) compression rings over the height of the structure. The ring beams are integral to the precast structure and also serve to support the composite decks. These appear to be in good condition with minimal indication of corrosion. It is understood from observations and the documents that the rings above El. 70' are galvanized. Each ring is connected to the precast units by steel clip angles welded to the ring and to steel embeds in the precast unit **(Photos 69-71).** Most of the welded connections are concealed in the precast joints and are not visually accessible for inspection. However, there are some connections that are exposed due to the geometry of the precast units. Those that were observed appeared to be in good condition with only minor surface corrosion. No section loss or significant corrosion was observed at or near any of the visibly exposed concealed connections.



The typical precast to compression ring connection from precast shop drawings. The actual conditions vary per level.

Based on the drawings provided and from observations, the connections were made by welding clip angles to the rings and to embed plates in the unit below, then the next precast unit was set on top on plastic shims. The precast shop drawings show the joints filled with grout 4" from the finished surface. At most rings, the interior facing joint is partially or completely obstructed by the ring tube and difficult to access for sealant application. The result is that the sealant is not always fully and cleanly installed around the clip angle at the precast joint. We did not observe any corrosion of these angles or corrosion staining, which would indicate corrosion of the concealed embeds or clip angles. The protection of these connections should be a priority to maintain the stability of the structure as they would be impossible to repair without dismantling the precast units. With some exceptions, the connections are protected from direct weather exposure and the sealant work appears to have successfully protected the connection and the joints. Wood blocking and excess grout were observed behind the rings at some connections.

As mentioned, in some instances the embed ring connections are partially exposed at the horizontal surface of the precast unit below **(Photos 73-74).** These exposed embed pockets have been filled flush to the adjacent surfaces with patching material, which in most cases has failed. Water can easily enter these exposed embed pockets. Our examination of the steel at one of these connections which was exposed by the contractor (El. 80'-0") presented minor surface corrosion with the welds in good condition. However, to reduce the risk of future corrosion, these connections should be proactively protected as indicated.

Recommendations

The recommendations for the protection of these critical ring connections has several components depending on each condition. These include the preparation and re-coating of the exposed clip angles and welds. The sealant should be removed so that the paint can be applied as far as possible into the joint then new sealant installed. Where the clip angle and sealant joint are not accessible, install corrosion resistant flashing or covers to divert water safely away from the connection. The removal of excess grout and wood blocking **(Photo 72)** is recommended to allow water to drain away from the joint. In some cases, the installation of reinforced fluid applied roofing membrane may be applied over these connections or a combination of these solutions.

4. Carillon Secondary Framing (above EL. 72′-9″)

Between the Belfry level deck and the Observation deck level there is a galvanized secondary framing system supporting the carillon bells and mechanism (bell beams). There are galvanized gratings in this framing to provide access for maintenance. The secondary framing appears to have been designed and installed separately than the structural steel, probably by the carillon fabricator/installer. These members bear on the Belfry level deck or are welded to the vertical galvanized steel structure **(Photo 75-76).** All field welds are coated with zinc-rich paint and no corrosion of the field welds or carillon framing members was observed. We did see significant bird droppings on several areas of the bell framing indicating that birds are roosting in the belfry.

Recommendations

We recommend that the secondary framing welds be checked for corrosion and recoated with zinc-rich coating as needed. Posts that bear on the Belfry level deck should be temporarily shored during repair of the deck and then reattached with stainless steel anchors. Waterproofing of posts bearing on decks is addressed later in this report. To address the bird roosting, a fine stainless-steel mesh should be installed to prevent birds from getting into the area with removable sections to allow for maintenance access.

5. Composite Decks

There are 4 composite concrete decks in the tower; at the Gallery level (El. 52′-6″) the Belfry level (El. 72′-9″), the Observation Deck level (136′-4″) and a smaller deck at the ground level which serves as a bridge to the elevator. Beside serving as walking platforms, the decks participate in the overall structural design. For the purposes of this architectural review, we will address the usability and durability of these decks. The repair design would be implemented in conjunction with any structural repairs.

The typical corrosion buildup between ring beam and deck edge. Note vertical streaks from water runoff.

Per the drawings, each 4″ deck is composed of a steel angle defining the perimeter, a corrugated steel deck that sits within the angle and on top of the steel beams and a concrete fill that is the walking surface. The decks are on constructed on the compression rings with the edge of deck aligned with the edge of the ring beam. The Observation deck edge angle and decking appear to be galvanized, consistent with its original design, but the lower decks are coated steel. Significant corrosion was observed between the rings and the deck angle **(Photos 77-78).** Based upon our observations the corrosion of the steel decking varies from severe to isolated areas of corrosion. The most severe corrosion of the decking was observed at the Belfry level where corrosion has led to 100% section loss of isolated areas of the corrugated decking and approximately 5% at some perimeter angles **(Photos 79-81).** This deck was intended to be the "roof" level with a skylight opening in the center and covering the elevator shaft. The surface of the concrete is uneven and rough at this level. The roofing and skylight were not installed which creates unintended and vulnerable deck edge conditions. Besides corrosion of the edge beams and angles, the unprotected edge of the elevator shaft **(Photos 82-83)** is of concern for water infiltration and potential damage within the elevator shaft. As mentioned, the interior of the shaft was not accessible for inspection.

The condition of the welded wire fabric embedded in the concrete deck as indicated in the drawings is unknown. The decks are coated with a slip resistant traffic coating which is in serviceable condition at all levels except the ground floor level. However, the edges of the decks are not sealed or coated which has allowed water to enter between the concrete fill and the steel edge angle **(Photos 77-81).** The result is the corrosion of the steel edge angle, corrugated deck and un-coated parts of the supporting steel below. At the Gallery level, there are some abandoned slab edge angles and tubes on the south and southwest sides for decking that was never installed. The water management and waterproofing of the decks are addressed elsewhere in this report.

Recommendations

Due to the changes in the enclosure from the original design and the lack of water management strategy, the deck structures have been subject to weather exposure that they were not designed for. The resulting deteriorated condition of the decks and associated steel members suggest that the three decks be rebuilt with corrosion resistant materials, a water management strategy and high-performance waterproof membrane system. While some decks are in worse condition than others, they will continue to deteriorate, and it is more economical to replace them all in the same project while the contractor is mobilized, and consistency of design and quality can be maintained. The decks should be replaced with galvanized steel or stainless-steel edge angles and stainless-steel corrugated decking. While exposed, the supporting steel members should be cleaned down to bare metal and recoated and any concealed ring beam clip angles and joints repaired. Further assessment during the removal of the existing decks. Unused or abandoned deck angles and tubes should be removed and exposed surfaces coated.

Necessary design modifications or improvements should be made during the deck replacement to improve the durability and longevity of the decks. These may include the edge angle relocation at the belfry opening, at elevator shaft, stair stringer or landing connections.

6. Precast Fin Connections

The narrow precast concrete fins on each side of the structure are connected to the structure at each of the compression rings via steel embeds and welded angles. Each connection has a different configuration due to the varying size and location of the compression rings. Steel angles field welded to embeds do not have a protective coating on the concealed surfaces causing surface corrosion and corrosion staining on the adjacent precast panels **(Photos 43-44).** Most of these connections are experiencing some level of corrosion. The steel embeds take up most of the width of each panel corner which appears to have caused local distress in the concrete panel at most embed locations.

There are also three narrow fins within each of the main precast piers connected with similar embeds and field welded steel angles and plates (Photos 57-59,84-86). These connections present some of the same conditions as noted above and some are patched over with an unknown patching material. These patches have failed in most locations and exhibit delamination and hairline cracking at the parent concrete.

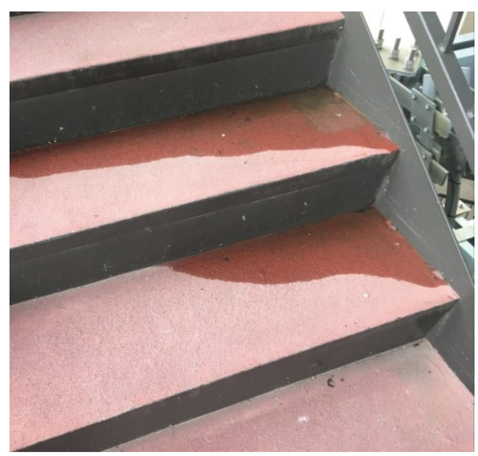
Recommendations

To ensure the continued viability of these connections, all corrosion that can be accessed should be removed to bare metal and the surfaces coated with the appropriate zinc-rich primer and high-performance paint. Further, the concrete immediately adjacent to the embeds should be treated with a clear mineral coating to reduce localized water penetration. After repainting, sealant should be installed at gaps in the steel to steel connections to prevent water penetration and the resulting discoloration from corrosion. The corrosion stains should be cleaned prior to making the above repairs. This is detailed work and will require tools that can fit into narrow openings. We would also recommend that the narrow concrete edges adjacent to each embed be excised to preclude continued spalling. This strategy is further discussed in the concrete section.

To protect the connections of the fins within the main piers, the patching material **(Photo 60)** should be removed, and the steel connections treated as noted above. To reduce the connection visibility, the new paint should match the adjacent concrete. Sealant should be installed as needed to prevent water infiltration into the welded connections and precast joints.

7. Steel Stairs

The stairs that provide access to all levels for the tower are fabricated from bent steel sheet with flat plate steel stringers. Each tread and riser are composed of one bent sheet of coated steel that is overlapped by the tread above and set on a small clip angle welded to the stringer **(Photo 60).** The original architectural drawings indicate non-slip structural treads, graphically shown as cast treads. The railing posts are welded to the outside of the stringer plate. Stair landings are made from 1/8”-3/16” steel sheet bearing on various cantilevered beams and ring beams. The stairs and railings do not appear to be galvanized at any level. The treads and landings have a red traffic coating that is delaminating at some locations. Vinyl tread covers have been installed at the stairs from the basement to the ground floor which are coming loose and pose a tripping hazard.



The typical corrosion buildup between ring beam and deck edge. Note vertical streaks

We observed widespread, minor surface corrosion of the stair treads, risers, landings and supporting clip angles **(Photos 87-89).** Without repair and a robust waterproofing of the treads, continued corrosion will require ongoing maintenance and could create hazardous conditions. The treads overlap the risers by about 1/2” making direct contact. These overlapping and contacting surfaces are not protected by paint and are exposed to water from runoff and wind-driven rain. Runoff staining is visible on the back side of the risers from wind-driven rain. Uncontrolled water runoff of the landings is allowing water to run between the landings and supporting steel. Standing water was noted on many of the treads due to backward slope. The stringers and railings are in generally good condition except for isolated paint failure and corrosion.

Recommendations

Considering the continued weather exposure and corrosion of the stairs, the waterproofing and drainage strategy is critical to its long-term viability. Water should be controlled on the stairs and landings so that it is directed to drainage points. This may include the addition of kick plates and drain ports on landings to prevent water from running off the edges of the landings. The overlapping metal to metal contact of the treads and risers should be resolved possible by tack welding them together prior to installing the waterproofing on the steps. The stairs and landings should be continuously waterproofed to prevent water from leaking through joints at the sides and treads. The proper long-term waterproofing system should be more robust than a simple traffic coating and must be able to withstand thermal movement of the stairs and bridge over gaps.

An alternate, more durable option would be to replace the existing treads and risers with non-corrosive, non-slip structural treads and risers. This would eliminate the need for surface preparation and recoating of the existing stairs and would solve the steel to steel contact points. This would also allow water to drain through the stairs and the connections to dry out, reducing the risk of corrosion.

IV. Non-Structural Components

Observations

A. Waterproofing, Roofing and Sealants

The moisture protection components of the structure include waterproofing, roofing and sealants. For the purposes of this report, waterproofing is defined as the membrane installed on the concrete structure below grade and on the basement roof slab to prevent water infiltration through the structural concrete into the lower level. Roofing is defined as the exposed membrane protecting horizontal surfaces on the exterior or interior of the structure. Sealant is defined as an elastomeric joint protection installed between different materials or components.

1. Waterproofing

Failure of the waterproofing system is evident from the staining and efflorescence emanating from cracks in the ceiling of the mechanical room in the basement. We did not see infiltration in the walls or floors of the basement. The elevator shaft and pit, and telecom room in the basement was not accessible for inspection. The location of water infiltration is directly below the ground level plaza which indicates a failure of the waterproofing system below the exposed wear slab. The composition of the plaza is a structural slab with a waterproofing membrane and an exposed wear slab. Typically, this concealed waterproofing is turned up onto the wall or window curb so that water cannot travel along the membrane and into the building. It is unknown without making exploratory openings if the membrane itself failed or if water is migrating from an exposed edge condition and under the membrane.

The exterior ground floor plaza is a combination of slab-on-grade and topping slab which requires the waterproofing to transition from vertical foundation walls to horizontal structural slab. We also noted that the plaza slab is higher than the interior floor level and slopes toward the building in some locations. The curb supporting the glazing system is right at plaza level which does not provide sufficient protection at the building perimeter **(Photos 99-100).** Some windows are set on exposed CMU and some are set on the exterior slab which extends into the building. Both conditions do not provide sufficient protection. From the small portion that was visible, the waterproofing appears to be turned up sufficiently behind the stone veneer between the glazed walls. Because of these deficiencies, water may be infiltrating the basement slab from the interior of the building or the transition between the interior and exterior slabs at the glazing line.

Recommendations

To ensure a durable and effective waterproofing assembly it necessarily involves modifications to the ground floor slope and elevation of the exterior and interior topping slabs, glazing curbs and waterproofing. We recommend that the exterior wear slab and slab on grade be replaced, the interior topping slab replaced and the waterproofing repaired or replaced. The glazing should be removed, a concrete curb of sufficient height installed and waterproofed and then the glazing reinstalled. The removal of the glazing system may damage it so replacement of the glazing system with correct sill pans and weep system may be advisable anyway. Waterproofing should be installed under the interior structural slab below topping slab the modifications to the slope and elevation of the interior topping slab is also necessary for proper interior water management and functionality.

2. Roofing

For the purposes of this report the roofing of horizontal surfaces includes the structural composite decks at the Gallery, Belfry and Observation Deck levels, the ground floor topping slab and deck, the carillon cabin roof, and the skyward surfaces at exterior facing precast ledges and the ladder beam at the highpoint of the roof. The roofing of these surfaces involves modifications to the water management and drainage strategy.

a. Structural Composite Decks

The composite decks currently have a polyurethane based slip resistant traffic coating which appears to be in serviceable condition. However, the traffic coating does not protect the underlying structure because it cannot bridge gaps, moving joints, transitions or the steel edge slab and be applied up onto penetrating steel.

Recommendations

To ensure a durable and effective roofing of these surfaces, the water management strategy should be developed to prevent uncontrolled water runoff This will involve the design of the new decks if they are replaced as recommended. The recommended roofing system for these surfaces and the ground floor topping slab and deck is a flexible reinforced fluid-applied urethane or Poly(methyl methacrylate) (PMMA) with a slip resistant aggregate coating and color top coat. The material is applied continuously up onto penetrating steel columns and railing posts.

b. Carillon Cabin Roof

The cabin roof appears to be a rubber roof which will exceed its lifespan soon. We also observed standing water on this roof indicating insufficient slope. This roof should be replaced during any reroofing project with the fluid-applied roofing mentioned above. The cabin enclosure itself was not reviewed as part of this report but should be inspected during the repair design.

c. Horizontal Ledges

There are numerous skyward facing surfaces on the precast piers including steps at each architectural setback, small surfaces at openings in the piers (Photos 101-104), and at the roof panels. Some of these ledges have a grouted post-tensioned dead end which should be protected. Damage to the grouted post tension pockets was not observed at any of the locations observed. The traffic membrane has been applied to some of these ledges and some have failed. There are additional ledges at vertical fins within the piers, some with exposed clip angle embed connections. The ladder beam at the peak of the roof has horizontal surfaces with sealant joints and exposed clip angles.

Recommendations

The skyward facing ledges should be protected by the installation of the same fluid-applied roofing recommended for the decks. The existing surface should be ground down to bare concrete and the membrane installed extending 2” onto the adjacent vertical surface. The membrane should be applied so that there is a positive slope off these surfaces and an industry standard vertical return to protect the horizontal to vertical corner transition. The horizontal surfaces on the roof ladder beams should receive the fluid applied roofing to prevent standing water from infiltrating the PC units. The joint sealant at the roof was not accessible for inspection but should be inspected and replaced if necessary during any repair program.

3. Sealants

Sealants are applied to transitions between different materials and between separate units of the same material.

a. Precast Concrete Joints

The silicone sealant joints between precast units were found to be installed as indicated in the construction documents and shop drawings. The grout within the joints is recessed and backer rod and sealant installed. A pull test was performed at two locations during the exterior inspections and we found the sealant at these joints to be in good condition with no evidence of cohesive or adhesive failure **(Photos 105-106).** The sealant was found to be discolored by environmental contaminants and no evidence of the leaching of plasticizers into the adjacent concrete was observed.

b. Other Sealant Joints

The sealant joints between the elevator enclosure and the precast was found to be poorly designed and installed and deteriorated due to UV exposure. Sealant at the glazing perimeter, base of the building and the plaza joints was found to be poorly installed, displaying both cohesive and adhesive failure. Sealant at the stone veneer copings and small roof is failed at many locations and deteriorated due to UV exposure.

It is recommended that these sealant joints be replaced with the appropriate sealant, properly designed, during any repair program.

B. Stone Veneer and Coping

The stone veneer at the base of the building is applied to the concrete structure and to CMU infill. The veneer and mortar joints appear to be in good condition with few hairline cracks in the mortar. The veneer transition to the slab is problematic as at some locations the slab is higher than the copper flashing and the flashing is turned up preventing proper drainage of the cavity **(Photos 107-110)**. It also appears that the veneer may bear on a CMU curb which is not waterproofed. This concealed transition may be a source of water infiltration into the basement as the waterproofing condition is unknown. The West side of the stone base has a larger roof adjacent to the elevator shaft of unknown material. The sealant joints on this roof appear to be deteriorated and there is not sufficient slope on the roof for proper drainage **(Photos 111-112)**. There are limestone coping stones at the top of the stone veneer **(Photos 112-114)**. We observed that the sealant joints between coping stones and between the stones and the concrete structure are deteriorated and may be allowing water infiltration into these walls. It is suggested that the coping stones be removed, a waterproofing membrane installed, and the coping stones reset with adequate drainage.

Recommendations

It is recommended that the condition at the base of the wall and waterproofing transition be further assessed and repaired during the removal of the plaza slab. Probe openings in the wall can be made to better understanding the hidden condition. In any case, a concrete curb should be installed to aligned with the window curb to raise the stone above the ground and allow for the application of continuous waterproofing termination and positive drainage above grade.

The coping stones should be reset to provide sufficient slope for drainage a through wall flashing installed and the joints resealed. based on the findings of probe openings and removal of the coping stones, a continuous water-resistant membrane may be recommended for the entire wall extending from the below-grade waterproofing, up over the CMU Back-up, tying into the concrete structure. The larger roof area should be modified to provide positive slope and a fluid-applied roofing membrane installed.

C. Window Wall

The window wall system was reviewed as part of the ground floor enclosure assembly. The glazing system is exposed to the weather from the top taking on water more than it is designed for and would if it was installed in a wall with the protection at the window head. We noted that there is some corrosion at the fasteners and base from excess water infiltration. This system is designed to drain water out at the base via sill flashing that directs water safely out. In this case, we noted that the sill is level, in some cases is level with or lower than the surrounding grade. This prevents proper water drainage which can lead to premature failure and infiltration into the interior. Proper detailing, sealing and installation of end dams and sill flashing is necessary to prevent premature failure and the attendant damage to the interior. In addition, the system is not designed for water infiltration from the interior increasing the need for additional capacity to evacuate water from the system.

The interior topping slab that was added after the window installation is applied so that it covers the bottom of the windows in some areas. Water that that enters from the interior or exterior side can become trapped and enter between the concrete deck and topping slab leading to deterioration of the topping, traffic membrane and window system. We noted that the window system is set onto un waterproofed concrete masonry units which are level with or sometimes lower than the surrounding pavement. This condition can saturate the CMU, leading to long term deterioration of the CMU. Sealant was noted to be in deteriorated condition or non-existent at the sill.

Recommendations

To protect the components of the window system and minimize water infiltration and deterioration of the supporting CMU, we recommend the following in order of importance. First, the windows should be raised so that they are at least 4" above the surrounding pavement. Ideally this is done by temporarily removing the windows, replacing the CMU sill with concrete curb and waterproofing. Reinstall the windows on the raised sill with new properly sealed sub-sill flashing with end dams. Add a cap flashing or metal coping to the top of the window system and reseal window perimeter. If additional protection is desired, angled glazing or sheet metal assembly can be added to the top of the window to reduce the amount of water falling on the interior walkway. The addition of the higher concrete sill will also allow the interior topping slab to meet a concrete sill instead of the window system.

D. Concrete Surface Discoloration

The concrete is discolored primarily with a biologic growth coincident to the high moisture content of the concrete. Naturally this is concentrated on the north sides of the concrete units which receive less solar driven evaporation. However, much of the interior is also discolored with the greenish dark brownish hues of biologic discoloration **(Photos 49-52).**

The building is designed to be a sieve to the elements. So, it will naturally remain a moist and humid environment. A large-scale cleaning with biocide options is a consideration with a regular interval for future cleanings. The chemical composition of the biocide must be researched for its long-term impact both on the concrete as well as the natural environment beyond the tower.

V. Conclusion

While the structure is in serviceable condition, repairs and an ongoing maintenance plan should be developed to address the current deficiencies and prevent further deterioration. The deficiencies and repair recommendations detailed in this report are necessary parts of a long-term strategy to assure the durability and longevity of the structural elements as well as the continued usability of the tower. Issues addressed above are the result of a combination of factors including, but not limited to the change of the design of the tower from partially enclosed to unenclosed, deficiencies in the design of the original structure at key failure locations, subsequent retrofits and repairs and poor material choices as in weather protective coatings.

A detailed and coordinated scope of work including the recommended repairs detailed and coordinated between ERA, EChem and Brush Architects should be developed to obtain budget pricing from several contractors. The project priorities should be considered in conjunction the overall scope of work so that efficiencies can be gained by reducing multiple mobilizations by the repair contractor. The repair program should address urgent structural repairs first. Then should follow logically from concrete repairs, deck and steel repairs, to roofing, stairs, then ground floor waterproofing and detailing to exterior slabs.

The repairs should attempt to maintain or improve the aesthetic impact and integrity of the original design. Based on this budget pricing, a prioritized, phased restoration plan can be developed to assist the City and the Riverwalk Commission with planning and budgeting for long term, durable repairs that support the iconic design of the Tower.

Sincerely,

Mary B. Brush, FAIA
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APPENDICES

Appendix 01 - Photographs

Appendix 02 – Drawings

A-0.0 Cover

A-1.0 Southeast Elevations

A-1.1 Northwest Elevations

A-1.2 Southwest Elevations

A-1.3 Northeast Elevations

A-1.4 South Concrete Fin Elevations

A-1.5 East Concrete Fin Elevations

A-1.6 North Concrete Fin Elevations

A-1.7 West Concrete Fin Elevations

A-2.0 Section Looking North

A-3.0 Plans

A-3.1 Plans

APPENDIX 01 - PHOTOGRAPHS

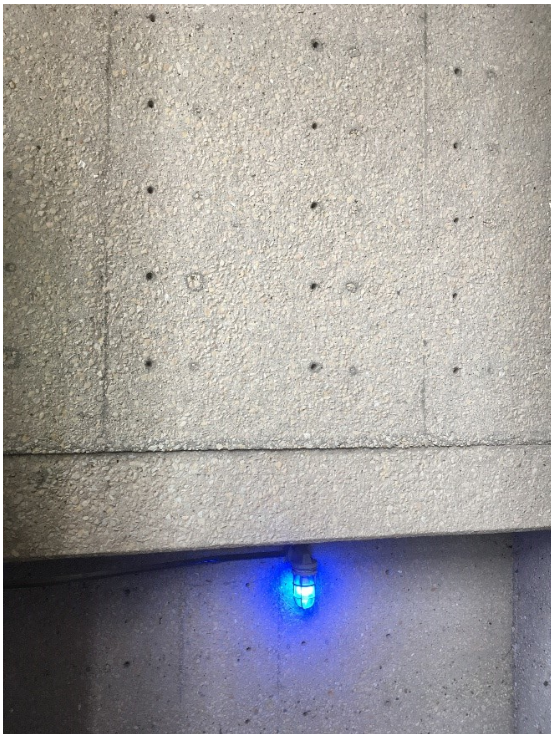
1. Aerial view of the tower from the Southeast with the raised plaza in the foreground.

2. View of the tower from the Southeast with the raised plaza in the foreground.

3. A detail view of the concrete foundation with precast unit above.

4. A detail view of the concrete beam supporting precast fin above.

5. A detail view of the concrete column and beam with precast unit above.



6. A detail view of the concrete foundation. Note the exposed aggregate finish.

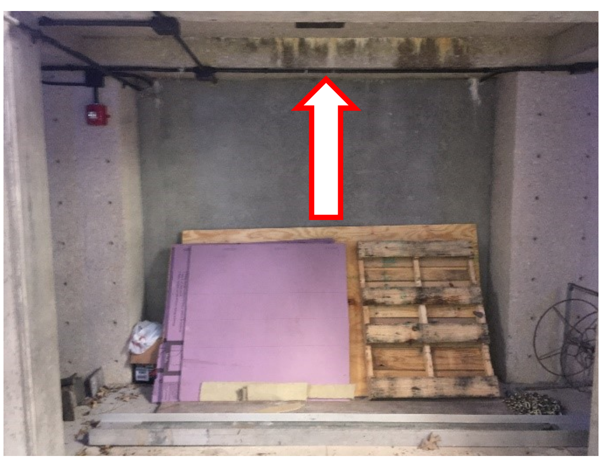
Brush Architects

7. A view of the concrete foundation with precast unit above.

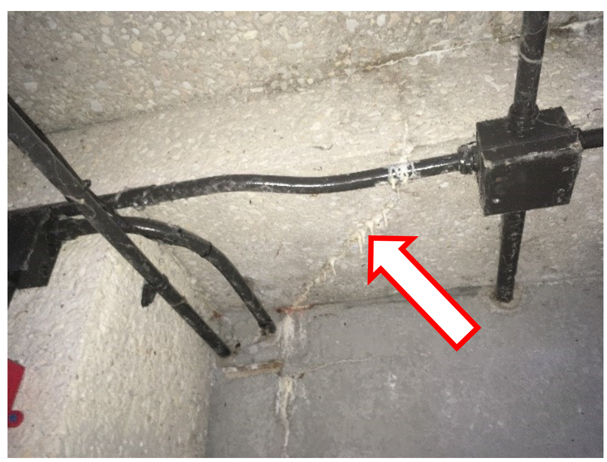
8. A close-up view of the concrete foundation with precast unit above.

9. A view of the basement foundation walls with exposed aggregate finish.

10. A view of the concrete foundation from basement level with hanging stair.



11. A close-up view of concrete cracks and water infiltration below front door.

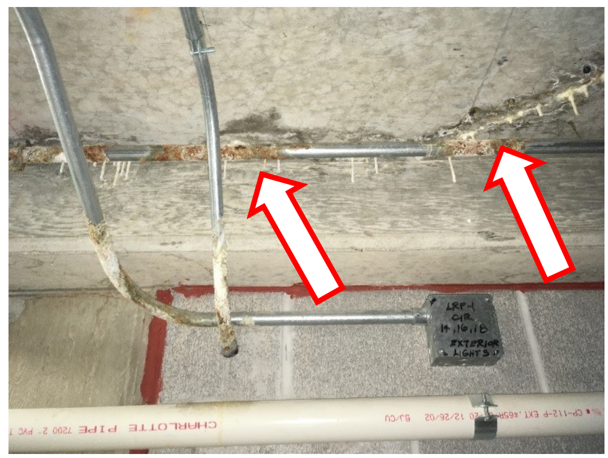


12. A close-up view of concrete cracks and water infiltration below front door.

13. A close-up view of concrete cracks and water infiltration below front door.

14. A view of concrete cracks and water infiltration in mechanical room below plaza.

15. A view of concrete cracks and mineral leaching in mechanical room below plaza.



16. A view of concrete cracks and mineral leaching in the mechanical room below plaza.

17. A view of concrete cracks and water infiltration in mechanical room below plaza.

18. A view of one ring beam connection to the precast. The configuration of ring level connection varies.

19. Wide flange beam connected to precast at the Gallery level.

20. A close-up view of the exposed top of the steel embed at the Gallery level beam.

21. A view of the hanging stair structure rom the basement. Note the 4 lateral braces.

22. A view of the hanging stair structure from the ground level.

23. A view of the lateral braces for the hanging stair structure attached to channel embeds.

24. A view of the slate textured precast units.

Brush Architects

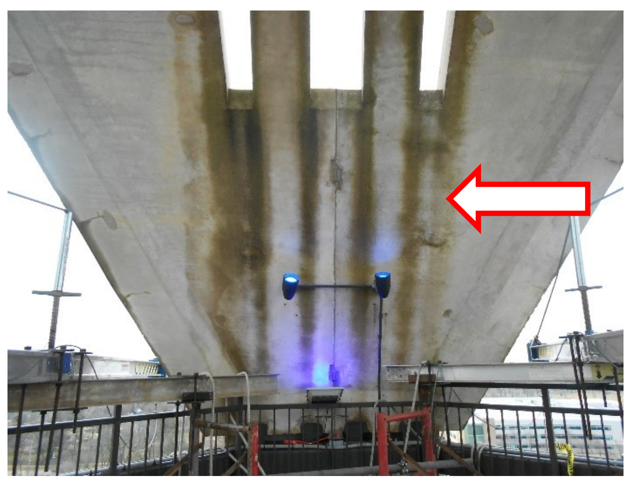
25. A close-up view of the slate texture on precast units.

26. A close-up view of the slate texture on precast units.

27. A close-up view of the stone textured precast units.

28. A close-up view of the knife edge of the roof panel with crack and water saturation.

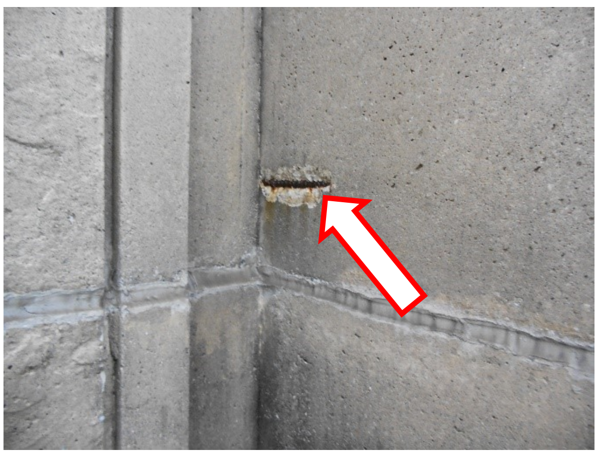
29. A close-up view of the knife edge of the roof panel with cracks and water saturation.



30. A view of the roof panel with knife edge both sides and water staining from runoff.

31. A close-up view of small rebar spalls

32. A close-up view of a small rebar spall. Note the previous patch.



33. A close-up view of small rebar spalls

34. A close-up view of small rebar spalls

35. A close-up of a precast fin panel with several diagonal hairline cracks.

36. A close-up of precast fin panel with several diagonal hairline cracks

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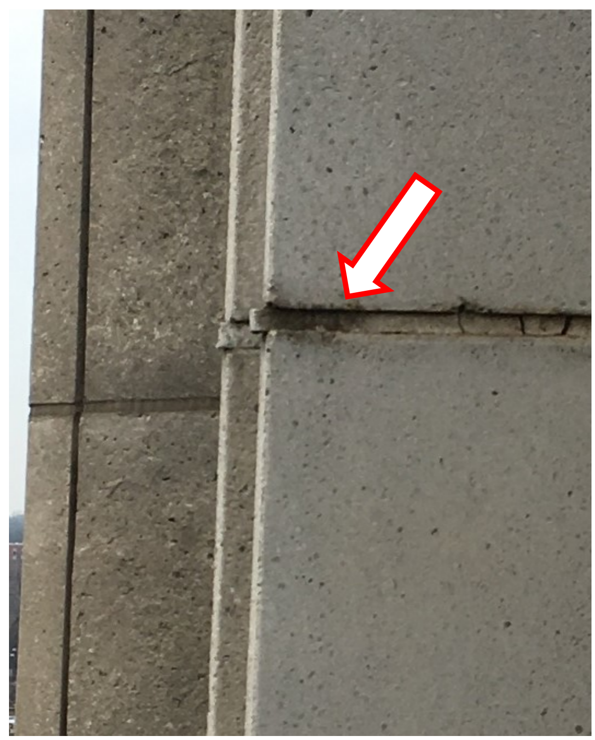
37. Standing water on stair landing.

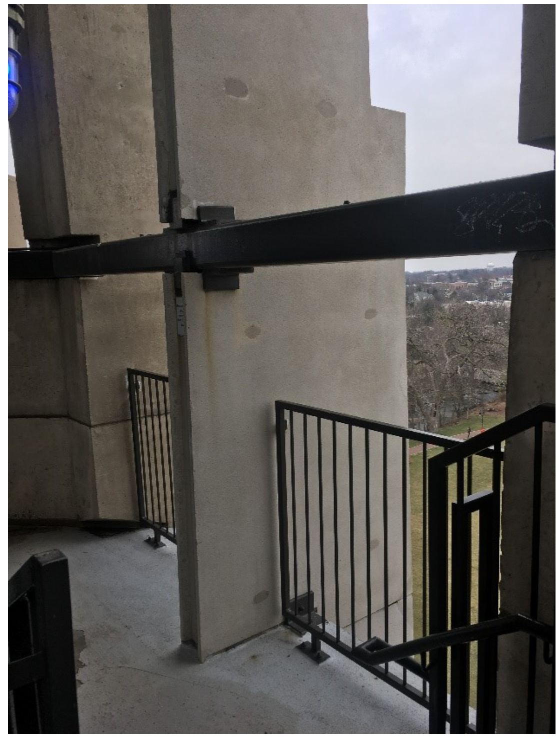
38. Standing water on stair landing.

39. A view of a precast fin. Note patch at outer edge at left.



40. A close-up of view of a precast fin joint. Note loose grout in joint.

41. A view of a precast fin. Note small lift point patches.



42. A view of a precast fin connection. Note small lift point patches.

43. A detail view of the precast fin connection. Note corrosion staining.

44. A close-up view of the precast fin weld connection. Note concrete crack at corner of. steel embed

45. A close-up of the precast fin showing surface finish and exposed aggregate.

46. close-up of the precast fin showing surface finish. Note typical lift point patch.

47. A view of the top of a precast fin. Note corrosion and runoff stains.

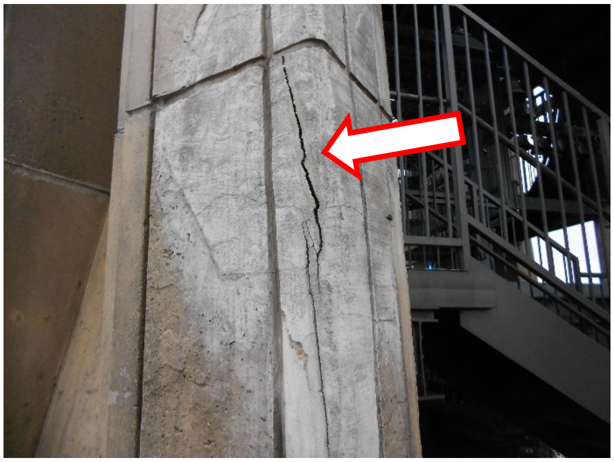
48. A view of the top of a precast fin. Note tensioning rod patches.

49. An example of surface discoloration from runoff.

50. An example of surface discoloration from runoff.

51. An example of surface discoloration from poor concrete consolidation.

52. An example of surface discoloration from biological growth.



53. A close-up view of failed patch at El. 116’-8” prior to repatching. .

54. A close-up view of failed patch at El. 116’-8” prior to repatching. .

55. A close-up view of failed patch on the SE pier El.116-8” after demolition. The patch that was removed did not have sufficient anchorage to prevent a fall. The exposed concrete cast around the Dywidag post-tensioned hardware is not fully engaged with the steel. The architectural concern for this condition due to its repetition on all sides at this level is that the Dywidag is not sufficiently engaged with the structural reinforcing to prevent repeated failure.

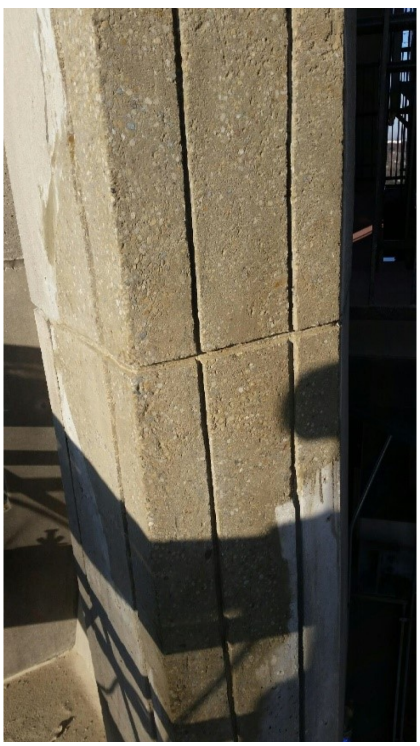
56. A view of previously made patches on the SW pier. At the same 116’ elevation as photo 55. This condition is visible on all sides of the structure at this location

57. A close-up of failed embed patch at small precast fin in pier.

58. A close-up of an intact embed patch.

59. A close-up of an embed patch with sealant.

60. A failed embed patch removed during inspection.



61. A view of the repaired patch on the SE pier made during the investigation. Photo by Golf Construction.

62. A view of the repaired patch on the SE pier made during the investigation. Photo by Golf Construction.

63. A view of the NW pier at the Observation deck. Note corrosion staining below the ring beam.

64. A close-up of corrosion and leak patterns at the NW pier below the ring beam.

65. A close-up of ledge adjacent to the interior leaks at the Observation deck.

66. A close-up of corrosion and leak patterns at the NW pier below the ring beam.

67. Galvanized roof framing at the Belfry level.

68. Galvanized roof framing at the Belfry level.

69. A close-up view of clip angle in precast concrete joint from above.

70. A close-up view of clip angle in precast concrete joint from above.

71. A close-up view of clip angle in precast concrete joint from below. Note corrosion staining from runoff.

72. A close-up view of leftover wood blocking at ring beam connection.

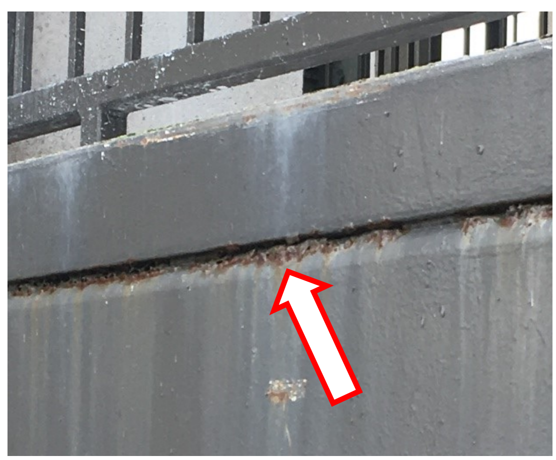
73. A view of a precast clip angle exposed at horizontal surface.

74. A view of a precast clip angle exposed at horizontal surface.

75. A view of horizontal carillon framing at the Belfry level.

76. A view of horizontal carillon framing.

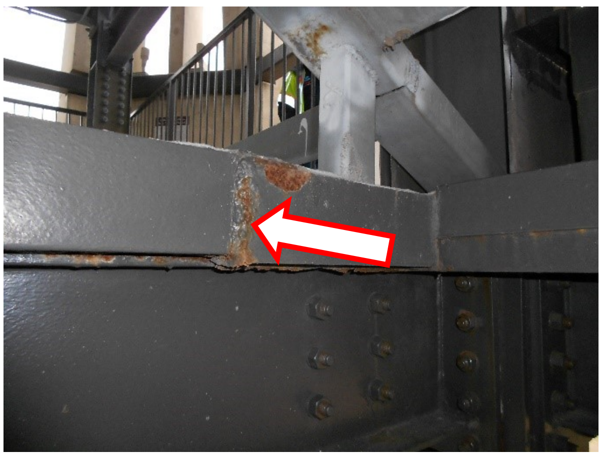
77. A view of typical corrosion at non-galvanized deck edge and ring beam. Note corrosion staining on precast fin.



78. A view of typical corrosion at non-galvanized deck edge and ring beam.

79. A close-up of corroded deck angle and Belfry level.

80. A close-up of corroded deck angle and Belfry level.



81. A close-up of corroded deck angle at the Belfry level.

82. A view of the corroded deck edge at elevator shaft at the Belfry level.

83. A close-up view of the corroded deck edge at elevator shaft at the Belfry level.

84. A close-up of fin channel embed connection in main piers.

85. A close-up of fin connection in main piers.

86. A close-up of fin connection in main piers.

87. A close-up of steel stair with standing water and corrosion.

88. A close-up of steel stair with delamination of traffic coating and corrosion.

89. A close-up of steel stair with corrosion runoff stains from uncoated metal to metal contact.

90. A close-up of bottom of steel stair with supporting clip angles.

91. A close-up of open deck angle and traffic coating at the Observation deck level. .

92. A view of the traffic coating on the Observation deck.

93. A view of abandoned deck support at the Gallery level.

94. A view of abandoned deck support at the Gallery level.

95. A view of abandoned deck support at the Gallery level.

96. A view of the edge of the sloped topping slab at the ground floor.

97. A view of the ground floor failed traffic coating with standing water.

98. A view of the ground floor damaged traffic coating.

99. A close-up view of the base of glazing walls. Note failed sealant and biological growth.

100. A close-up view of the base of glazing walls. Note failed sealant and biological growth.

101. A close-up of precast ledge with traffic coating and standing water.

102. A close-up of precast ledge with traffic coating and standing water.

103. A close-up of precast ledge with intact traffic coating.

104. A close-up of precast ledge with failed waterproofing and standing water.

105. A close-up of 1 of 2 sealant probes performed during the investigation.

106. A close-up of typical precast sealant joint.

107. A close-up of the failed sealant and flashing at the stone veneer and plaza.

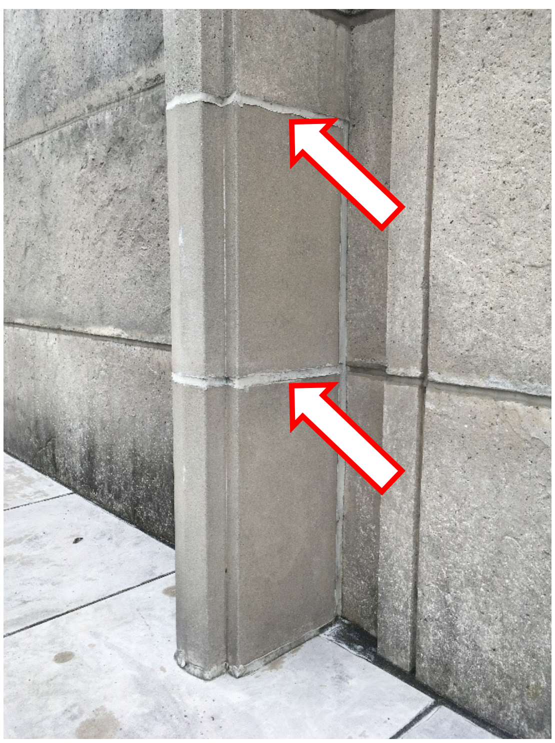
108. A close-up of the failed sealant and damaged flashing at the stone veneer and plaza.

109. A close-up of the failed sealant and damaged flashing at the stone veneer and plaza.

110. A close-up of the failed sealant and flashing at the stone veneer and plaza.

111. A close-up of the small roof at ground floor. Note standing water.

112. A close-up of the failed sealant at the West side of ground floor coping.



113. A view of a precast fin intersecting the Northwest base coping. Note failed sealant between sections.

114. A close-up of a precast fin intersecting the Northwest base coping. Note failed sealant.

APPENDIX F: Brush Architect Drawings

MOSER TOWER AND CARILLON

NAPERVILLE, IL
CONDITION ASSESSMENT

OWNER:

NAPERVILLE RIVERWALK
400 S Eagle St, Naperville, IL 60540

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ENGINEERING RESOURCE ASSOCIATES
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Warrenville, IL 60555

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Poughkeepsie, NY 12601

ARCHITECT:

BRUSH ARCHITECTS, LLC
4200 N Francisco, Chicago, IL 60618

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MOSER TOWER AND CARILLON
Naperville, IL

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SHEET TITLE:

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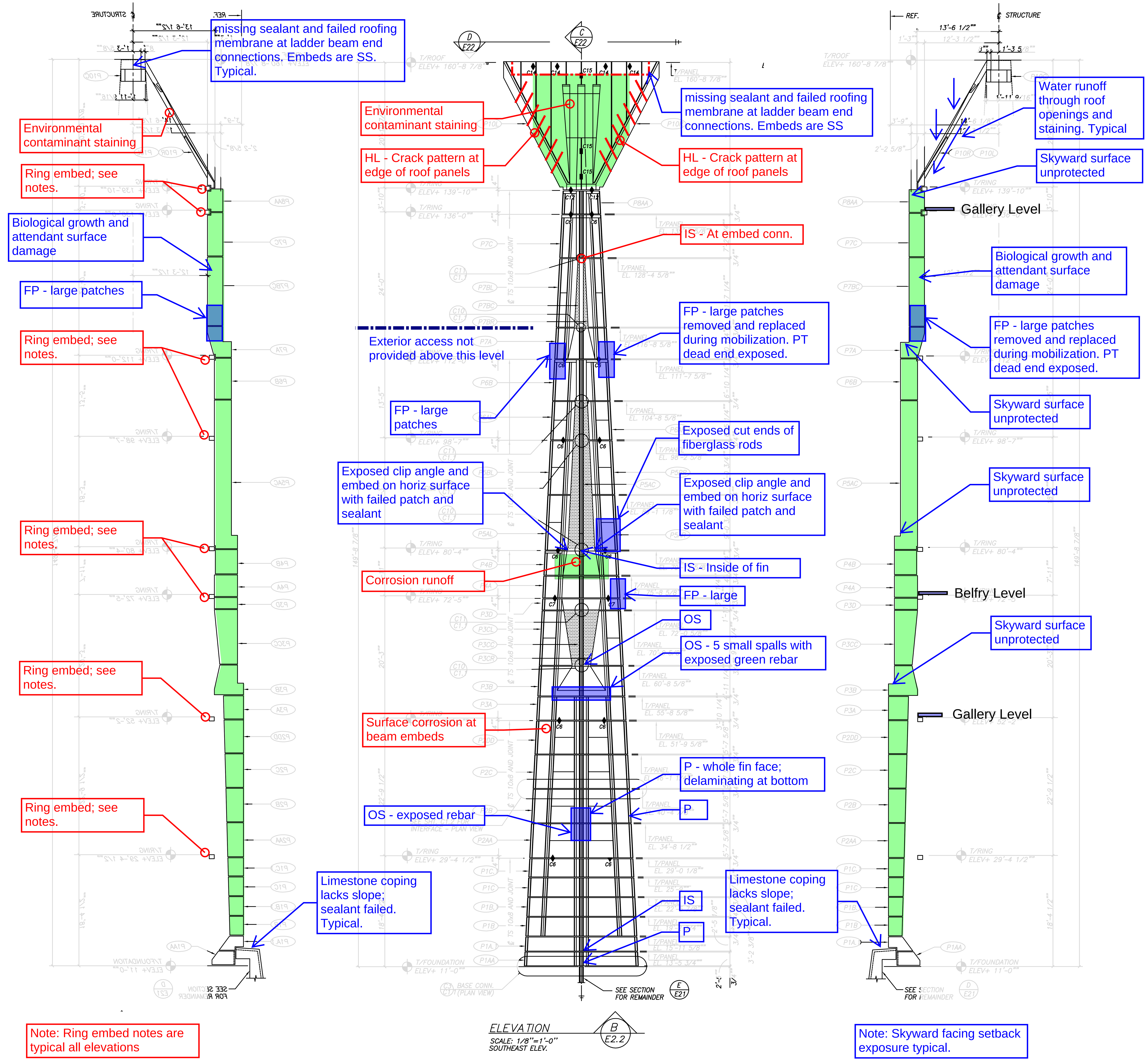

Note: Ring embed notes are typical all elevations

Note: Skyward facing setback exposure typical.

KEY NOTES
OS = Open spall
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FINISH TYPES
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GENERAL NOTES
1. Compression ring connections present in varying degrees of corrosion. Sealant work protecting the connections from water infiltration is inconsistent and/or not serviceable.
2. Corrosion staining is present in varying degrees at interior ring embed connections, beam embeds and channel embed locations.
3. Skyward facing setbacks are not sloped to shed water. Some appear to have a waterproofing membrane which needs to be replaced.
4. Composite decks at the Gallery, Belfry and Observation deck levels present varying levels of corrosion from severe to moderate.
5. Stair treads, risers and tread angle supports present ongoing corrosion.
6. Stair tread traffic membrane deteriorated and delaminating.
7. See drawings and report for additional notes and observations.
8. These notes are based on cursory observations and are not meant to be an exhaustive survey of the conditions.
9. Drawings may be amended during for detailed construction documents.

DRAWING NOTES
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SHEET NUMBER: A-1.0
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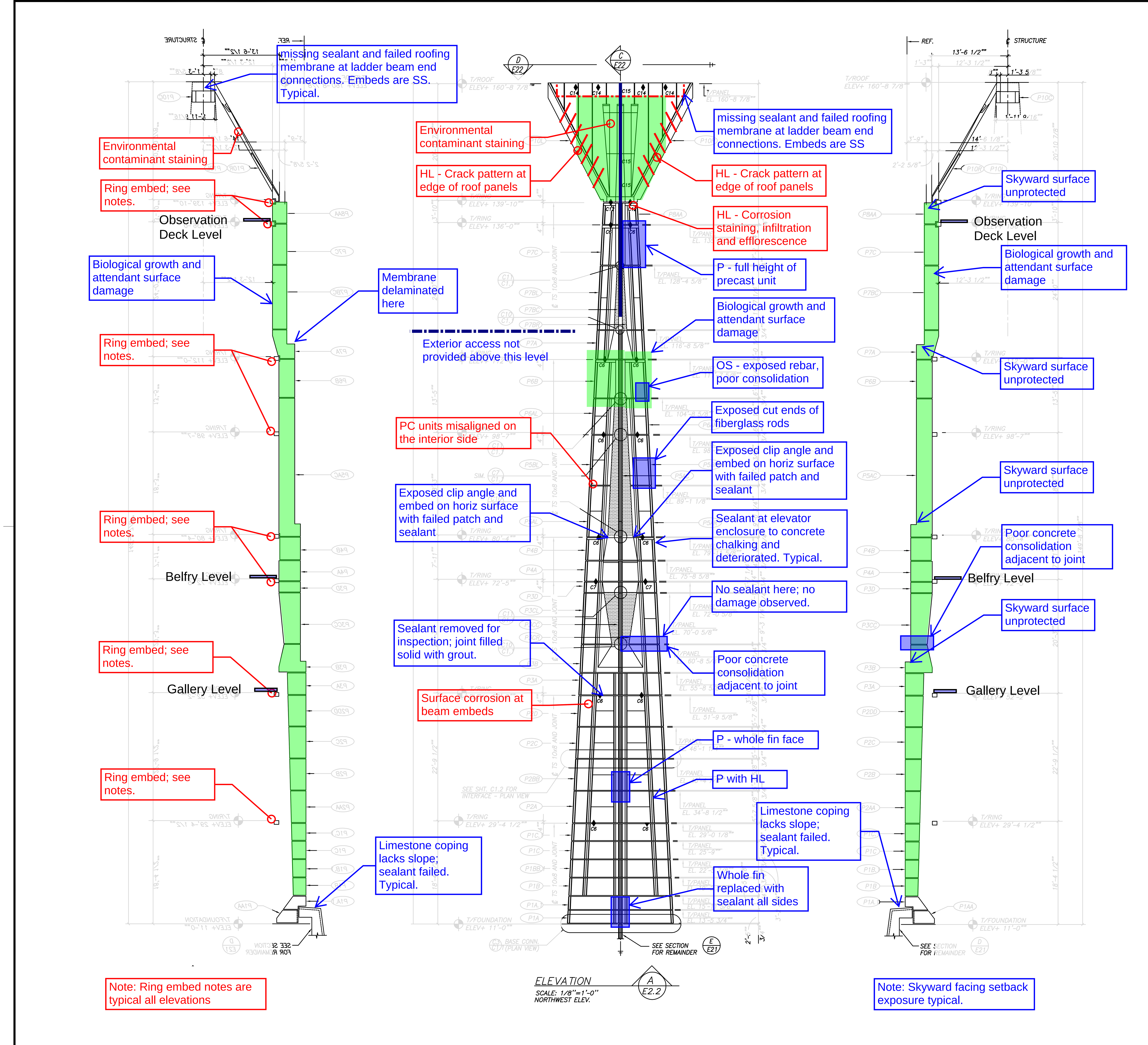

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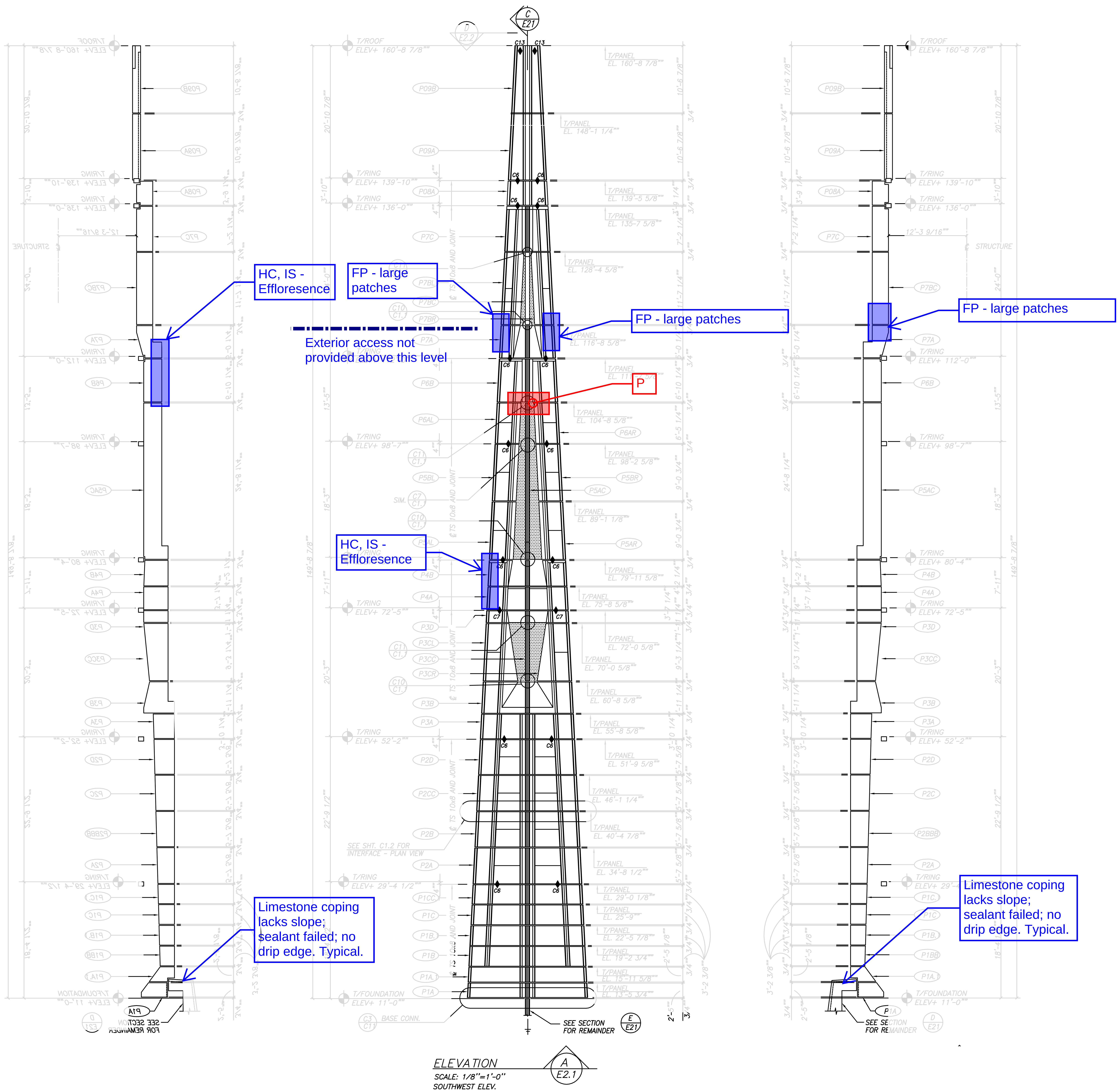

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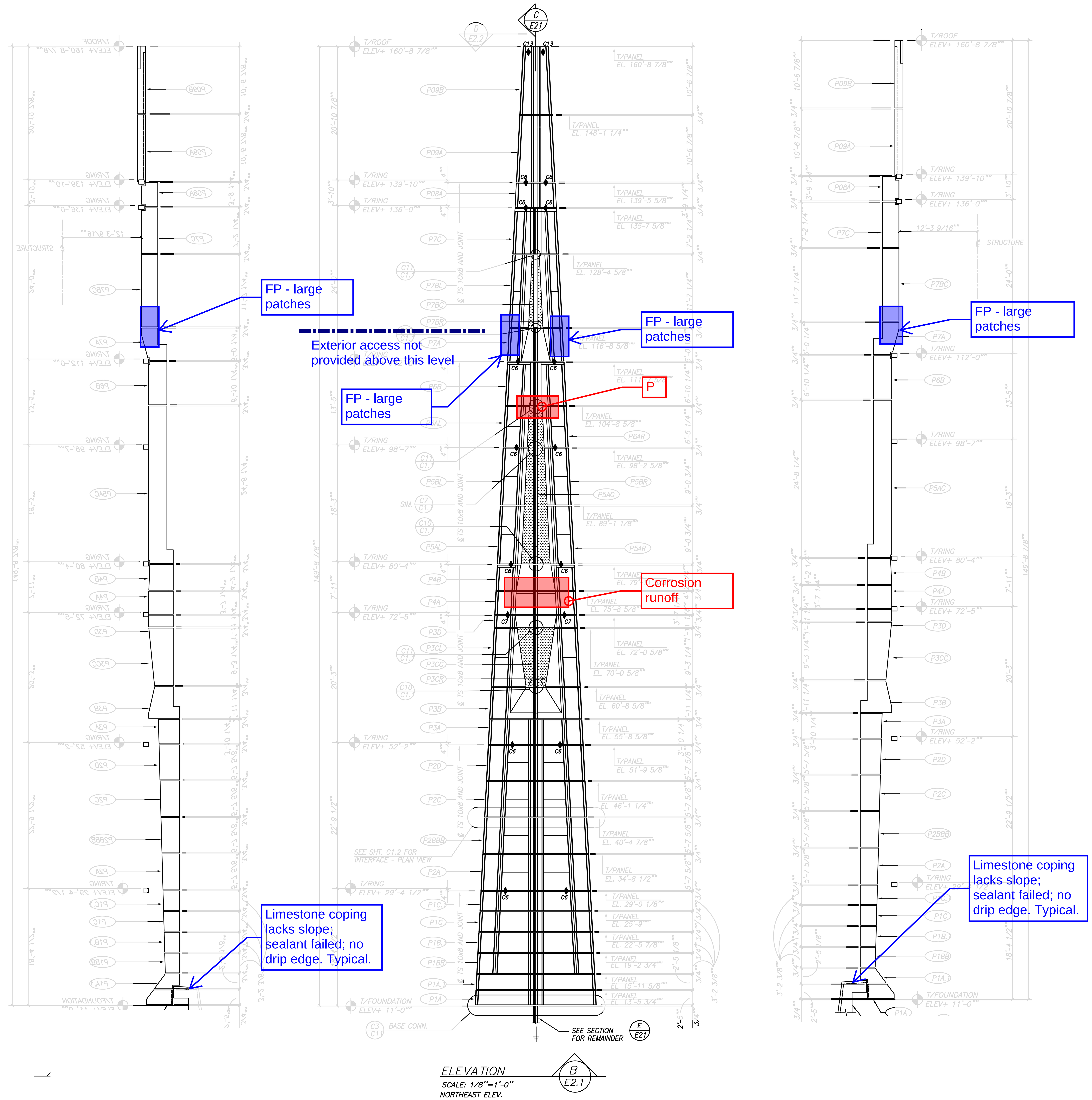

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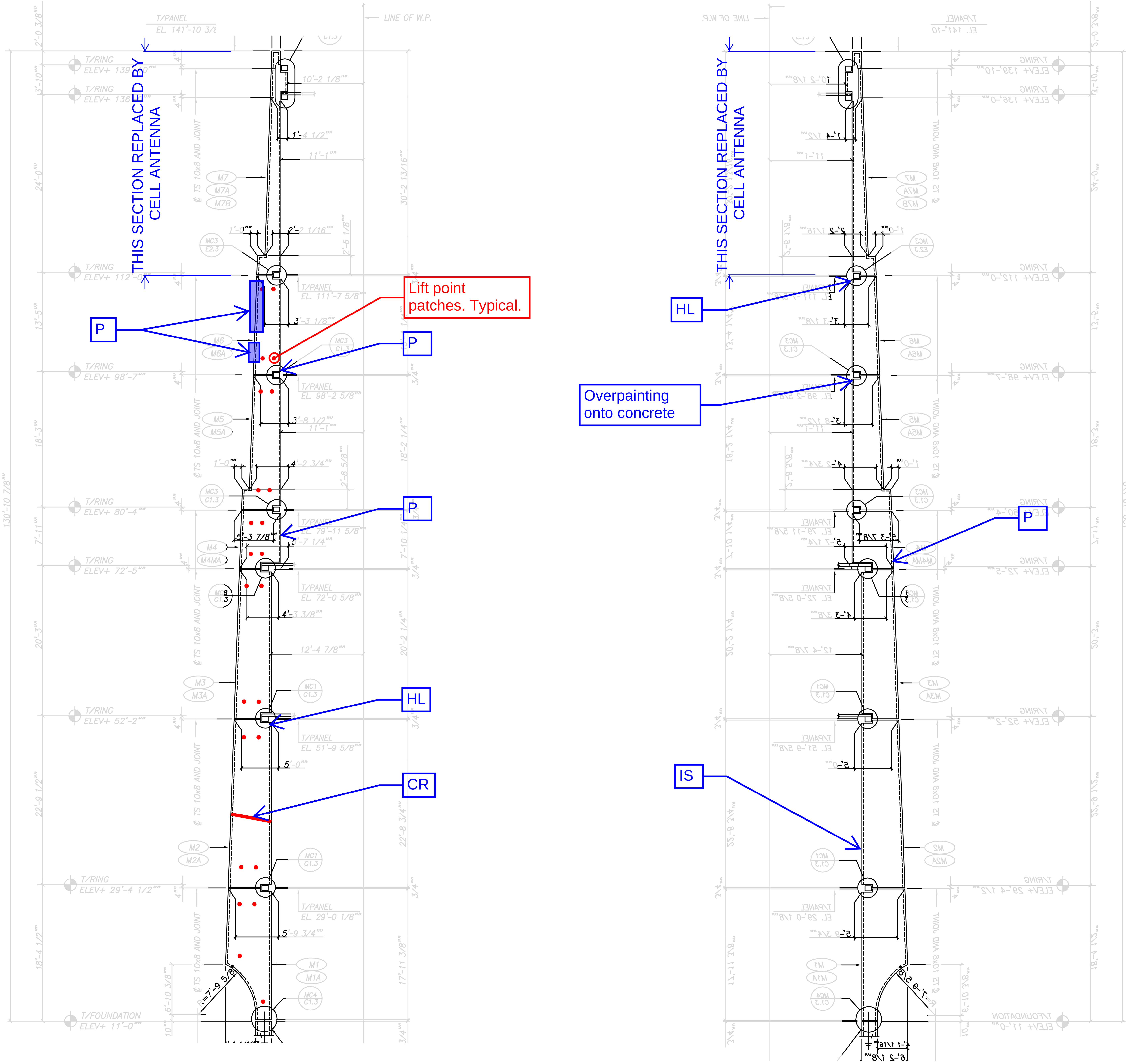

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88867-002

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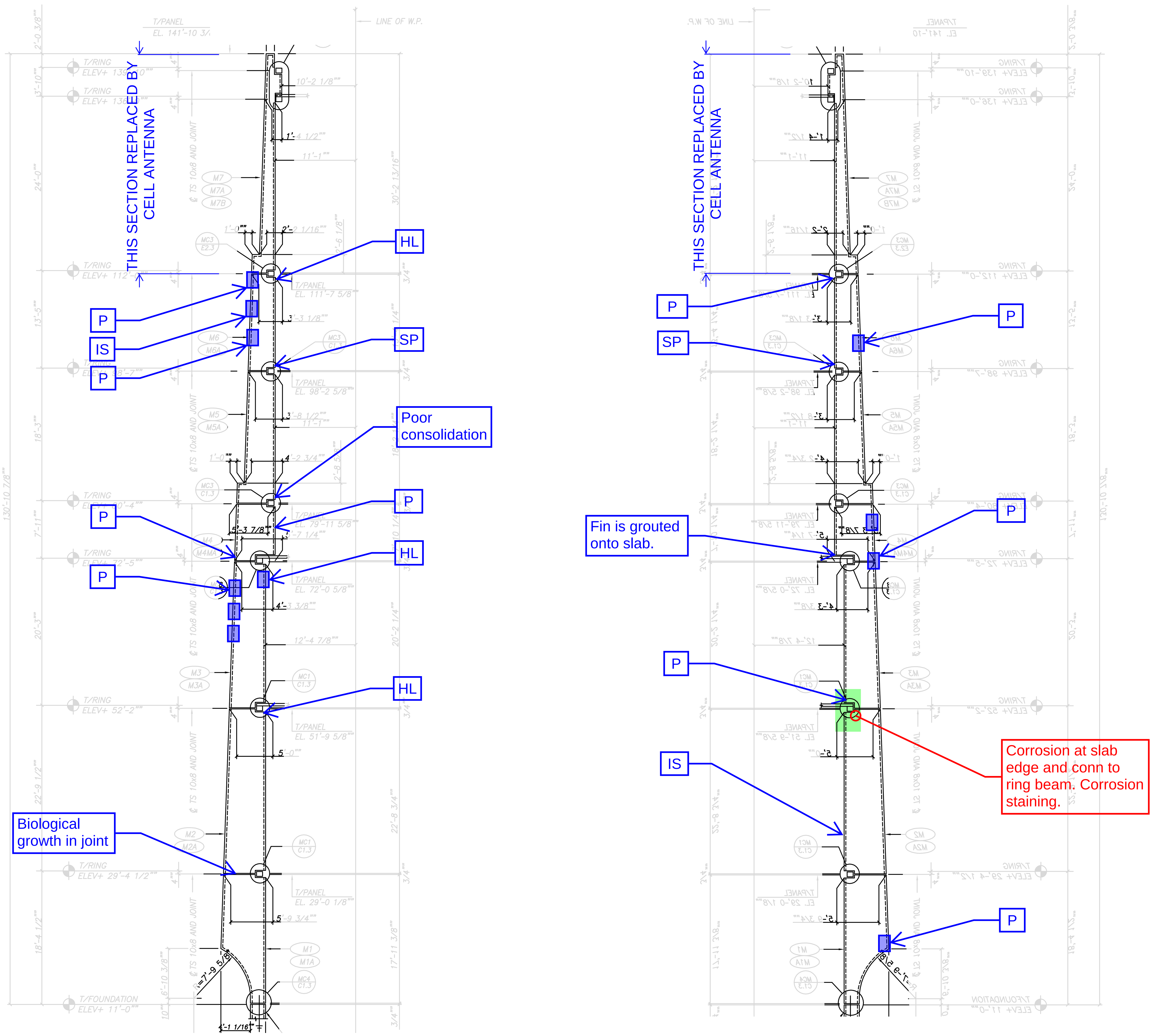

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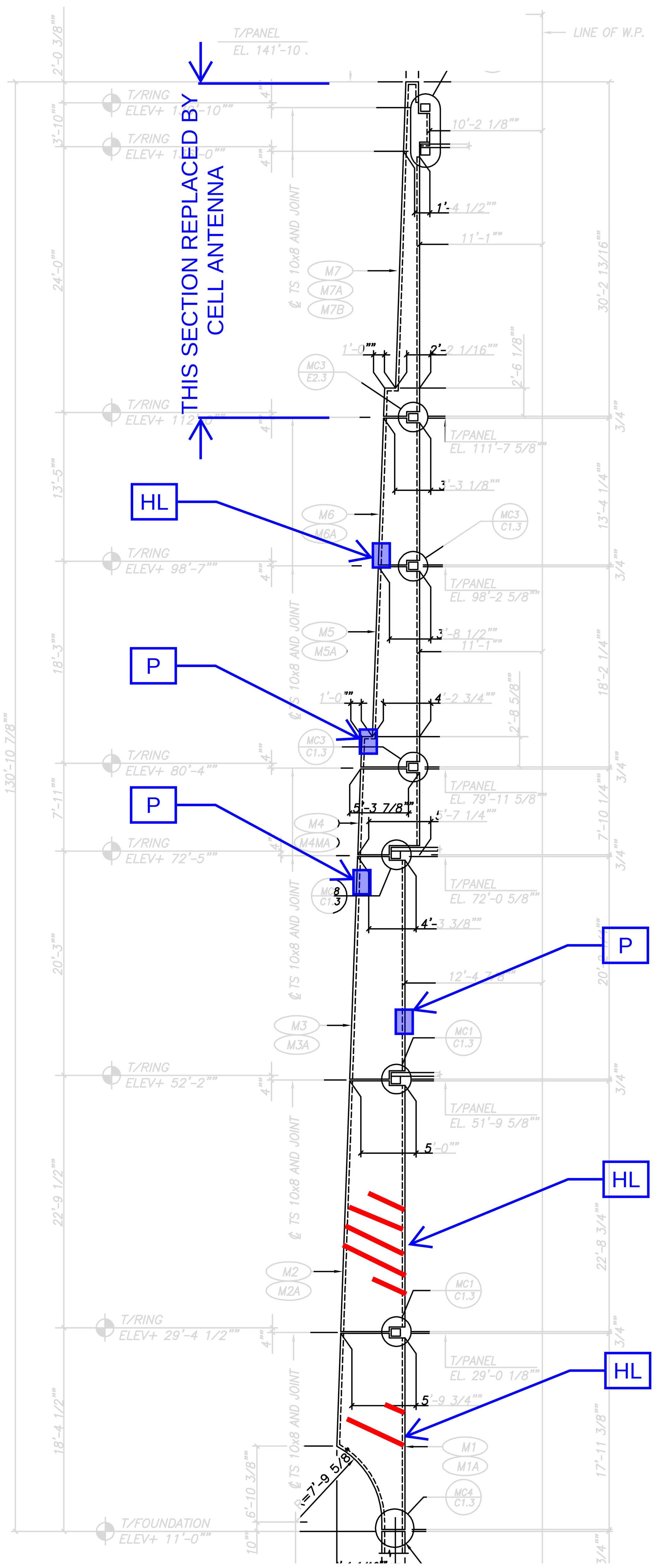

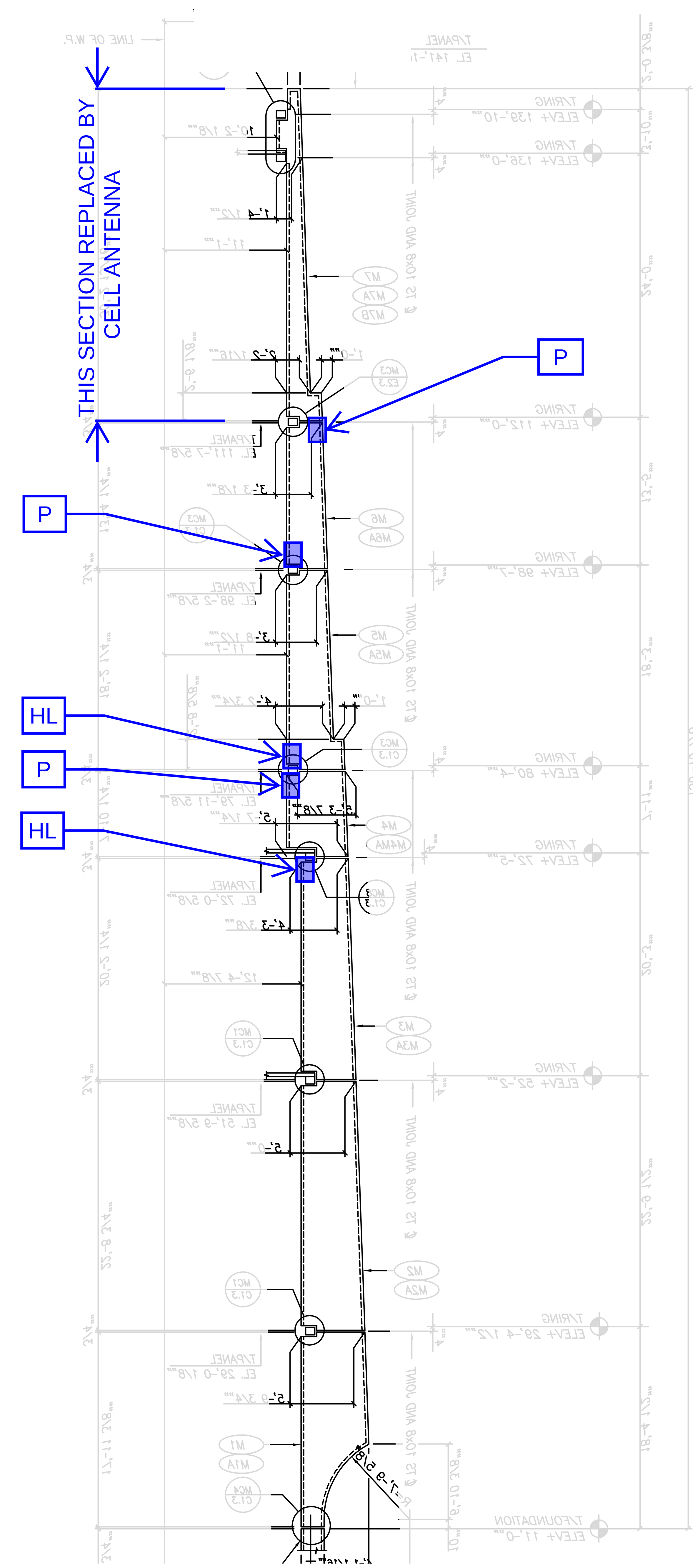

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IS = Insipient spall
P = Patch
FP = Failed patch
HC = Hairline crack
C = Crack

FINISH TYPES
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P-01 = Smooth form finish
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(C - Cast in place Concrete
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GENERAL NOTES
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2. Corrosion staining is present in varying degrees at interior ring embed connections, beam embeds and channel embed locations.
3. Skyward facing setbacks are not sloped to shed water. Some appear to have a waterproofing membrane which needs to be replaced.
4. Composite decks at the Gallery, Belfry and Observation deck levels present varying levels of corrosion from severe to moderate.
5. Stair treads, risers and tread angle supports present ongoing corrosion.
6. Stair tread traffic membrane deteriorated and delaminating.
7. See drawings and report for additional notes and observations.
8. These notes are based on cursory observations and are not meant to be an exhaustive survey of the conditions.
9. Drawings may be amended during for detailed construction documents.

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d. Elevations - red notes with round leaders indicate interior observations
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f. SE and NW elevations were inspected close-hand via swing stage. Other were inspected from the ground and tower interior

OWNER:

NAPERVILLE Riverwalk FOR GENERATIONS TO COME

NAPERVILLE RIVERWALK
400 S Eagle St, Naperville, IL 60540

ENGINEER OF RECORD:

ENGINEERING RESOURCE ASSOCIATES

ENGINEERING RESOURCE ASSOCIATES
3s701 West Avenue, Suite 150
Warrenville, IL 60555

MATERIAL CONSULTANT:

e²chem

ECHEM CONSULTANTS, LLC
4 Jefferson Plaza, Suite 500
Poughkeepsie, NY 12601

ARCHITECT:

Brush Architects

BRUSH ARCHITECTS, LLC
4200 N Francisco, Chicago, IL 60618

ISSUE:	DESCRIPTION:	DATE:	DRWN	CHK	APPR
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PROJECT:

MOSER TOWER AND CARILLON
Naperville, IL

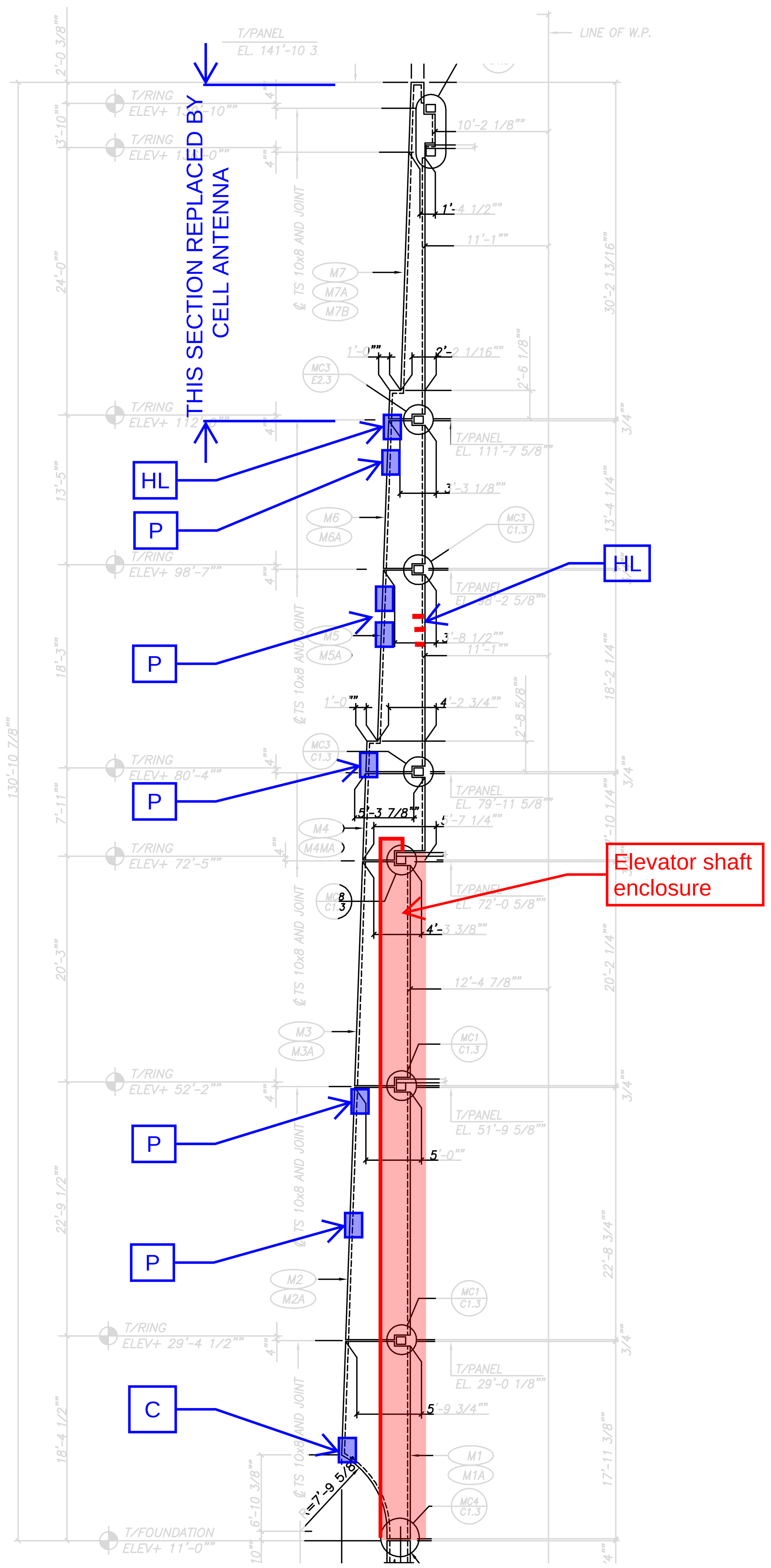
CONDITION ASSESSMENT

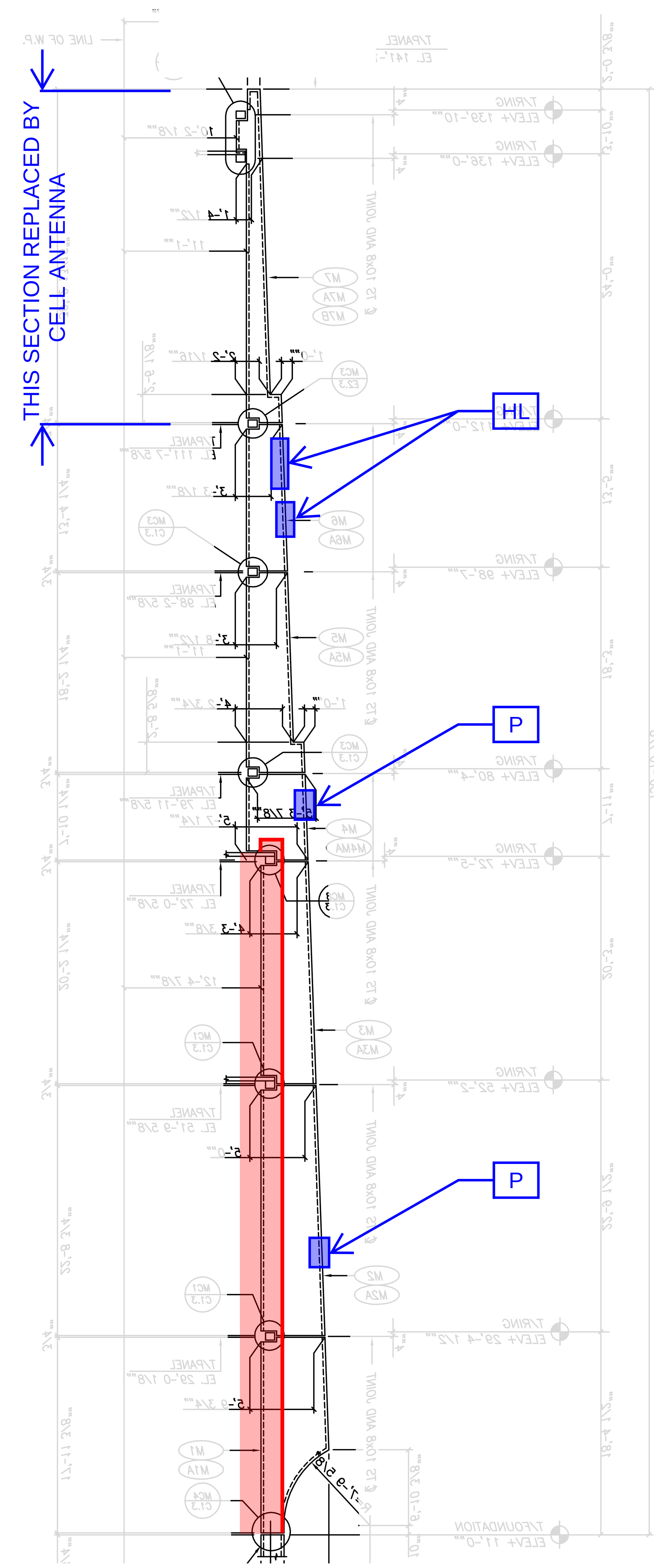
SHEET TITLE:

NORTH CONCRETE FIN ELEVATIONS

PROJECT REFERENCE NUMBER:
88867-002

SHEET NUMBER: A-1.6
REVISION:





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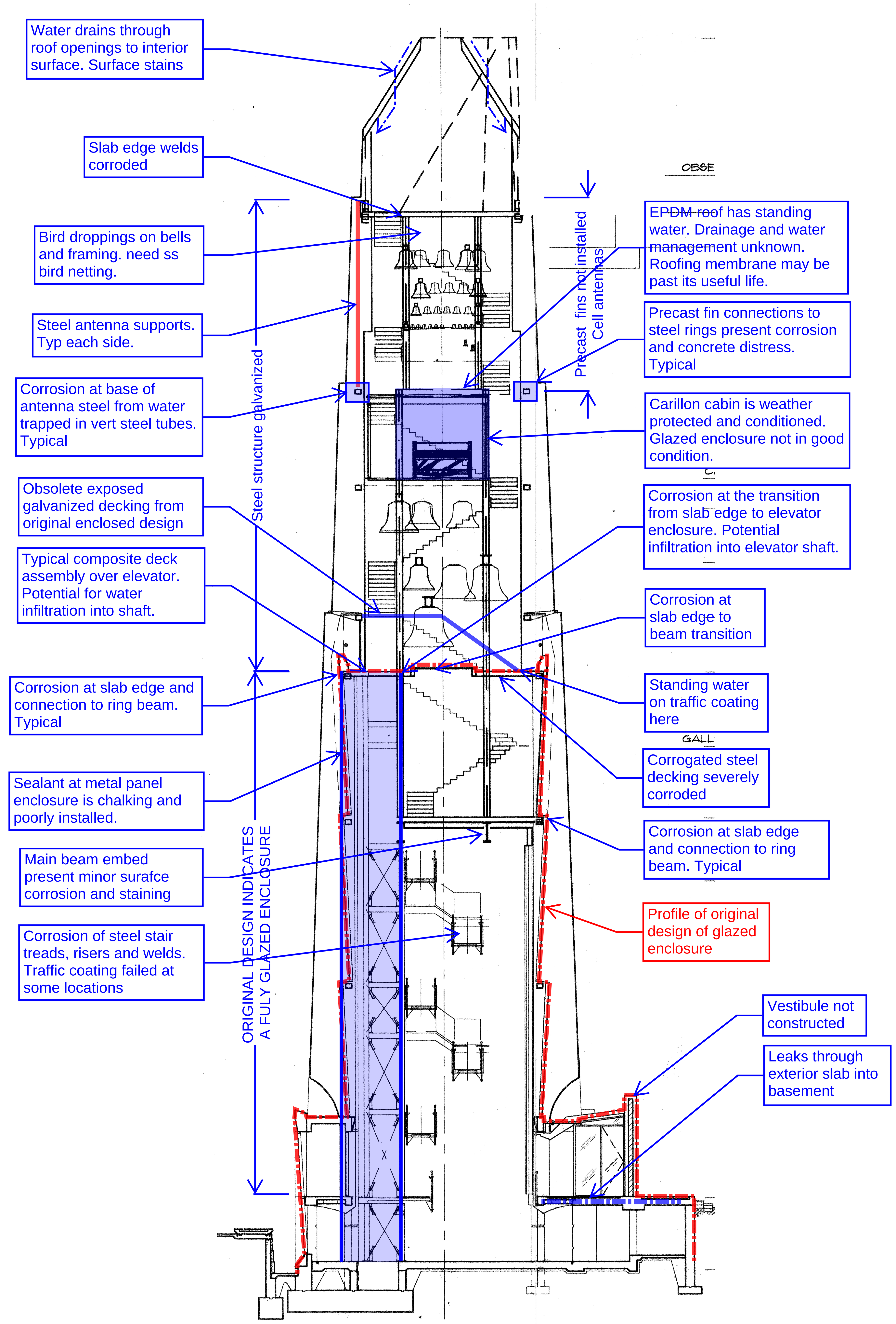
PROJECT:
MOSER TOWER AND CARILLON
Naperville, IL

CONDITION ASSESSMENT

SHEET TITLE:
WEST CONCRETE FIN ELEVATIONS

PROJECT REFERENCE NUMBER:
88867-002

SHEET NUMBER: A-1.7
REVISION:



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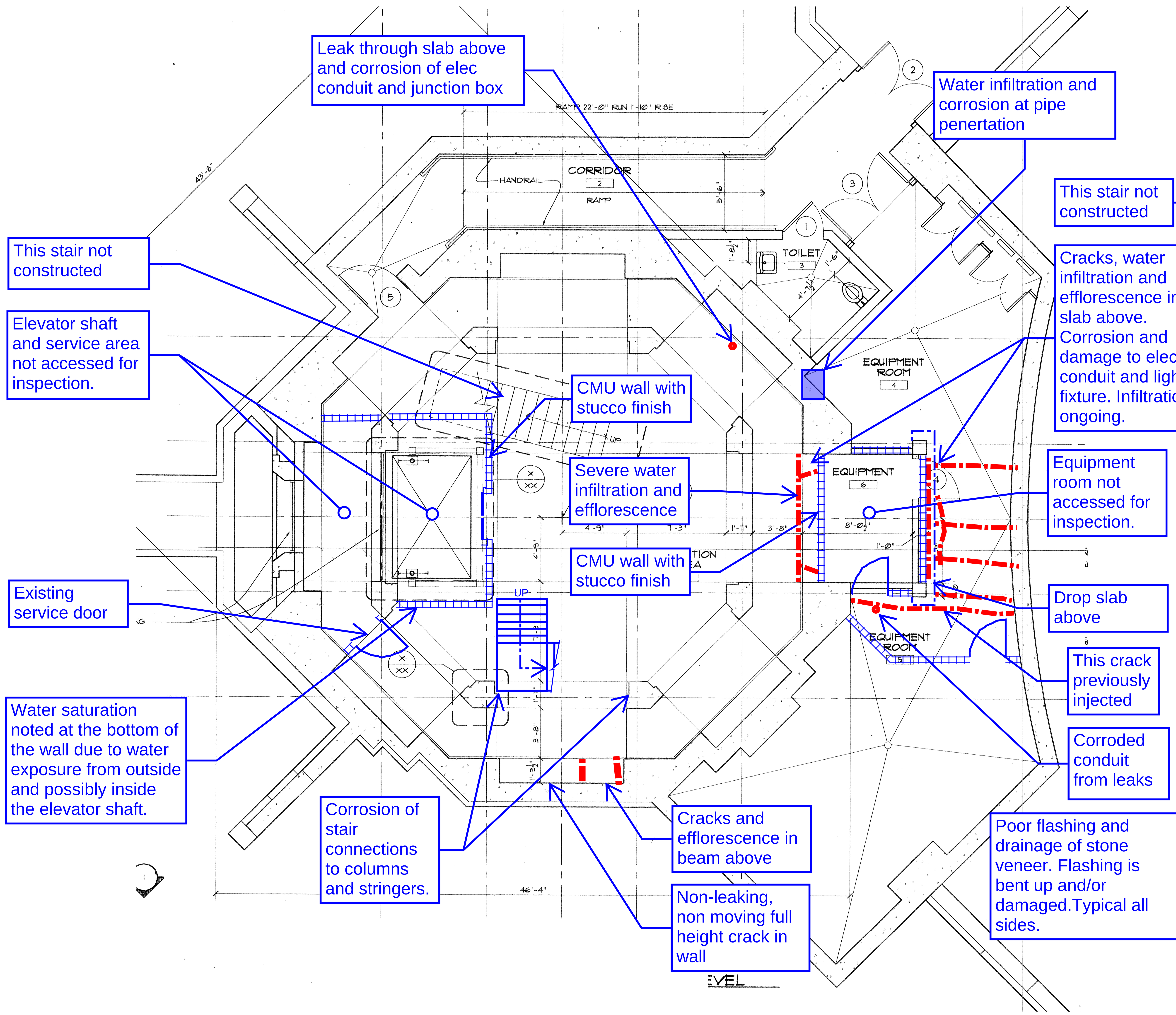
Naperville, IL

CONDITION ASSESSMENT

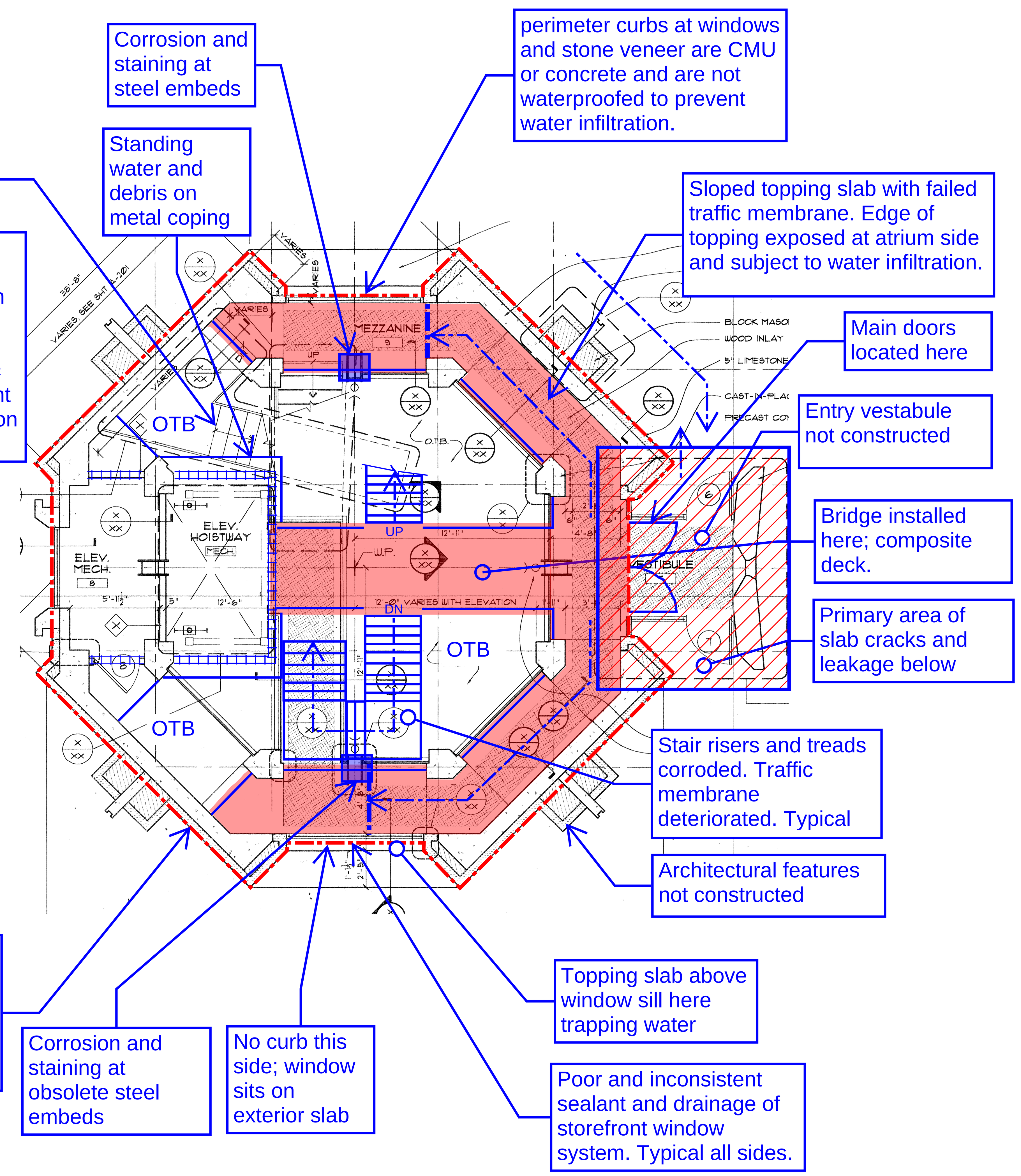
SHEET TITLE:

SECTION LOOKING NORTH

PROJECT REFERENCE NUMBER:
88867-002
SHEET NUMBER: A-2.0
REVISION:



Lower Level Plan



Entry Level Plan

Note: The elevation of the surrounding topping slab is too high, preventing proper drainage of window systems and masonry veneer flashing.

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CONDITION ASSESSMENT

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PLANS

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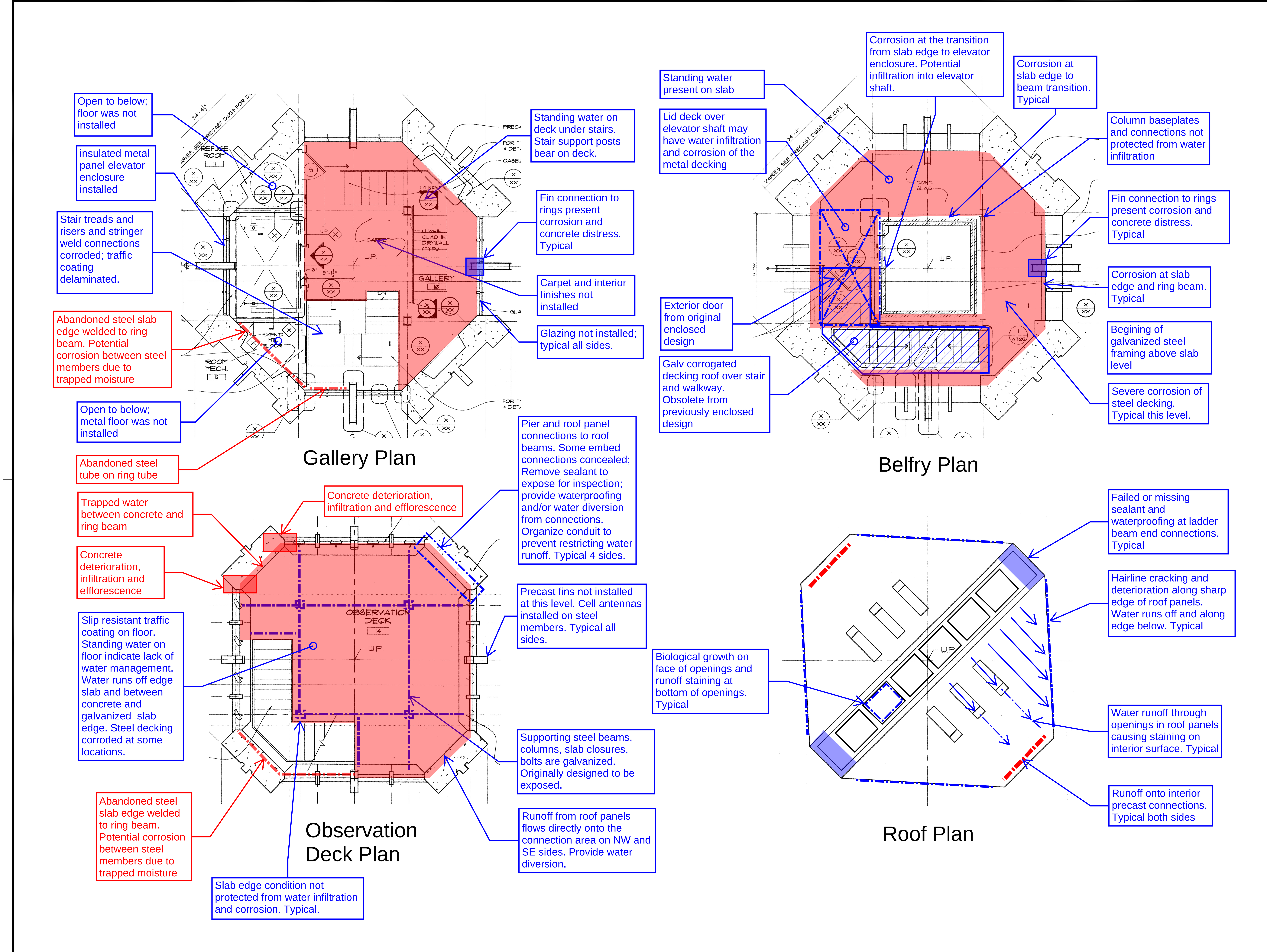
SHEET NUMBER: A-3.0
REVISION:



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