Calculation Cover Sheet

Project/Task: 105-P D&D	Calculation No.: T-CLC-P-00004	Project/Task No.: N/A
Title: Long Term Assessment of 105-P Structure for in-situ D&D Alternatives	Functional Classification: GS	Sheet 1 of 49
	Discipline: Structural Mechanics	
Calc Level: ☒ Type 1 ☐ Type 2	Type 1 Calc Status: ☐ Preliminary ☒ Confirmed	
Computer Program No. NA ☒	Version/Release No.: N/A	

Purpose and Objective

The purpose of this calculation is to evaluate the long-term integrity and condition of the 105-P reinforced concrete structure given different levels of facility preparation and/or long-term maintenance. The results of this calculation, in the form of "timelines," can be used to evaluate the effectiveness of the many alternatives regarding the end state of the facility. The results may provide inputs to the feasibility study.

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(Name and Title)
Date: 8/26/08

Summary of Conclusion

See Section 4.0. Table 4.1 compares some of the important events in all three alternatives considered.

It is recommended that the top three feet of the cap over the reactor vessel be considered sacrificial and not be considered as part of the barrier required for radiological or isolation purposes.

Revisions

Rev. No.	Revision Description
0	Initial Issue

Sign Off

Rev. No.	Originator (Print) Sign/Date	Verification/Checking Method	Verifier/Checker (Print) Sign/Date	Manager (Print) Sign/Date
0	Shawn Carey Shawn Carey 8-6-08	Document Review	J.A. Amin J.A. Amin 8/6/08	W.N. Kennedy W.N. Kennedy 8/26/08

Additional Reviewer -- (Print) N/A	Signature N/A	Date N/A
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Revision History

Revision	Description	Sheets Affected
0	Initial Issue	All

Open Items

There are no open items.

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1 INTRODUCTION AND SCOPE

1.1 Introduction

Preparation is underway for the decommissioning of the 105-P facility. A feasibility study (FS) is being prepared to evaluate several alternatives being considered for the final in-situ decommissioning of the 105-P facility. The proposed plan for decommissioning the facility involves removal of portions of the facility above grade and grouting the portions below grade. However, many alternatives exist within this plan such as how much of the above grade structure to remove, what state to leave the remaining portion, and what, if any, continuing maintenance will be provided. A primary goal of the decommissioning of 105-P is prevention of contamination of the groundwater. As the primary contamination in 105-P is contained in the reactor vessel, the longer one can prevent water from reaching the vessel, the less contamination remains.

1.2 Purpose

The purpose of this calculation is to evaluate the long-term integrity and condition of the 105-P reinforced concrete structure given different levels of facility preparation and/or long-term maintenance. The results of this calculation, in the form of “timelines,” can be used to evaluate the effectiveness of the many alternatives regarding the end state of the facility. The results may provide inputs to the feasibility study. The goal is to provide a conservative, yet realistic estimate of the long-term condition of the structure.

2 INPUT AND ASSUMPTIONS

2.1 Input

Dimensions, reinforcing patterns and material strengths for the 105-P facility are taken from the design drawings. Reference drawings are noted in the body of the calculation.

Functional Classifications

The 105-P facility is currently classified as a GS structure and will remain so after closure.

Loads and Capacities

Load are determined as noted in the body of the calculation. Capacities are determined using the ACI-318 code [1]. However, ϕ factors and load factors are set equal to 1.0 since the interest of this calculation is the collapse of the structure under possible loads, not margin against collapse.

Note that the alternatives identified in this document do not reflect actual conditions but are used as representations of different levels of intervention in order to evaluate the effectiveness of different alternatives.

2.2 Evaluation Alternatives and Assumptions

The long-term integrity and condition of the 105-P facility is evaluated for three different alternatives (A-C) listed below. These are similar to the options suggested in Reference 12:

Alternative A) No intervention, so vegetative growth is allowed on all roofs
Alternative B) Vegetative growth is prevented on roofs over the process room.
Alternative C) Vegetative growth is prevented on all roofs.

For each alternative, the long term integrity and condition of the facility will depend heavily on the assumptions made regarding what preparations are made to the facility and what, if any, periodic maintenance will be performed. The following assumptions apply to all alternatives considered:

1. The stack is removed to elevation +55.
2. The steel superstructure at EL +48 over the shield doors is removed.
3. Shield door slots are capped to prevent water intrusion.
4. The actuator tower is not removed.
5. The above grade disassembly basin structure above El 0.0 is removed up to the expansion joint at column line FR.

6. The reactor tank and the area immediately above it are encapsulated with grout. Integral crystalline waterproofing (ICW) additives will be added to the grout cap to prevent/minimize water infiltration.
7. All areas below grade and the purification cells are filled with grout (void space in piping and equipment is acceptable)
8. All below grade penetrations are sealed to provide isolation and prevent the formation of preferential paths for water intrusion.

Additional assumptions are made for each alternative as listed below:

Alternative A)

1. No special provisions are made to seal above grade penetrations. Only basic measures are taken to prevent human and animal intrusion.
2. Roofs are left essentially “as is”. No preventative maintenance is performed so roof drains will eventually plug, resulting in vegetative growth on the roofs eventually.
3. The grout cap over reactor vessel will be sloped to direct any water that does infiltrate away from the process room into the crane maintenance area.

Alternative B)

1. All above grade penetrations that could provide a preferential path for water infiltration into the Process Room (thick-walled structure with an approximate extent from column line AL-AR and 1-6) such as ventilation ducts, piping, etc. are sealed with the exception of the shield door openings into the crane maintenance area.
2. The grout cap over the reactor vessel will be sloped to direct any infiltrating water away from the process room into the crane maintenance area.
3. For the Process Building (approximate extent from column line AL-AR and 1-6), including the actuator tower, measures are taken to prevent vegetative growth on the flat roofs. This could include removal of parapets, addition of sloping grout, or annual maintenance to ensure roof drains are functioning.
4. Roofs on other portions of the facility (Assembly, Stack, etc.) are left in “as-is” condition, as is assumed in Alternative A.

Alternative C)

1. All above grade penetrations into the 105-P Building are sealed, including ventilation ducts, piping, etc, to prevent the formation of preferential paths for water intrusion.
2. For the entire remaining above grade portion of the facility, including the actuator tower, measures are taken to prevent vegetative growth on the flat roofs. This could include removal of parapets, addition of sloping grout, or annual maintenance to ensure roof drains are functioning.
3. The grout cap over the reactor vessel will be sloped to direct any infiltrating water away from the process room into the crane maintenance area.

3 METHODOLOGY

3.1 Degradation mechanisms

Many degradation mechanisms have the potential to impact the 105-P reinforced concrete structure after decommissioning. These include, but are not limited to, freeze/thaw, leaching, steel reinforcing corrosion, cracking due to seismic and other NPH loads, vegetative growth, etc. All known degradation mechanisms are reviewed and the controlling mechanisms are identified.

3.2 Material properties

Material properties such as concrete and steel reinforcing strength are taken from the design drawings. A comparison to existing testing programs of concrete of similar age and a review of the current condition of the facility are used to validate the design material properties are appropriate.

3.3 Evaluation Methodology

The prediction of the long-term condition of the 105-P facility is a complex problem. Over time, no matter the preparation taken at final decommissioning, eventually the structure will collapse or erode away. After that degradation mechanisms will continue to decompose the rubble and grout block left in the ground. Since the 105-P structure is a complex building, it is extremely difficult to predict how failure may occur. However, general engineering principles are used to make engineering judgments about the general order of and time to collapse. For example, degradation will decrease the capacity of any element, so the lower the demand to capacity (D/C) ratio an element has, the greater degradation the element could withstand before collapsing.

The structure can be divided into several portions that will act independent of other portions for the most part. Each portion can be analyzed by itself, and the results combined with the results from other portions to form a complete picture of the condition of the facility.

The general portions of the structure that are considered are the process room, areas north, south, east, and west of process room, assembly area, purification, and stack. These general areas these portions cover is shown in Figure 3.1. Figure 3.2 shows a cross-section through the structure and Figure 3.3 shows a recent aerial photograph of the facility.

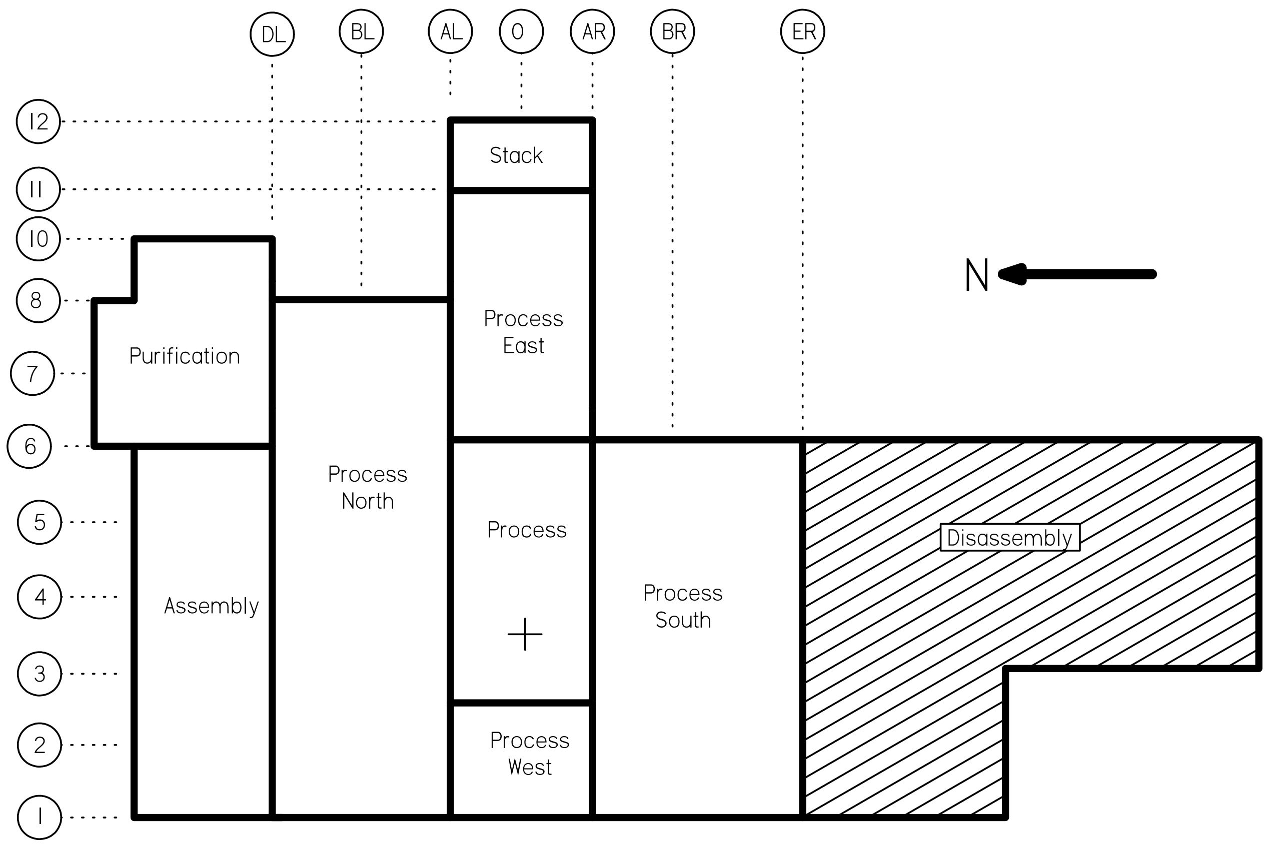

Figure 3.1 General Areas/ Layout of 105-P Facility.

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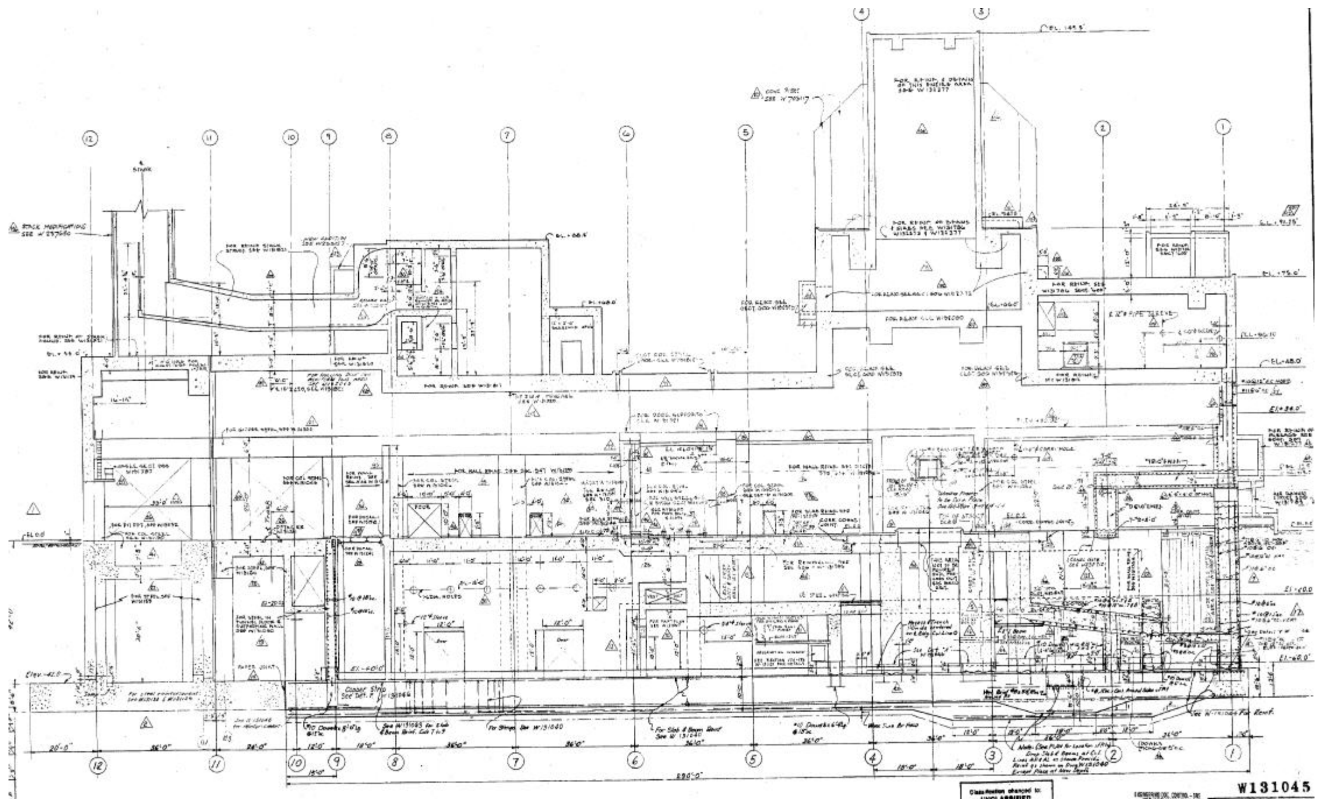

Figure 3.2 Cross-Section of 105-P Facility (looking south).

Figure 3.3 Aerial Picture of 105-P Facility.

4 RESULTS AND CONCLUSIONS

This calculation evaluated the long-term integrity and condition of the 105-P reinforced concrete structure for three different alternatives representing different levels of facility preparation and/or long-term maintenance. The results of this evaluation is presented in the form of “timelines,” that can be found in Section 6.5. Table 4.1 below compares some of the important events in all three alternatives considered.

Due to low D/C ratios, a catastrophic collapse is not expected. Therefore, large debris is not expected to fall on the cap over the reactor vessel. However, due to the potential for spalling and erosion due to water, it is recommended that the top three feet of the cap be considered sacrificial and not be considered as part of the barrier required for radiological or isolation purposes.

The below grade portion of the structure, essentially a large block of grout, is expected to remain essentially intact, with the possible exception of cracks forming at the expansion joint locations. As expansion joints are located far away from the reactor vessel, this is not a concern.

Table 4.1 Comparison of times to significant events (years) for three alternatives.

Event	Alternative A	Alternative B	Alternative C
Roofs away from Process room begin to collapse	150	150	1350
Water infiltration into Process Room due to roof degradation/collapse	200	1400	1400
Water infiltration through slab directly over reactor vessel	225	1550	1550
Cap exposed due to roof collapses	400	1700	2700
Only rubble left above grade	1000	1000 (all but Process Room structure) 2500 (Process Room)	2500

Note: For more detailed information, see Section 6.5.

5 REFERENCES

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9) Oak Ridge National Laboratory. (2007). "Primer on Durability of Nuclear Power Plant Reinforced Concrete Structures - A Review of Pertinent Factors", NUREG CR/6927.
10) P-Area Historical Groundwater Data. Excel sheet provided by SRS SGCP, June 2008.
11) PDCS-SEG-2008-00022 "105P Closure – Structural Input to Feasibility Study" G.R. Baldwin. April 2008.
12) PDCS-SEG-2008-00036 "Structural Mechanics Scope for Input to 105-P D&D Feasibility Study" S.A. Carey. July 2008.
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14) Shuman R., Roger, V.C., Shaw, R.A. (1989). "The Barrier Code for Predicting Long-Term Concrete Performance". *Waste Management 89*, University of Arizona.
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17) T-CLC-K-00236, "KAC Process Building Vertical Soil-Structure Interaction Analysis", Revision 0.
18) T-CLC-P-00003, "Building 105P, 2006 Structural Integrity Review of 105P", Revision 0.
19) T-CLC-R-00001, "2007 Structural Integrity Review of Building 105-R", Revision 0.
20) U.S. Army Core of Engineers, (1996). "Petrographic Investigation of Concrete from Building 221-H, Savannah River Site".
21) Walton, J.C., Plansky, L.E., and Smith, R.W. (1990). "Models for Estimation of Service Life of Concrete Barriers in Low-Level Radioactive Waste Disposal". NUREG/CR-5542.

6 CALCULATIONS

6.1 In-situ Material Properties

The 105-P building was constructed in the early 1950's to Specification 3019 [15]. Specification 3019 and the design drawings for 105-P call for 2500 psi concrete and 40 ksi reinforcing steel to be used. While concrete core bore and steel rebar coupon tests have not been performed for 105-P, these tests have been performed on sample taken from F- and H-canyons which were built to the same Specification 3019 as the reactor buildings.

The results from these tests indicate the walls and slabs have a median strength concrete strength of 4080 psi and the concrete columns have a median strength of 2500 psi. The median yield strength of the reinforcing steel was 44.6 ksi [16]. Petrographic testing was also performed on the concrete samples taken from both the exterior and interior of the canyon structures. The testing indicated that alkali-silica reaction was not present and only minor carbonation had occurred [20].

Assessments of the present state of the 105-P facility indicate the exterior of the structure is generally in good condition with little signs of degradation observed. Efflorescence was observed in local areas, but no rust stains were observed indicating corrosion of the reinforcing steel. The only major degradation inside the facility was around the dry cave area in the disassembly area that will be removed.

Based on these inputs, it is appropriate to use the design values of 2500 psi concrete and 40 ksi reinforcing steel. The use of design values rather than in-situ values will give slightly more conservative results, but this is judged acceptable.

6.2 Controlling degradation mechanisms

All reinforced concrete structures will eventually deteriorate and fail, though the degradation rate may be slow. There are several well-known examples of large stone and concrete building around the world that are at least several thousand years old. These include the pyramids in Egypt, the Pantheon in Rome (among other Roman structures), and Angkor Wat in Cambodia. However, these structures rely entirely on the concrete or stone to support the loads on the structure. Modern reinforced concrete relies on the concrete to carry compressive loads and steel reinforcing to carry tensile loads. The concrete protects the reinforcing steel from exposure to the elements and subsequent corrosion. Since the corrosion products take up more volume than the uncorroded steel, concrete will typically spall, exposing more steel for corrosion. Therefore concrete typically degrades quickly once the reinforcing steel begins to corrode. The concrete itself can also degrade due to weather or chemical attack.

Many degradation mechanisms can affect reinforced concrete. Depending upon the environment around the concrete and whether the concrete is above or below grade, different mechanisms may control. Known degradation mechanisms of concrete are discussed below and applicability to the 105-P is determined. Much of the discussion of degradation mechanisms is taken from Walton et al. (1990) [21].

6.2.1 Concrete below grade

Concrete below grade will be exposed continually to soil and ground moisture/water, but protected from atmospheric conditions. The following degradation mechanisms can occur:

Sulfate and Magnesium Attack

Sulfur in groundwater reacts with tricalcium aluminate (C_3A) to form calcium aluminum sulfates. A related problem is the reaction of magnesium with the cement to form Brucite [$Mg(OH)_2$]. The products of these reactions have considerably greater volume than the compounds they replace, which leads to expansion and disruption of the concrete. The depth of surface deterioration can be approximated by the following equation:

$$x = 0.55(C_s)(Mg^{2+} + SO_4^{2-})(t) \qquad \text{(Eq 6.1a)}$$

where: x = depth of deterioration (cm)

C_s = weight percent of C_3A in unhydrated cement

Mg^{2+}, SO_4^{2-} = concentration in bulk solution (mol/L)

t = time (yr)

From data taken from monitoring wells around 105-P since 1990, the average sulfate concentration is 7.17 mg/L and the average magnesium concentration is 1.12 mg/L in the groundwater [10]. Typical portland cement contains a maximum of 12% C_3A [8]. Using these values, the equation reduces to

$$x = 7.973\text{e-}4(t) \qquad \text{(Eq. 6.1b)}$$

Since the below grade portion of the facility will be filled with grout, the entire below grade portion of the facility can be considered as a large block of grout. The reactor vessel, with a bottom elevation of -20, will be at more than 30 ft from the bottom of the basemat at a minimum and even further from the edges of the grout block. Therefore thousands upon thousands of years would pass before sulfate and magnesium attack degraded the concrete to the reactor vessel.

Alkali and Calcium Hydroxide Leaching

As concrete is exposed to water, cement compounds will be leached from the concrete. The leaching process can be described in four stages (Atkinson (1985) and Atkinson et al. (1988)):

1) Initially the pH is around 13.5 due to the presence of alkali metal oxides and hydroxides. The alkali metals are the first components to leach from the concrete.

2) After the alkali metals are leached, the pH is controlled at 12.5 by solid $Ca(OH)_2$.

3) Following the loss of calcium hydroxide, the calcium silicate hydrate (CSH) gel phases begin to dissolve incongruently while the pH slowly moves down to 10.5. During this period, the calcium to silicon ratio drops to 0.85.

4) In the final phase, the pH is held at 10.5 by congruent dissolution of the CSH gel.

The leaching of calcium hydroxide tends to lower the strength of the concrete 1.5% for every 1% of calcium lost (Lea, 1970). This decrease in strength would only affect the portion of the concrete from which the calcium leaching has occurred. However, both sophisticated and simplistic models suggest that leaching beyond the very surface of the concrete requires thousands of years. For example for the average Ca^{2+} concentration of 3.8 mg/L in the groundwater around 105-P, leaching would have only occurred in the outer 0.05 cm of the concrete. Based on this evidence, leaching is not considered in this collapse analysis. This is further justified because the fact that the concrete design strength is used rather than the in-situ strength.

Carbonation

Carbonation is the reaction of carbon dioxide (CO_2) with cement to form calcium carbonate. In general carbonation in itself does not cause degradation of the concrete. In some cases the formation of calcium carbonate may slow the migration of radionuclides through solid solution reactions. However a negative side effect also occurs due to carbonation - a drop in pH. Carbonation lowers the concrete pH towards neutral, from 12 to about 8. Typically the reinforcing steel is protecting by a thin passivating layer that forms in the alkaline (high pH) environment of the concrete. This layer can break down if the pH decreases due to carbonation increasing the likelihood of rebar corrosion.

At high relative humidities, which are typical of subsurface conditions, carbonation occurs slowly since the CO_2 must be transported in the liquid phase rather than the gas phase. While the lower pH can lead to faster accelerated corrosion of the reinforcing steel, it takes thousands of year for carbonation to reach the depth of the reinforcing steel. Also, since the below grade portions of the structure will be grouted, it does not matter if the reinforcing steel corrodes. Therefore carbonation is neglected in the below grade degradation of concrete.

Alkali-Silica Reaction

As discussed in Section 6.1, petrographic testing indicates that alkali silica reaction is not present in the concrete used for 105-P.

Freeze-thaw

Due to the mild climate at the Savannah River Site, freezing does not occur below grade. Therefore, for the below grade portion of the structure, freeze-thaw damage is not a concern.

Cracking

Quality concrete has a very low hydraulic conductivity, typically on the order of 1e-8 to 1e-10 cm/s. The presence of just a few cracks, however, can increase the hydraulic conductivity by several orders of magnitude [13]. Cracks in the concrete allow water to penetrate beyond the exposed surface allowing degradation mechanisms to act on concrete further inside.

Cracking can be caused by many things. In mass concrete pours, such as the very thick reactor walls and slabs, cracking can occur during the curing process due to the heat of hydration. Specification 3019 allowed for the addition of pozzolans to the mixture to slow curing and minimize cracking. After concrete has cured, cracking can occur due to long-term concrete shrinkage, thermal loads, and operating loads. Inspection of the structure [18] indicates only minor cracking exists. Any water infiltration in the below grade portion of the structure has been linked to degraded expansion joints.

ACI 318 [1] (present and past) contains provisions for the minimum amount of reinforcing steel required to minimize and control cracking. A comparison of the provided reinforcing ratio to the ACI minimum ratio in Table 6.1 shows that the building is more than adequately reinforced to minimize and control cracking due to normal loads.

Cracking may also occur due to differential settlement of the structure. Any settlement due to the initial construction of the structure would have already occurred. Most settlement due to construction would have been uniform, but due to uneven foundation pressure some differential settlement may have occurred. Some differential settlement has been observed, but primarily around expansion joints and no large cracks have been observed [18]. The filling of the below grade portions of the structure with grout will add a large load to the foundation so additional settlement would be expected. However, this would be fairly uniform due to the uniform load from the grout. The largest contributor to differential settlement over long time periods is post-seismic differential settlement. Cracks would be most likely to occur at the expansion joints which are far away from the reactor vessel. The addition of an integral crystalline waterproofing (ICW) admixture will allow for self-healing of minor cracks if exposed to water.

Table 6.1 Comparison of provided reinforcing to code minimum.

ABOVE GRADE	WALL THICKNESS (ft)	REINFORCING PROVIDED	RATIO OF REINFORCING TO CONCRETE AREA PROVIDED	CURRENT CODE RATIO REQ'D FOR CRACK MINIMIZATION PER ACI 318
	1	#6@12" EWEF	ρ_v = 0.0061 ρ_h = 0.0061	ρ_v = 0.0015 ρ_h = 0.0025
	3	#10@7" EF Vert. #10@8" EF Horiz.	ρ_v = 0.0101 ρ_h = 0.0088	ρ_v = 0.0015 ρ_h = 0.0025
	5	#11@6" EF Vert. #10@12" EF Horiz.	ρ_v = 0.0087 ρ_h = 0.0035	ρ_v = 0.0015 ρ_h = 0.0025
BELOW GRADE	4	#11@6" EF Vert. #10@18" OF Horiz full height #10@9" IF Horiz, 0 to -10' #10@6" IF Horiz. -10'to-20' #11@6" IF Horiz. -20'to-30' #11@5" IF Horiz. -30'to-35' #11@4" IF Horiz. -35'to-40'	ρ_v = 0.0108 ρ_h = 0.0044 ρ_h = 0.0059 ρ_h = 0.0069 ρ_h = 0.0080 ρ_h = 0.0096	ρ_v = 0.0015 ρ_h = 0.0025
	6	#11@6" EF Vert. #9@12 EF Horiz	ρ_v = 0.0072 ρ_h = 0.0023*	ρ_v = 0.0015 ρ_h = 0.0025
	7	#11@6" OF Vert. #10@6" IF Vert. #11@6" EF Horiz. 0'to-30' #10@12" EF Horiz. -30'to-40'	ρ_v = 0.0056 ρ_h = 0.0062 ρ_h = 0.0025	ρ_v = 0.0015 ρ_h = 0.0025

* The horizontal reinforcement for the 6' thick, below grade walls is slightly less than the ACI 318 minimum reinforcing requirement for crack control in walls, although the difference is insignificant.

Summary

In summary, the degradation mechanisms that will act on the below grade portion of the 105-P structure will occur very slowly. The controlling degradation mechanism for the breakdown of the concrete over time would be sulfate and magnesium attack. The largest contributor to cracking would be differential settlement due to earthquakes. All other degradation mechanisms are neglected for the rest of this calculation.

6.2.2 Above grade degradation

The above grade portion of the structure is primarily exposed to atmospheric conditions rather than soil and groundwater conditions. Therefore, degradation mechanisms that depend on groundwater exposure, such as sulfate and magnesium attack and leaching do not generally apply. Other degradations mechanisms including the important issue of vegetative growth on the flat roofs of the facility that can occur are discussed below:

Alkali-Silica Reaction

As discussed in Section 6.1, petrographic testing indicates that alkali silica reaction is not present in the concrete used for 105-P.

Freeze-thaw

Freezing and thawing cycles have the potential to degrade concrete. The damage is primarily related to hydraulic pressure and ice accretion. As water expands during freezing, it can exert a large hydraulic pressure on the concrete pores. Ice accretion lowers the vapor pressure in large pores resulting in additional forces. Only concrete elements subjected to continuous or frequent wetting are susceptible to damage by freeze-thaw cycling. For the mild climate at SRS, on average, there are 36 days a year where the temperature falls below freezing. For this small number of cycles, freeze-thaw damage would be minimal. In fact, Barrier code equations [14] require a minimum of 50 freeze-thaw cycles before damage occurs. Visual inspection of the 105-P structure indicates no freeze-thaw damage has occurred during the more than 50 year life of the structure so far. Based on this, freeze-thaw damage is neglected for the above grade portion of the structure as well.

Carbonation

As discussed previously, carbonation is the reaction of carbon dioxide (CO_2) with cement to form calcium carbonate. Carbonation lowers the concrete pH towards neutral, from 12 to about 8 which increases the likelihood of reinforcing bar corrosion. Since the above grade portion is not grouted, corrosion of the reinforcing steel can lead to eventual collapse of the structure.

Unlike below grade structures where the presence of groundwater forces CO_2 to be transported slowly in the liquid phase, CO_2 can be transported much faster in the gaseous phase to above grade concrete leading to much faster carbonation rates. Many factors can affect the rate of carbonation, including temperature, relative humidity, and composition of the cement paste. It is generally accepted that the rate of carbonation at exposed surfaces is roughly proportional to the square root of time for concrete kept continuously dry at normal relative humidities [9]. i.e. an interior exposure. Carbonation rates for exterior exposure are generally lower due to the presence of more moisture (precipitation).

Carbonation depth data is not presently available for 105-P, but a model can be estimated using the results of testing performed on samples taken from the interior columns of H-Canyon in 1996 [20]. Phenolphtalein testing indicated carbonation had occurred to a depth of 0.375 inches. It should be noted that this is based upon interior exposure. As noted above, carbonation rates are less for exterior exposure. However, the interior concrete sample taken from H-Canyon had been painted, which may tend to slow down the diffusion rate of CO_2 thus decreasing the carbonation rate. For the purposes of this calculation, it is assumed that exterior carbonation rate can be approximated by the rate of carbonation of interior painted concrete.

Using the data point of time, t, equals 45 yrs and depth of carbonation, d_c, equals 0.375 inches, an equation for the depth of carbonation over time can be found. If $d_c = A\sqrt{t}$, then $0.375 = A\sqrt{45}$ and A = 0.0559. Therefore the depth of carbonation over time can be approximated as:

$$d_c = 0.0559\sqrt{t} \qquad \text{(Eq. 6.2)}$$

where d_c is in inches and t is in years.

Rebar Corrosion

Rebar corrosion can occur due to two mechanisms: oxic corrosion and anoxic corrosion. In addition to reducing the cross-sectional area of steel, the corrosion byproducts take up more volume. This can lead to cracking and spalling of the concrete.

Oxic corrosion occurs as oxygen is electrochemically reduced and iron is converted to iron oxide. In the alkaline environment of standard concrete, a passivating layer is formed around the reinforcing steel that protects the steel from oxic corrosion. However, this passivating layer can be weakened if the pH is lowered or chloride ions diffuse to the reinforcing steel. Oxic corrosion therefore occurs in two steps.

First the passivating layer protecting the reinforcing must be broken down. This can happen due to carbonation or chloride ion penetration. The time it takes carbonation to reach the reinforcing steel can be found using Eq 6.2. The time to onset of oxic corrosion due to chloride ion penetration can be approximated by:

$$t_c = \frac{129 x_c^{1.22}}{(WCR)(Cl^{0.42})} \qquad \text{(Eq. 6.3)}$$

where: t_c = time to onset of oxic corrosion, (yr)

x_c = thickness of concrete cover over rebar (in)

WCR = water to cement ratio (by mass)

Cl = chloride ion concentration in groundwater (ppm)

However, since SRS is not a marine environment, the chloride content of precipitation is negligible so carbonation will likely control the depassivation of the reinforcing steel. Next, the oxic corrosion reaction then proceeds with a loss of reinforcing steel that can be approximated by:

$$\%remaining = 100\left(1 - \frac{4\left(9.4\frac{cm^3}{mole}\right)(s)(D_i)(C_{gw})(t - t_c)}{\pi(d^2)(x_c)}\right) \qquad \text{(Eq. 6.4)}$$

where: s = spacing between reinforcing bars

D_i = oxygen diffusion coefficient in concrete (cm^2/sec)

C_{gw} = oxygen concentration in groundwater

t = time (sec)

t_c = time to onset of oxic corrosion, (sec)

d = diameter of rebar (cm)

x_c = thickness of concrete cover over rebar (cm)

This equation was developed for below grade concrete continuously exposed to groundwater (i.e. saturated), so its applicability to above grade concrete is questionable.

Rebar corrosion can also occur due to anoxic corrosion (hydrogen evolution reaction). In this mechanism, the H^+ ion from the water molecule is used as the source of oxidant for corrosion [6]. Anoxic corrosion can be approximated simply by a corrosion rate (i.e. cm/yr). A literature review by Brandstetter and Lolcama indicated the anoxic corrosion rate was dependant on pH. For a high pH environment, typical of concrete, the reaction rate is on the order of 1.5e-4 to 1e-5 cm/yr. If the pH drops below 9, the reaction rate increases to values on the order of 1e-3 to 1e-4 cm/yr. As this depends on a water saturated environment, its applicability to above grade concrete is questionable.

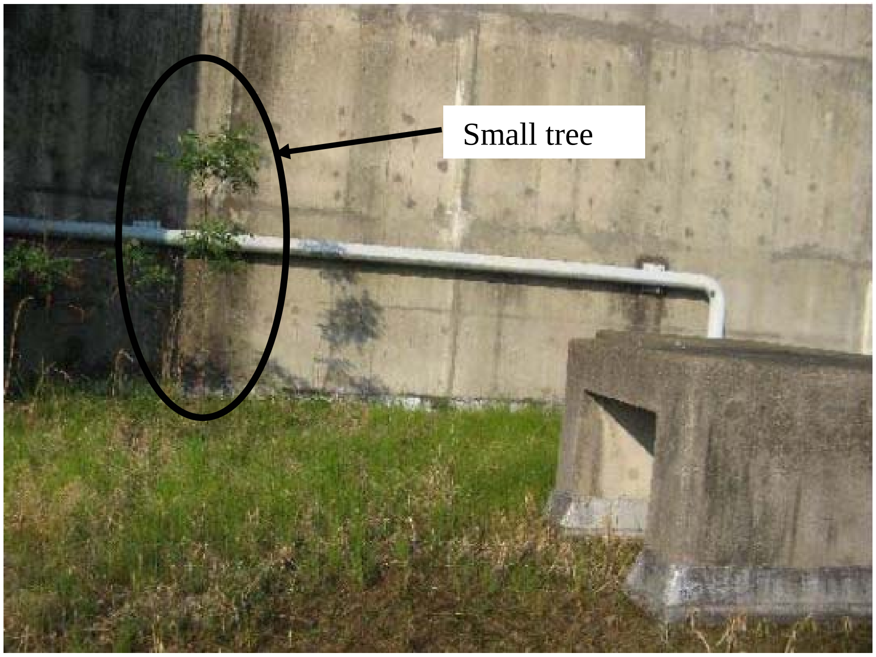
As seen from above, the loss of reinforcing steel in intact concrete is fairly slow. Even more detrimental to the concrete than the loss of steel volume is the increased volume of the corrosion products which can lead to cracking and spalling of the cover concrete, exposing the reinforcing steel leading to greatly accelerated corrosion due to direct exposure to the elements. Based on this, it is judged that once the reinforcing steel has become depassivated, the steel on the exterior side of the roof or wall (exposed to the elements) will not last more than 150 years. The reinforcing side on the interior side of the roof or wall will not corrode as fast since it is not exposed directly to the elements, so it is judged it will last 300 years at most.

Vegetative Growth

A degradation mechanism that is very difficult to quantify is vegetative growth. Almost all of the roof slabs for the 105-P structure are flat with parapets and embedded roof drains. With no ongoing maintenance, it is inevitable the roof drains will clog. The parapets will then allow for water to be retained on the flat roof. This will promote the growth of algae, moss, and eventually a soil layer will form that can support grasses, bushes and trees. The soil will further retain moisture which could further degrade and possibly cause cracking on the surface of the concrete. Roots will penetrate into cracks breaking up the concrete and exposing the reinforcing bars. This is similar to the natural weathering process where vegetation can take root in small cracks in rocks and eventually split them apart and the rock is weathered into soil.

The length of time for this to occur is very difficult to gauge. Lack of maintenance on R-reactor has allowed small trees and grasses to grow on the roof in certain areas as shown in Figure 6.1 [19]. This has occurred in less than 25 years. However, this vegetative growth has not yet caused any water infiltration or rebar corrosion according to visual observation. However, water infiltration and rebar corrosion would eventually occur.

For the purposes of this calculation the roof drains are assumed to be plugged from day 1. It is judged that if vegetative growth is allowed, a roof slab would allow significant water infiltration and at least partially collapse within 150 to 250 years. Slabs with little excess capacity would fail sooner than slabs with greater excess capacity.



a) El +34 near Elevator #2.

b) El +48 under shield door frame

Figure 6.1 Vegetative growth of roof of 105-R [19].

Cracking

As discussed previously, quality concrete has a very low hydraulic conductivity, typically on the order of 1e-8 to 1e-10 cm/s. The presence of just a few cracks, however, can increase the hydraulic conductivity by several orders of magnitude [13]. Cracks in the concrete allow water to penetrate beyond the exposed surface allowing degradation mechanisms to act on concrete further inside.

Cracking can be caused by many things. As presented in Table 6.1 previously, the above grade walls contain more than the minimum amount of steel as required by current design codes. Inspection of the structure indicates only minor cracking exists [18]. Most water infiltration in the above grade portion of the structure has been linked to degraded expansion joints.

Since the below grade portion of the structure will be filled with grout, the above grade portion of the structure is supported on a large block of grout/concrete. Cracking due to settlement is unlikely to occur in this arrangement. No large settlement cracks have been observed in the structure [18].

Summary

In summary the controlling degradation mechanisms for above grade concrete is vegetative growth if it allowed. At least partial collapse would be expected in no more than 250 years depending upon the excess capacity. If vegetative growth is not allowed, carbonation and eventual corrosion of the steel would control. Once carbonation has reached the reinforcing steel (per Eq 6.2), rebar corrosion would commence and the top steel exposed to the elements would not last more than 150 years. The interior steel would not be expected to last more than 300 years after carbonation has taken place.

6.3 Below Grade Evaluation

The evaluation of the below grade structure is the same for all alternatives considered. Any penetrations in the below grade structure will be sealed to prevent the formation of preferential flow paths for water intrusion. As discussed in Section 6.2.1, sulfate and magnesium attack is the primary degradation mechanism that could degrade the concrete. As shown by Eq. 6.1b, however, this rate is very slow. After 5000 yrs, the amount of concrete lost due to sulfate and magnesium attack is only 5000 x 0.0007973 = 3.99 cm = 1.57 in. This is insignificant compared to the wall thickness of several feet, not to mention the grout fill beyond that. The distance from the reactor tank to the edge of the process building or an expansion joint is at least 90-ft.

It should also be noted that the minimum depth to the ground water table around the 105-P structure is about 37-38 ft [10]. This places it just above the floor level at El -40, but still below the bottom of the reactor tank at El -20. So even if a preferential water path did exist below grade, water would not tend to flow from the water table up to the reactor vessel.

Differential settlement is the other mechanism that could cause cracking of the grout block below grade and allow water intrusion. The building would be able to span over local areas of differential settlement. As the width of settlement increased, a point would be reached eventually where the grout block could no longer span over the settlement and cracking could occur.

Determine the width of a zone the grout block could span under its own weight plus weight of build above.

Grout block depth: $d := 50\text{ft}$ (40 ft plus 10 ft foundation)

grout unit weight: $w_g := 130\text{pcf}$

concrete strength: $f_c := 2500\text{psi}$

modulus of rupture: $f_r := 7.5\sqrt{f_c \cdot \text{psi}}$ $\quad f_r = 375\,\text{psi}$

shear capacity: $f_v := 2\sqrt{f_c \cdot \text{psi}}$ $\quad f_v = 100\,\text{psi}$

moment of inertia: (1-ft strip) $I := \frac{1}{12} \cdot 1\text{ft} \cdot d^3$ $\quad I = 1 \times 10^4\,\text{ft}^4$

estimated bearing pressure under reactor building $w_{dr} := \frac{312805\text{kip}}{(250\text{ft})^2}$ $\quad w_{dr} = 5.005\,\text{ksf}$

(based on weight of K-reactor process building)

For a fixed-fixed span: (1-ft strip) $w_u := \left(w_g \cdot d + w_{dr}\right) \cdot 1\text{ft}$ $\quad w_u = 11.505\,\frac{\text{kip}}{\text{ft}}$

$$M_u(L) := \frac{w_u \cdot L^2}{12} \qquad V_u(L) := \frac{w_u \cdot L}{2}$$

$$\sigma_u(L) := \frac{M_u(L) \cdot d}{2 \cdot I} \qquad \sigma_v(L) := \frac{V_u(L)}{d \cdot 1\text{ft}}$$

Find max span to reach moment capacity, neglect any rebar

$L := 1\text{ft}$

Given

$f_r = \sigma_u(L)$

$L_{um} := \text{Find}(L)$ $\quad L_{um} = 153.194\,\text{ft}$

Calculation Continuation Sheet

Calculation No.	Sheet No.	Rev.
T-CLC-P-00004	26	0

Find shear stress at this span, for shear, span can be reduced by 2 x depth. Since depth is so larg
only use one times depth

$$\sigma_v\left(L_{um} - d\right) = 82.447\,\mathrm{psi}$$ < 100 psi, OK

Regardless of the settlement depth, the grout block can support loads over a span of approximately 150-ft. This is more than half of the dimension of the footprint of the building. Assuming the design bearing capacity is similar to K-area (15 ksf) [6], it is likely the bearing capacity of the soil would be exceeded and additional settlement would occur resulting in greater support. The expected differential settlement due to a PC-3 or PC-4 level event would be on the order of only a few inches. It is concluded that settlement will have little impact on the grout block and rigid body rotation will occur instead of cracking. Only an extremely large differential settlement would challenge the integrity of the grout block.

6.4 Above Grade Evaluation

This section contains evaluations for the above grade structure. Capacities of different elements are compared to determine a general order of degradation. The general way in which the structure would likely degrade and eventually collapse is discussed. Also discussed are the effects of seismic loads on the structure. Controlling degradation mechanisms, as determined in Section 6.2.2 are applied to the three alternatives evaluated.

6.4.1 Time to carbonation

The concrete cover for the 105-P structure is 2 inches [15]. Given Equation 6.2 and the 55 year existence of the structure, the length of time before the reinforcing bars become depassivated due to carbonation is roughly 1200 years as shown below:

$$d_c = 0.0559\sqrt{t} \Leftrightarrow t = \left(2/0.0559\right)^2 = 1280\,years$$
$$1280\,years - 55\,years \approx 1200\,years$$

This estimate neglects any additional time due to protection given by the roofing or any possible grout cover.

6.4.2 Roof/Slab Capacities

As discussed in the methodology, the lower the demand to capacity (D/C) ratio, the longer an element will likely remain intact. The slabs are evaluated in this section in order to gauge a rough idea on the possible order in which the roof slabs may develop significant cracks, allowing water infiltration and eventually collapse, Slabs are evaluated as one way slabs under dead load plus a nominal load representative of vegetative growth. Since the negative moment reinforcing steel at the beam supports is closer to the elements and therefore more likely to corrode, it is neglected. This makes the slab effectively a simply supported beam. For the purposes of this calculation all load or resistance factors are set equal to 1.0 in order more closely reflect the true demand and strength on the structure.

All major portions of the roof are evaluated, with the focus primarily on the areas that have parapets and the process room. Smaller areas, such as ventilation ducts or air intakes typically do not have parapets, allowing water to shed, and by inspection will have very low D/C ratios due to small span length.

The results for the slab evaluation are summarized in Table 6.2. Calculations for the D/C’s in mathcad follow the table.

Table 6.2 Roof Slab Demand to Capacity Ratio (Dead load only)

Area	Slab Elevation	Slab Thickness	D/C	Notes
Purification	+53 and +49	3'-0"	0.52	
Purification	+22	1'-8"	0.34	
Assembly	43	2'-6"	1.22	two-way slab action would reduce to below 1.0, but still very high
Stack	+55	4'-0"	0.06	low due to removal of stack
Process Area North	+34	3'-6"	0.46	
Process Area North	+22	1'-7"	0.46	
Process Area North	+60	1'-0"	0.11	
Process Area South	+34	3'-0"	0.36	stair tower additional
Process Area South	+15	2'-6"	0.60	two-way action and additional load from +34 offset
Process Area South	+60	1'-0"	0.17	
Process Area South	+48	2'-6"	0.20	
Process Area West	+75	5'-0"	0.38	EL +88 roof on process west also, but if it fails, water will drain to EL +75 due to curb at air intakes
Process Area West	+48	5'-0"	0.32	150 yrs after +75 fails
Process Room East	+68	5'-0"	0.26	
Process Room East	+88	5'-0"	0.35	
Process Room East	+48	5'-0"	0.39	150 yrs after +68 and +88 fail
Process Room East	+55	5'-0"	0.51	additional load if filter racks remain
Process Area	+149	1'-6"	0.33	load distributed to beams w/ very low D/C due to removal of forest
Process Area	+91	1'-6"	0.20	large beams and buttresses to distribute actuator tower loads to walls
Process Area	+66	5'-0"	0.15	150 yrs after +91 and/or +149
Process Area	+66 Deep deams	10'-0"	0.38	
Process Area	+48	5'-0"	0.37	

Find D/C ratio of roof slabs assuming simply supported conditions (neglect top reinforcing) and roof is full of wet soil to parapet level.

$f_c := 2500psi \quad f_y := 40ksi$

$cover := 2in \quad \gamma_c := 0.15\frac{kip}{ft^3} \quad \gamma_s := 0.12\frac{kip}{ft^3}$

Purification Area

3'-0" thick slab (EL +53 and +49) - main reinforcing is #11@9, span is approximately 40-ft (Ref W143264)

$A_s := \frac{1.56in^2}{9in} \quad t := 3ft \quad L := 40ft$

$d := t - cover - 0.75in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c} \quad a = 3.263\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) \quad M_n = 219.222\frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft \quad w_u = 0.57\,ksf$

$M_u := \frac{w_u \cdot L^2}{8} \quad M_u = 114\frac{kip \cdot ft}{ft} \quad \boxed{\frac{M_u}{M_n} = 0.52}$

1'-8" thick slab (EL +22) - main reinforcing is #10@12, span is approximately 22.5-ft (Ref W143264)

$A_s := \frac{1.27in^2}{ft} \quad t := 20in \quad L := 22.5ft$

$d := t - cover - 0.6in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c} \quad a = 1.992\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) \quad M_n = 69.443\frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft \quad w_u = 0.37\,ksf$

$M_u := \frac{w_u \cdot L^2}{8} \quad M_u = 23.414\frac{kip \cdot ft}{ft} \quad \boxed{\frac{M_u}{M_n} = 0.337}$

Assembly Area

2'-6" thick slab (EL +43) - main reinforcing is #10@8", span is approximately 57-ft
(Ref W157224)

$A_s := \frac{1.27in^2}{8in}$ $t := 2.5ft$ $L := 57ft$

$d := t - cover - 0.6in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $a = 2.988\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $M_n = 164.502\,\frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $w_u = 0.495\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $M_u = 201.032\,\frac{kip \cdot ft}{ft}$ $\boxed{\frac{M_u}{M_n} = 1.222}$

some two way slab action

Stack Area

4'-0" thick slab (EL +55) - main reinforcing is #11@6", span is approximately 17-ft
(Ref W131139)

$A_s := \frac{1.56in^2}{6in}$ $t := 4ft$ $L := 17ft$

$d := t - cover - 0.8in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $a = 4.894\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $M_n = 444.631\,\frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $w_u = 0.72\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $M_u = 26.01\,\frac{kip \cdot ft}{ft}$ $\boxed{\frac{M_u}{M_n} = 0.058}$

Very low D/C due to removal of stack

Process Area North

3'-6" thick slab (EL +34) - main reinforcing is #11@5", span is approximately 51-ft
(Ref W131816)

$A_s := \frac{1.56in^2}{5in}$ $t := 3.5ft$ $L := 51ft$

$d := t - cover - 0.75in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $a = 5.873\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $M_n = 453.193\,\frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $w_u = 0.645\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $M_u = 209.706\,\frac{kip \cdot ft}{ft}$ $\boxed{\frac{M_u}{M_n} = 0.463}$

1'-7" thick slab (EL +22) - main reinforcing is #10@10", span is approximately 28-ft
(Ref W131816)

$A_s := \frac{1.27in^2}{10in}$ $t := 19in$ $L := 28ft$

$d := t - cover - 0.75in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $a = 2.391\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $M_n = 76.478\,\frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $w_u = 0.357\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $M_u = 35.035\,\frac{kip \cdot ft}{ft}$ $\boxed{\frac{M_u}{M_n} = 0.458}$

1'-0" thick slab (EL +60) - main reinforcing is #7@8", span is approximately 9.33-ft
(Ref W131785)

$A_s := \dfrac{0.6in^2}{8in}$ $\quad t := 1ft$ $\quad L := 9.33ft$

$d := t - cover - 0.4in$

$a := \dfrac{A_s \cdot f_y}{0.85 \cdot f_c}$ $\quad a = 1.412\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \dfrac{a}{2}\right)$ $\quad M_n = 26.682\,\dfrac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $\quad w_u = 0.27\,ksf$

$M_u := \dfrac{w_u \cdot L^2}{8}$ $\quad M_u = 2.938\,\dfrac{kip \cdot ft}{ft}$ $\quad \boxed{\dfrac{M_u}{M_n} = 0.11}$

Process Area South

3'-0" thick slab (EL +34) - main reinforcing is #10@5", span is approximately 40-ft
(Ref W131816)

$A_s := \dfrac{1.27in^2}{5in}$ $\quad t := 3ft$ $\quad L := 40ft$

$d := t - cover - 0.6in$

$a := \dfrac{A_s \cdot f_y}{0.85 \cdot f_c}$ $\quad a = 4.781\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \dfrac{a}{2}\right)$ $\quad M_n = 315.056\,\dfrac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $\quad w_u = 0.57\,ksf$

$M_u := \dfrac{w_u \cdot L^2}{8}$ $\quad M_u = 114\,\dfrac{kip \cdot ft}{ft}$ $\quad \boxed{\dfrac{M_u}{M_n} = 0.362}$

but support of ducts and stair tower above will increase D/C

2'-6" thick slab, (EL +15) - main reinforcing is #11@6", span is approximately 50-ft (Ref W131627)

$A_s := \frac{1.56in^2}{6in}$ $t := 2.5ft$ $L := 50ft$

$d := t - cover - 0.6in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $a = 4.894\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $M_n = 259.511 \frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $w_u = 0.495\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $M_u = 154.687 \frac{kip \cdot ft}{ft}$ $\boxed{\frac{M_u}{M_n} = 0.596}$

More two way slab action with this slab, but also must support some load from EL +34 slab, EL 19 duct and wall above.

1'-0" thick slab (EL +60) - main reinforcing is #5@8", span is approximately 8.5-ft (Ref W131785)

$A_s := \frac{0.31in^2}{8in}$ $t := 1ft$ $L := 8.5ft$

$d := t - cover - 0.3in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $a = 0.729\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $M_n = 14.47 \frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $w_u = 0.27\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $M_u = 2.438 \frac{kip \cdot ft}{ft}$ $\boxed{\frac{M_u}{M_n} = 0.169}$

2'-6" thick slab (EL +48) - main reinforcing is #6@12", span is approximately 11.5-ft (Ref W131785)

$$A_s := \frac{0.44\,in^2}{12\,in} \qquad t := 2.5\,ft \qquad L := 11.5\,ft$$

$$d := t - cover - 0.3\,in$$

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c} \qquad a = 0.69\,in$$

$$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) \qquad M_n = 40.121\,\frac{kip \cdot ft}{ft}$$

$$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft \qquad w_u = 0.495\,ksf$$

$$M_u := \frac{w_u \cdot L^2}{8} \qquad M_u = 8.183\,\frac{kip \cdot ft}{ft} \qquad \boxed{\frac{M_u}{M_n} = 0.204}$$

Process Area West

5'-0" thick slab (EL +75) - main reinforcing is 2 layers #10@4.5", span is approximately 66-ft (Ref W131786)

$$A_s := \frac{2 \cdot 1.27\,in^2}{4.5\,in} \qquad t := 5\,ft \qquad L := 64\,ft$$

$$d := t - cover - 1.27\,in$$

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c} \qquad a = 10.625\,in$$

$$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) \qquad M_n = 1160.895\,\frac{kip \cdot ft}{ft}$$

$$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft \qquad w_u = 0.87\,ksf$$

$$M_u := \frac{w_u \cdot L^2}{8} \qquad M_u = 445.44\,\frac{kip \cdot ft}{ft} \qquad \boxed{\frac{M_u}{M_n} = 0.384}$$

If EL +88 roof fails, water will drain to EL +75 roof due to curb at air intakes

5'-0" thick slab (EL +48) - main reinforcing is 2 layers #11@4.5", span is approximately 66-ft
(Ref W131812)

$A_s := \frac{2 \cdot 1.56 in^2}{4.5 in}$ $\quad t := 5ft$ $\quad L := 64ft$

$d := t - cover - 1.27in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $\quad a = 13.051\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $\quad M_n = 1392.338 \frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $\quad w_u = 0.87\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $\quad M_u = 445.44 \frac{kip \cdot ft}{ft}$ $\quad \boxed{\frac{M_u}{M_n} = 0.32}$

fan and equipment loading is minor

Process Area East

2'-6" thick slab (EL +68) - main reinforcing is #6@12", span is approximately 13-ft
(Ref W131817)

$A_s := \frac{0.44 in^2}{12 in}$ $\quad t := 2.5ft$ $\quad L := 13ft$

$d := t - cover - 0.4in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $\quad a = 0.69\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $\quad M_n = 39.974 \frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $\quad w_u = 0.495\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $\quad M_u = 10.457 \frac{kip \cdot ft}{ft}$ $\quad \boxed{\frac{M_u}{M_n} = 0.262}$

2'-6" thick slab (EL +88) - main reinforcing is #10@12", span is approximately 25-ft (Ref W131817)

$$A_s := \frac{1.27in^2}{12in} \qquad t := 2.5ft \qquad L := 25ft$$

$$d := t - cover - 0.6in$$

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c} \qquad a = 1.992\,in$$

$$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) \qquad M_n = 111.777\,\frac{kip \cdot ft}{ft}$$

$$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft \qquad w_u = 0.495\,ksf$$

$$M_u := \frac{w_u \cdot L^2}{8} \qquad M_u = 38.672\,\frac{kip \cdot ft}{ft} \qquad \boxed{\frac{M_u}{M_n} = 0.346}$$

5'-0" thick slab (EL +48) - main reinforcing is 2 layers #10@5", span is approximately 62-ft
(Ref W131812)

$$A_s := \frac{2 \cdot 1.27in^2}{5in} \qquad t := 5ft \qquad L := 62ft$$

$$d := t - cover - 0.6in$$

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c} \qquad a = 9.562\,in$$

$$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) \qquad M_n = 1069.214\,\frac{kip \cdot ft}{ft}$$

$$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft \qquad w_u = 0.87\,ksf$$

$$M_u := \frac{w_u \cdot L^2}{8} \qquad M_u = 418.035\,\frac{kip \cdot ft}{ft} \qquad \boxed{\frac{M_u}{M_n} = 0.391}$$

additional load from exhaust fans and +88 structure

5'-0" thick slab (EL +55) - main reinforcing is 2 layers #10@5", span is approximately 70-ft
(Ref W131817)

$A_s := \frac{2 \cdot 1.27in^2}{5in}$ $t := 5ft$ $L := 70ft$

$d := t - cover - 1.27in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $a = 9.562\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $M_n = 1055.6\,\frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $w_u = 0.87\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $M_u = 532.875\,\frac{kip \cdot ft}{ft}$ $\boxed{\frac{M_u}{M_n} = 0.505}$

Additional load from filter racks and stack duct will increase D/C some

Process Area

1'-6" thick slab (EL +149 actuator tower) - main reinforcing is #5@8", span is approximately 13.5-ft
(Ref W132377)

$A_s := \frac{0.31in^2}{8in}$ $t := 1.5ft$ $L := 13.5ft$

$d := t - cover - 0.3in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $a = 0.729\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $M_n = 23.77\,\frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $w_u = 0.345\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $M_u = 7.86\,\frac{kip \cdot ft}{ft}$ $\boxed{\frac{M_u}{M_n} = 0.331}$

Load distributed to beams with very low D/C due to removal of forest system.

1'-6" thick slab (EL +91.25) - main reinforcing in slab is #7@10", span is approximately 8-ft
(Ref W131786)

$A_s := \frac{0.6in^2}{10in}$ $t := 1.5ft$ $L := 8ft$

$d := t - cover - 0.3in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $a = 1.129\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $M_n = 36.325 \frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $w_u = 0.345\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $M_u = 2.76 \frac{kip \cdot ft}{ft}$ $\boxed{\frac{M_u}{M_n} = 0.076}$

Large beams and buttresses to distrubute actuator tower loads

1'-6" thick slab (EL +91.25) - embedded 4'-0" wide beams in slab- (10) #9 bars, span is approximately 14-ft , trib. width is 11-ft
(Ref W131786)

$A_s := 10 \cdot 1.0in^2$ $t := 1.5ft$ $L := 14ft$ $b := 48in$ $t_w := 11ft$

$d := t - cover - 0.3in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b}$ $a = 3.922\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $M_n = 457.974\,kip \cdot ft$

$w_u := \left(\gamma_c \cdot t + \gamma_s \cdot 1ft\right) \cdot t_w$ $w_u = 3.795\,ft\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $M_u = 92.977\,kip \cdot ft$ $\boxed{\frac{M_u}{M_n} = 0.203}$

Large beams and buttresses to distrubute actuator tower loads

5'-0" thick slab (EL +66) - main reinforcing is #8@6", span is approximately 20-ft (Ref W132089)

$A_s := \frac{0.79in^2}{6in}$ $\quad t := 5ft$ $\quad L := 20ft$

$d := t - cover - 0.3in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $\quad a = 2.478\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $\quad M_n = 297.36\,\frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $\quad w_u = 0.87\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $\quad M_u = 43.5\,\frac{kip \cdot ft}{ft}$ $\quad \boxed{\frac{M_u}{M_n} = 0.146}$

Check deep beams supporting EL +66 slab:

Beams are 10-ft deep, 7-ft wide, w/ (60) #11 bars for positive rmomjent reinforcing

$t_b := 10ft$ $\quad b := 7ft$ $\quad t_s := 5ft$ $\quad L := 61ft$

$A_s := 60 \cdot 1.56in^2$ $\quad A_s = 93.6\,in^2$

$d_b := 9.5ft$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b}$ $\quad a = 20.975\,in$

$M_n := A_s \cdot f_y \cdot \left(d_b - \frac{a}{2}\right)$ $\quad M_n = 3.23 \times 10^4\,kip \cdot ft$

$w_u := \gamma_c \cdot t_b \cdot b + \gamma_c \cdot t \cdot (28ft - b)$ $\quad w_u = 26.25\,\frac{kip}{ft}$

$M_u := \frac{w_u \cdot L^2}{8}$ $\quad M_u = 1.221 \times 10^4\,kip \cdot ft$ $\quad \boxed{\frac{M_u}{M_n} = 0.378}$

5'-0" thick slab (EL +48) - main reinforcing is 2 layers #10@5", span is approximately 62-ft
(Ref W132373)

$A_s := \frac{2\,1.27in^2}{5in}$ $t := 5ft$ $L := 60ft$

$d := t - cover - 1.27in$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $a = 9.562\,in$

$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $M_n = 1055.6\,\frac{kip \cdot ft}{ft}$

$w_u := \gamma_c \cdot t + \gamma_s \cdot 1ft$ $w_u = 0.87\,ksf$

$M_u := \frac{w_u \cdot L^2}{8}$ $M_u = 391.5\,\frac{kip \cdot ft}{ft}$ $\boxed{\frac{M_u}{M_n} = 0.371}$

6.4.3 Seismic Load Effects

It is likely the 105-P structure could withstand a PC-3 level (~ 2500 yr return period) seismic event in its current state. This is based on the fact that K-reactor and L-reactor are qualified for PC-3 level seismic load. K- and L-reactors do have a different layout, but enough similarities exist with 105-P to allow for comparisons. From the K- and L seismic analyses [17] the vertical acceleration of a roof slab is at most about 0.5g. Since most roof slabs have D/C ratios less than 0.66 (under essentially dead load only), they would be able to withstand a 0.5g seismic load in the simply supported state. Only the Assembly area, far away from the Process room, would be affected.

Prior to significant degradation, the slabs can withstand the PC-3 seismic load. Once degradation of the top layer of steel has occurred, the slabs can still withstand the PC-3 load. Since the return period (2500 years) is much larger than the time it would take between significant water infiltration and collapse due to degradation to occur once vegetative growth or carbonation has occurred (several hundred years), the PC-3 level event will not have much of an effect on the overall collapse of the slabs. As events with a shorter return period have lower accelerations, they will also have little effect. Events with larger return periods will have higher accelerations, but given the long return period, are unlikely to occur before the slabs degrades due to other mechanisms.

The walls of the structure have lower D/C's than the roof slabs since they are controlled by compression under vertical load and have larger thicknesses to resist in-plane loads. The largest weakness for the walls will be the lack of out-of-plane support if an adjoining slab has failed. However, it has already been shown that seismic loads will not control the slab degradation. It is concluded that seismic loads in general will have little affect on the degradation of the structure. The structure can withstand probable seismic loads during the time it takes for other degradation mechanisms to cause at least local collapses. It is possible for larger seismic events to occur and cause damage prior to other degradation mechanisms, but not probable.

6.4.4 Expansion Joints

A primary weak spot for water infiltration into the structure will be the expansion joints. Expansion joint degradation has not been observed in 105-P, but has been observed in other reactor buildings. For example, in R-reactor the expansion joint between the Assembly and Purification areas allows for water build-up in the 2 inch gap between the buildings, producing a large amount of efflorescence, as shown in Figure 6.2. Left as-is, the expansion joints would likely degrade and allow water infiltration within 50-100 years.

The expansion joint locations in 105-P are shown in Figure 6.3. It is noted that all the expansion joint locations are away from the Process room and reactor vessel. Degradation of the expansion joints would allow for more water infiltration into the structure, but not in critical areas. Only localized degradation would be expected in the roof slab around the joints. The only potential exception is the joint between the Assembly and Purification areas where the wall could be degraded. This would still have no effect on the process room however. The expansion joint between the process and disassembly building will be removed with the above grade portion of the disassembly building.

In summary, expansion joint degradation would be expected in 50-100 years, but have little impact to the overall degradation of the structure, especially the Process room.

Figure 6.2 Efflorescence in R-reactor Assembly area [19].

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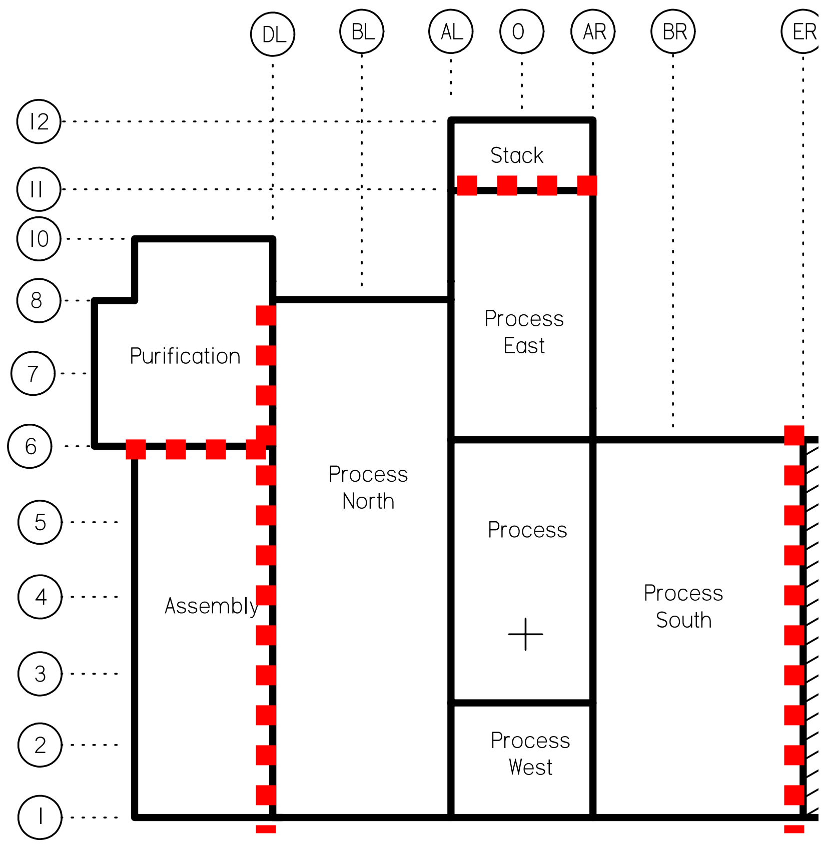

Figure 6.3 Expansion joint locations.

6.4.5 General Progression of Degradation and Collapse

From the D/C ratios in Table 6.2, it is observed that, with the exception of the assembly area roof slab, all D/C's are less than or equal to 0.6, with most between 0.3 and 0.5 for dead load only. This means there is a lot of reserve capacity in the slabs. This is somewhat expected due to the thickness of the slabs resulting from radiological requirements and a 1000 psf blast load design requirement.

The low D/C ratios are an important characteristic of the structure. Due to the low D/C's, in general, significant degradation (i.e. cracking and water infiltration) would take place before a collapse finally occurs. Also, due to the excess capacity, a catastrophic collapse of an entire roof slab or portion of the structure at once is unlikely to occur. Instead, localized areas are likely to collapse first, providing a preferential water infiltration path. Degradation would then tend to progress from these local collapse areas as reinforcing corrodes and further breaks up the concrete. The length of time between significant water infiltration and the point where most of the slab has collapsed is difficult to gauge. It is judged to be no more than a hundred years, which is insignificant over a time scale of thousands of years.

The roof slabs are expected to collapse first. The walls will last for a while longer, due to the lower D/C ratios and the fact they are controlled by compression. However, the walls would be expected to experience accelerated degradation after some slabs collapse due to increased exposure to the elements, exposure of reinforcing steel due to collapse, and potential trapped water in a bathtub effect. It is estimated the smaller walls will last no more than 150 years, while the thicker Process room walls could remain up to 300 years.

6.4.6 Actuator Tower Degradation

The collapse progression of the actuator tower is of particular interest due to its location directly over the reactor vessel. As noted in the previous section, a catastrophic collapse of the structure is not expected. This is particularly true with the actuator tower due to the buttresses. The buttresses provide a large shear area and stiffen the actuator tower considerably while providing additional support to the actuator tower walls if the roof slabs at El +91 and +149 collapsed. Large beams provide additional vertical support.

Debris from the actuator tower would fall on the EL +66 slab. This slab has one of the lowest D/C ratios, primarily due to the removal of the forest and hanging platforms in the process room below. The beams that help support this slab also have a low D/C. Therefore, collapse (occurring in pieces) of the actuator tower will not cause the EL +66 slab to collapse. It will, however, allow for exposure and water infiltration on the slab directly over the reactor unit. If the numerous penetrations in the EL +66 slab from the safety rods and cables are not sealed, then water will be able to fall directly onto the grout cap over the reactor unit.

If the penetrations are sealed, it will take longer for water to infiltrate into the Process room over the reactor vessel. Debris from the actuator tower collapse would likely cause the cover concrete to crack and/or chip. Also, depending upon available pathways, the area could act like a bathtub, ponding water. This would allow for faster degradation of the reinforcing steel. It is estimated significant water infiltration would occur within 100 years and the slab itself would begin to collapse after about 200 years.

6.4.7 Grout Cap Evaluation

Inside the Process room, the primary barrier for the reactor vessel will be a grout cap. The grout cap will be sloped to allow for any water that does infiltrate to drain off into the crane maintenance area. Integral crystalline waterproofing (ICW) will also be added to the grout in order to self-heal cracks. Presumably, as long as the grout is not chemically changed, the ICW additive will remain inert until it is activated by water at a crack location.

Two mechanisms could degrade the cap. Once water began to infiltrate the EL +66 slab directly over the reactor vessel, water falling from the underside of the slab could begin to erode the grout cap, especially if sand and other coarse material is suspended in the water. This mechanism has been observed personally by the author at home as water falling off the roof (approximately 15-ft drop) has eroded the paste away from the surface aggregate of concrete pavers in just a few years. If it is assumed a 1/16th of an inch erodes per year, than nearly 3-ft will be eroded after 500 years (500 x 0.0625 = 31.25 in). This would only be in localized areas and would be affected by the EL +66 slab collapsing a couple hundred years after significant water infiltration began anyway.

The EL +66 slab will not collapse catastrophically all at once, but as collapse does occur, pieces of debris will fall onto the grout cap, possibly causing spalling and chipping. The reinforcing pattern is #8@6” one way and #8@12” the other way. Based on this pattern, the largest block of concrete that could fall would be approximately 6” x 12” x 5’0”. The distance the concrete debris could penetrate into the grout cap can be estimated with the modified National Defense Research Committee (NDRC) formulae [2].

Penetration into grout cap (Modified NDRC formula) -

Velocity of concrete debris (assume 60-ft drop)

$h := 60ft$ $\qquad V := \sqrt{2 \cdot g \cdot h}$ $\qquad V = 62.136 \frac{ft}{sec}$

Weight of concrete debris: $W := 6in \cdot 1ft \cdot 5ft \cdot 0.15 \frac{kip}{ft^3}$ $\qquad W = 375 lbf$

Reference velocity: $U := 200 \frac{ft}{sec}$

Effective missile diameter: $D := 10in$

Shape factor: $N := 1.0$

Grout strength (assumed): $f_c := 2500psi$

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$$x_{pen} := in\sqrt{4\cdot\frac{180}{\sqrt{\frac{f_c}{psi}}}\cdot N\cdot\frac{W}{lbf}\cdot\frac{D}{in}\cdot\left(\frac{\frac{V}{\frac{ft}{sec}}}{1000\frac{D}{in}}\right)^{1.8}} \qquad x_{pen} = 2.4\,in$$

Thickness required to prevent perforation (Modified NDRC formula)

Missile mass: $M := \frac{W}{g}$ $\qquad M = 375\,lb$

$$x_{perf} := \left(\frac{U}{V}\right)^{0.25}\cdot\left(\frac{M\cdot V^2}{D\cdot f_c}\right)^{0.5} \qquad x_{perf} = 6.225\,in$$

Perforation thickness really doesn't apply since grout cap is fully support by slab below, bu
it gives an idea of the amount of damage that could be done.

Based on the calculations above, the required grout cap thickness due to erosion and falling debris is about three feet. This should be considered in addition to the amount required for radiological purposes.

6.5 Alternative Timelines

Using the discussion from the previous sections, timelines of the degradation of the structure can be assembled for the three different alternatives evaluated.

Alternative A - No intervention, so vegetative growth is allowed on all roofs

Time (yrs)	Event
0-50	Roof drains plug
50-100	Expansion joints fail. Water able to infiltrate structure, but not Process Room.
150	Assembly area roof begins to collapse.
200-250	Most roof slabs begin to collapse, including actuator tower roof.
200	Water infiltration into Process Room, due to EL +48 roof degradation/collapse.
225	Water infiltration into Process Room over reactor vessel through open penetrations in EL +66 slab (exposed due to actuator tower roof collapse)
400	Process East (EL +48 over crane maintenance area), EL +66 (over reactor vessel), and Process West (EL +48) slabs begin to collapse. Walls away from Process Room collapsing, cap now exposed.
600	Process Room walls are mostly collapsed.
750	Significant vegetative growth over cap
1000	Pile of rubble enveloped in vegetative growth remains. Grout block still intact with less than ½ inch removed due to sulfate and magnesium attack. Localized degradation in top of grout block where slabs and remains of wall have exposed reinforcing steel corroding.

The 105-P structure in this alternative can withstand forces from PC-3 earthquakes until the roofs begin to collapse (approximately 200 years).

Alternative B - Vegetative growth is prevented on roofs over the process room (approximate extent from Column lines AL-AR and 1-6).

Time (yrs)	Event
0-50	Roof drains (not over process room) plug.
50-100	Expansion joints fail and water is able to infiltrate structure, but not in Process Room.
150	Assembly area roof begins to collapse.
200-250	Most roof slabs begin to collapse, except over Process Room (Col. AL-AR and 1-6).
400	Process East slab at EL +48 (over crane maintenance area) begins to collapse and walls away from Process Room are collapsing
1000	Mostly rubble enveloped in vegetative growth around Process Room structure.
1200	Reinforcing steel in Process Room structure becomes depassivated due to carbonation.
1400-1500	Roofs over Process Room (Col. AL-AR and 1-6) begin to collapse.
1400	Water infiltration into Process Room, primarily due to EL +48 roof degradation/collapse.
1550	Water infiltration through EL +66 slab directly over reactor vessel.
1700	EL+66 (over reactor vessel) and Process West (EL +48) slabs begin to collapse. Cap is exposed.
1900	Process Room walls are mostly collapsed.
2050	Significant vegetative growth over cap.
2500	Pile of rubble enveloped in vegetative growth remains. Grout block still intact with approximately ¾ inch removed due to sulfate and magnesium attack. Localized degradation in top of grout block where slabs and remains of walls have exposed reinforcing steel corroding.

In this alternative, the 105-P structure (excluding Process Room structure) can withstand forces from PC-3 earthquakes until the roofs begin to collapse (approximately 200 years). The Process Room structure can withstand forces from PC-3 earthquakes for about 1200 years.

Alternative C - Vegetative growth is prevented on all roofs.

Time (yrs)	Event
50-100	Expansion joints fail. Water is able to infiltrate, but not in Process Room
1200	Reinforcing steel becomes depassivated due to carbonation.
1350	Assembly area roof begins to collapse.
1400-1500	Most roof slabs begin to collapse, including actuator tower roof.
1400	Water infiltration into Process Room, due to EL +48 roof,
1550	Water infiltration into PR over reactor vessel through EL +66 slab.
1700	Process East, (EL +48 over crane maintenance area), EL +66 (over reactor vessel), and Process West (EL +48) slabs begin to collapse, walls away from Process Room are collapsing, cap now exposed
1900	Process Room walls mostly collapsed,
2050	Significant vegetative growth over cap.
2500	Pile of rubble enveloped in vegetation remains. Grout block still intact with less than ¾ inch removed due to sulfate and magnesium attack. Localized degradation in top of grout block where floor slabs and remains of walls have exposed reinforcing steel corroding.

The 105-P structure in this alternative can withstand forces from PC-3 earthquakes for approximately 1200 years.