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10 CFR 50.59
10 CFR 50.90

RS-03-045

February 26, 2003

U. S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, DC 20555-0001

Dresden Nuclear Power Station, Units 2 and 3
Facility Operating License Nos. DPR-19 and DPR-25
NRC Docket Nos. 50-237 and 50-249

Subject: Request for License Amendment Related to Heavy Loads Handling

References:

(1) Letter from J. S. Abel (Commonwealth Edison Company) to U. S. NRC, "Dresden Station Units 2 and 3, Quad Cities Station Units 1 and 2, Dresden Special Report No. 41, Quad Cities Special Report No. 16, 'Reactor Building Crane and Cask Yoke Assembly Modifications,' AEC Dckt. 50-237, 50-249, 50-254 and 50-265," dated November 8, 1974

(2) Letter from J. S. Abel (Commonwealth Edison Company) to U. S. NRC, "Dresden Station Units 2 and 3, Quad Cities Station Units 1 and 2, Dresden Special Report No. 41, Supplement A, Quad Cities Special Report No. 16 – Supplement A, 'Reactor Building Crane and Cask Yoke Assembly Modifications,' NRC Dckts. 50-237, 50-249, 50-254 and 50-265," dated June 3, 1975

(3) Letter from K. R. Jury (Exelon Generation Company, LLC) to U. S. NRC, "Request for License Amendment Related to Heavy Loads Handling," dated September 26, 2002

In accordance with 10 CFR 50.90, "Application for amendment of license or construction permit," and 10 CFR 50.59, "Changes, tests, and experiments," Exelon Generation Company (EGC), LLC, is requesting a change to Facility Operating License Nos. DPR-19 and DPR-25, for Dresden Nuclear Power Station (DNPS), Units 2 and 3. The proposed change will allow DNPS to revise the Updated Final Safety Analysis Report (UFSAR) to include a description of a load drop analysis performed for handling reactor cavity shield blocks weighing greater than 110 tons with the Unit 2/3 reactor building crane during power operation.

Between 1974 and 1976, Commonwealth Edison (ComEd) Company, now EGC, extensively modified the DNPS reactor building crane with the intent of qualifying the crane as single failure-proof for its full rated capacity of 125 tons. In support of a Technical Specifications amendment

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request to support spent fuel cask handling, we provided information regarding these modifications in References 1 and 2. In this information we stated that the fuel casks used would weigh up to 100 tons with a 10 ton lifting rig.

In a teleconference with the NRC on September 20, 2002, the NRC stated that it considers the DNPS reactor building crane approved as meeting single failure-proof criteria only for loads of up to 110 tons. The top layer of reactor cavity shield blocks at DNPS, Units 2 and 3, consists of two pieces. For DNPS, Unit 3, each piece of the top layer of reactor cavity shield blocks weighs less than 116 tons. Based on a review of dimensional drawings, it is expected that each piece of the top layer of reactor cavity shield blocks for DNPS, Unit 2 will weigh more than 110 tons and less than or equal to 116 tons. The reactor cavity shield blocks may be moved prior to and during reactor disassembly, and can be stored on the refueling floor of the operating unit.

Since the reactor building crane is only approved as single failure-proof for loads of up to 110 tons, the proposed use of the crane to move the reactor cavity shield blocks weighing greater than 110 tons with a unit at power increases the possibility of a load drop which could damage safety-related equipment. This requires NRC approval in accordance with 10 CFR 50.59. However, as demonstrated in Attachment 1, the proposed change involves no significant hazards consideration.

In Reference 3, EGC submitted, and the NRC approved a one-time use of the reactor building crane to handle loads up to and including 116 tons in order to handle the Unit 3 reactor cavity shield blocks during refueling outage D3R17. EGC committed to submit an additional license amendment request to permanently resolve this situation. EGC is evaluating the reactor building crane to determine the feasibility of increasing its single failure-proof rating. This process will not be completed in time to allow lifting the reactor cavity shield blocks for refueling outage D2R18, which is scheduled to begin in early November 2003. EGC has completed a load drop analysis following the guidelines of NUREG-0612, "Control of Heavy Loads at Nuclear Power Plants," Appendix A for handling reactor cavity shield blocks and has determined that the handling of these shield blocks can be completed safely without increasing the single failure-proof rating of the reactor building crane.

We request NRC approval of the proposed change by October 24, 2003, to permit heavy load handling operations for refueling outage D2R18.

This request is subdivided as follows.

1. Attachment 1 gives a description and safety analysis of the proposed change.

2. Attachment 2 provides the proposed revisions to the UFSAR.

3. Attachment 3 provides a copy of the load drop analysis calculation to assist the NRC review.

The proposed change has been reviewed by the DNPS Plant Operations Review Committee and approved by the Nuclear Safety Review Board in accordance with the requirements of the EGC Quality Assurance Program.

EGC is notifying the State of Illinois of this request for a change to the operating license by transmitting a copy of this letter and its attachments to the designated State Official.

Should you have any questions concerning his letter, please contact Mr. Allan R. Haeger at (630) 657-2807.

Respectfully,

Patrick R Simpson

Patrick R. Simpson
Manager - Licensing
Mid-West Regional Operating Group

Attachments: Affidavit
Attachment 1: Description and Safety Analysis for Proposed change
Attachment 2: Proposed Revision to the UFSAR
Attachment 3: Calculation DRE02-0064, Rev. 0 and Rev. 0A, "D2/3 Load Drop Evaluation of the Reactor Shield Plugs"

cc: Regional Administrator – NRC Region III
NRC Senior Resident Inspector – Dresden Nuclear Power Station
Office of Nuclear Facility Safety – Illinois Department of Nuclear Safety

STATE OF ILLINOIS)

COUNTY OF DUPAGE)

IN THE MATTER OF)

EXELON GENERATION COMPANY, LLC) Docket Numbers

DRESDEN NUCLEAR POWER STATION, UNITS 2 AND 3) 50-237 and 50-249

SUBJECT: Request for License Amendment Related to Heavy Loads Handling

AFFIDAVIT

I affirm that the content of this transmittal is true and correct to the best of my knowledge, information, and belief.

Patrick R. Simpson

Patrick R. Simpson
Manager - Licensing
Mid-West Regional Operating Group

Subscribed and sworn to before me, a Notary Public in and

for the State above named, this 26th day of

February, 2003

"OFFICIAL SEAL"
TIMOTHY A. BYAM
NOTARY PUBLIC STATE OF ILLINOIS
COMMISSION EXPIRES 12/04/05

Timothy Byam
Notary Public

Attachment 1
Request for License Amendment Related to Heavy Loads Handling

Description and Safety Analysis for Proposed Change

1.0 Introduction

In accordance with 10 CFR 50.90, "Application for amendment of license or construction permit," and 10 CFR 50.59, "Changes, tests, and experiments," Exelon Generation Company (EGC), LLC, is requesting a change to Facility Operating License Nos. DPR-19 and DPR-25, for Dresden Nuclear Power Station (DNPS), Units 2 and 3. The proposed change will allow DNPS to revise the Updated Final Safety Analysis Report (UFSAR) to include a description of a load drop analysis performed for handling reactor cavity shield blocks weighing greater than 110 tons with the Unit 2/3 reactor building crane during power operation.

EGC requests NRC approval of the proposed change by October 24, 2003, to permit heavy load handling operations for a refueling outage which is scheduled to begin in early November 2003.

2.0 Proposed Change

UFSAR Section 9.1.4.3.2, "Reactor Building Overhead Crane," will be revised to add the following statement.

> A load drop analysis has been performed for handling the Units 2 and 3 reactor cavity shield blocks weighing greater than 110 tons for the designated safe load path to show that a postulated load drop will not affect any safety-related equipment, as there will be no scabbing or perforation of the concrete under the refueling floor, and the overall response of the floor system is acceptable. This load drop analysis was performed in accordance with the guidelines of NUREG-0612, Appendix A. The load drop analysis methodology was reviewed and approved by the NRC. The designated safe load path, hoisting height restrictions, and the weight of the load on which the analysis was based are described in station procedures. When handling shield plugs weighing greater than 110 tons, crane controls incorporate travel limits and hoisting height restrictions.

3.0 Background

Between 1974 and 1976, Commonwealth Edison (ComEd) Company, now EGC, extensively modified the DNPS reactor building crane with the intent of qualifying the crane as single failure-proof for its full rated capacity of 125 tons. In support of a Technical Specifications amendment request to support spent fuel cask handling, ComEd provided information regarding these modifications in References 1 and 2. In this information we stated that the fuel casks used would weigh up to 100 tons with a 10 ton lifting rig.

In a teleconference with the NRC on September 20, 2002, the NRC stated that it considers the DNPS reactor building crane approved as meeting single failure-proof criteria only for loads of up to 110 tons. The top layer of reactor cavity shield blocks at DNPS, Units 2 and 3, consists of two pieces. For DNPS, Unit 3, each piece of the top

layer of reactor cavity shield blocks weighs less than 116 tons. Based on a review of the dimensional drawings, each piece of the top layer of reactor cavity shield blocks for DNPS, Unit 2 is expected to weigh more than 110 tons and less than or equal to 116 tons. The reactor cavity shield blocks are moved prior to and during the refueling outage, and are stored on the refueling floor of the operating unit.

In Reference 3, EGC submitted, and the NRC approved, a one-time use of the reactor building crane to handle loads up to and including 116 tons in order to handle the Unit 3 reactor cavity shield blocks during refueling outage D3R17. EGC committed to submit an additional license amendment request to permanently resolve this situation.

EGC is evaluating the reactor building crane to determine the feasibility of increasing its single failure-proof rating. This process will not be completed in time to allow lifting of the reactor cavity shield blocks for D2R18, which is scheduled to begin in early November 2003. EGC has completed a load drop analysis for handling reactor cavity shield blocks weighing greater than 110 tons and up to 116 tons following the guidelines of NUREG-0612, Appendix A and has determined that the handling of these shield blocks can be completed safely without increasing the single failure-proof rating of the reactor building crane.

The proposed UFSAR change requires NRC approval in accordance with 10 CFR 50.59. The lifting of the reactor cavity shield blocks weighing greater than 110 tons at power and movement to the refueling floor of the operating unit increases the possibility of a load drop, which could damage safety-related equipment, since the crane is not single failure-proof for this load. However, as demonstrated in this amendment request, the proposed change does not create a credible possibility of a new accident.

4.0 Technical Analysis

The reactor building crane is designed to handle loads up to 125 tons and has been designated as single failure-proof for loads of $\leq$ 110 tons. Thus, the reactor building crane is capable of lifting reactor cavity shield blocks (the heaviest of which weighs approximately 116 tons) without significant probability of a load drop.

Safe load paths for the movement of the reactor cavity shield blocks have been designated to minimize the potential effect of a load drop while remaining within the practical limitations due to the size of the reactor cavity shield blocks and the space available on the refueling floor. These load paths are governed by the following considerations.

- General practices incorporated into DNPS procedures as a result of NUREG-0612 ensure that heavy load heights are maintained as low as practical and that the movement of heavy loads over the spent fuel pool and open reactor cavity is prohibited. The load path incorporates these considerations.

- The radius of the semi-circular top layer of reactor cavity shield blocks is approximately 21 feet 6 inches. The load path ensures that the reactor cavity shield

blocks remain over reactor building structural members supporting the refueling floor during movement.

- Since the reactor cavity shield blocks are handled only on the refueling floor, which contains no safety-related equipment, a drop of the reactor cavity shield blocks in the designated safe load paths would not directly impact any such equipment.

A load drop analysis has been completed for the designated safe load paths. The load drop analysis used 116 tons as the maximum weight of the pieces from the top layer of Unit 3 reactor cavity shield blocks. Based on a review of dimensional drawings, it is expected that each piece of the top layer of the Unit 2 reactor cavity shield blocks will weigh less than or equal to 116 tons. The weight of each piece of the top layer of the Unit 2 reactor cavity shield blocks will be verified to be within the assumptions of the analysis. If necessary, the load drop analysis will be adjusted for variations in weight above 116 tons, using the methodology described in the current calculation.

The load drop analysis shows that a postulated drop of the reactor cavity shield blocks from the heights assumed in the analysis will not affect the capability of safety-related equipment located on the floors below the refueling floor to perform its function, as there will be no scabbing or perforation of the concrete under the refuel floor, and the overall response of the floor system is acceptable. This load drop analysis is contained in Attachment 3 and was performed in accordance with the applicable assumptions described in NUREG-0612, Appendix A, as follows.

- The load is dropped in an orientation that causes the most severe consequences.
- The load is dropped at any location in the designated safe load path.
- The analysis postulates the maximum damage that could result, i.e., the analysis considered that all energy is absorbed by the structure that is impacted.

Conformance to all of the guidelines of NUREG-0612, Appendix A is further discussed in the attached calculation.

In addition, the following controls, which are not discussed in the attached calculation, will be implemented in accordance with NUREG-0612, Appendix A.

- Mechanical stops, electrical interlocks, or similar automatic controls will restrict travel outside the designated safe load path.
- Mechanical stops, electrical interlocks, or similar automatic controls will be provided to prohibit lifting the reactor cavity shield blocks above the height assumed in the analysis.
- These controls will be designed to allow activation of the travel and lifting restrictions when handling the reactor cavity shield blocks, unless a particular piece is shown to weigh less than 110 tons.

Further, existing procedural controls will be modified to include the following to ensure that the load drop analysis assumptions are preserved.

- The applicable procedures will describe the weight of the shield blocks assumed in the analysis, the safe load path, and the hoisting height restrictions for the reactor cavity shield blocks.
- The applicable procedures will ensure that the mechanical stops, interlocks, or automatic controls are not bypassed during handling of the reactor cavity shield blocks, unless a particular piece is shown to weigh less than 110 tons.

In summary, the load drop analysis and controls ensure that a postulated drop of the reactor cavity shield blocks weighing greater than 110 tons will have no effect on spent fuel, fuel in the reactor vessel, or safety-related equipment.

5.0 Regulatory Analysis

5.1 No Significant Hazards Consideration

In accordance with 10 CFR 50.90, "Application for amendment of license or construction permit," Exelon Generation Company (EGC), LLC, is requesting a change to Facility Operating License Nos. DPR-19 and DPR-25, for Dresden Nuclear Power Station (DNPS), Units 2 and 3. Specifically, the proposed change will allow EGC to revise the DNPS Updated Final Safety Analysis Report (UFSAR) to include a description of a load drop analysis performed for handling the reactor cavity shield blocks weighing greater than 110 tons with the reactor building crane during power operation.

The proposed changes do not involve a significant increase in the probability or consequences of an accident previously evaluated.

The proposed change will allow use of a load drop analysis performed for handling the reactor cavity shield blocks weighing greater than 110 tons with the reactor building crane during power operation. The load drop analysis demonstrates that dropping a reactor cavity shield block within the designated safe load path from the heights assumed in the analysis will not affect the capability of safety-related equipment to perform its function. Therefore, the proposed change does not involve a significant increase in the probability or consequences of an accident previously evaluated.

The proposed changes do not create the possibility of a new or different kind of accident from any accident previously evaluated.

The proposed change will allow use of a load drop analysis performed for handling the reactor cavity shield blocks weighing greater than 110 tons with the reactor building crane during power operation. The load drop analysis demonstrates that dropping a reactor cavity shield block within the designated safe load path from the heights assumed in the analysis will not affect the capability of safety-related equipment to perform its function. Therefore, the proposed change will not create the possibility of a new or different kind of accident from any accident previously evaluated.

The proposed changes do not involve a significant reduction in a margin of safety.

The proposed change will allow use of a load drop analysis performed for handling the reactor cavity shield blocks weighing greater than 110 tons with the reactor building crane during power operation. The load drop analysis demonstrates that dropping a reactor cavity shield block within the designated safe load path from the heights assumed in the analysis will not affect the capability of safety-related equipment to perform its function. Therefore, it is concluded that the proposed change does not result in a significant reduction in the margin of safety.

Conclusion

Based upon the above evaluation, EGC has concluded that the criteria of 10 CFR 50.92(c), "Issuance of amendment," are satisfied and that the proposed UFSAR change involves no significant hazards consideration.

5.2 Applicable Regulatory Requirements and Criteria

In NUREG-0612, the NRC provided regulatory guidelines in two phases (Phase I and II) to assure safe handling of heavy loads in areas where a load drop could impact stored spent fuel, fuel in the reactor core, or equipment that may be required to achieve safe shutdown or permit continued decay heat removal. Phase I guidelines address measures for reducing the likelihood of dropping heavy loads and provide criteria for establishing safe load paths, procedures for load handling operations, training of crane operators, design, testing, inspection, and maintenance of cranes and lifting devices, and analyses of the impact of heavy load drops. Phase II guidelines address alternatives for mitigating the consequences of heavy load drops, including using either (1) a single failure-proof crane for increased handling system reliability, or (2) electrical interlocks and mechanical stops for restricting crane travel, or (3) load drops and consequence analyses for assessing the impact of dropped loads on plant safety and operations. NUREG-0612, Appendix A provides guidance regarding load drop analyses.

The Phase II guidelines apply specifically to the proposed change discussed in this amendment request. As discussed above the proposed change meets the Phase II guidelines.

Generic Letter (GL) 85-11, "Completion of Phase II of Control of Heavy Loads at Nuclear Power Plants, NUREG-0612," dated June 28, 1985, dismissed the need for licensees to implement the guidelines of NUREG-0612 Phase II based on the improvements obtained from the implementation of NUREG-0612 Phase I. GL 85-11, however, encouraged licensees to implement actions they perceived to be appropriate to provide adequate safety.

In NRC Bulletin 96-02, the NRC staff addressed specific instances of heavy load handling concerns and stated that licensees were responsible to ensure that heavy load handling activities with the reactor in operation did not constitute an unreviewed safety question by creating the possibility of an accident not previously evaluated or by increasing the probability or consequences of an accident previously evaluated.

As discussed above, the proposed change is being submitted for review in accordance with 10 CFR 50.59, because the proposed change increases the possibility of a load drop. However, as noted in Section 5.1, the proposed change does not create the credible possibility of a new accident.

6.0 Environmental Assessment

EGC has evaluated the proposed change against the criteria for identification of licensing and regulatory actions requiring environmental assessment in accordance with 10 CFR 51.21, "Criteria for and identification of licensing and regulatory actions requiring environmental assessments." EGC has determined that the proposed change meets the criteria for a categorical exclusion set forth in 10 CFR 51.22, "Criterion for categorical exclusion; identification of licensing and regulatory actions eligible for categorical exclusion or otherwise not requiring environmental review," paragraph (c)(9), and as such, has determined that no irreversible consequences exist in accordance with 10 CFR 50.92, "Issuance of amendment," paragraph (b). This determination is based on the fact that this change is being proposed as an amendment to a license issued pursuant to 10 CFR 50, "Domestic Licensing of Production and Utilization Facilities," which changes a requirement with respect to installation or use of a facility component located within the restricted area, and the amendment meets the following specific criteria:

(i) The proposed changes involve no significant hazards consideration.

As demonstrated in Section 5.1, the proposed change does not involve a significant hazards consideration.

(ii) There is no significant change in the types or significant increase in the amounts of any effluent that may be released offsite.

The proposed change will allow use of load drop analysis for handling the reactor cavity shield blocks weighing greater than 110 tons with the Unit 2/3 reactor building crane during power operation. The load drop analysis demonstrates that dropping a reactor cavity shield block within the designated safe load path from the heights assumed in the analysis will not affect the capability of safety-related equipment to perform its function. There will be no significant increase in the amounts of any effluents released offsite. The proposed change does not result in an increase in power level, does not increase the production, nor alter the flow path or method of disposal of radioactive waste or byproducts. Therefore, the proposed change will not affect the types or increase the amounts of any effluents released offsite.

(iii) There is no significant increase in individual or cumulative occupational radiation exposure.

The proposed change will not result in changes in the configuration of the facility. There will be no change in the level of controls or methodology used for processing

of radioactive effluents or handling of solid radioactive waste, nor will the proposal result in any change in the normal radiation levels within the plant. Therefore, there will be no increase in individual or cumulative occupational radiation exposure resulting from this change.

7.0 References

1. Letter from J. S. Abel (Commonwealth Edison Company) to U. S. NRC, "Dresden Station Units 2 and 3, Quad Cities Station Units 1 and 2, Dresden Special Report No. 41, Quad Cities Special Report No. 16, 'Reactor Building Crane and Cask Yoke Assembly Modifications,' AEC Dckt. 50-237, 50-249, 50-254 and 50-265," dated November 8, 1974

2. Letter from J. S. Abel (Commonwealth Edison Company) to U. S. NRC, "Dresden Station Units 2 and 3, Quad Cities Station Units 1 and 2, Dresden Special Report No. 41, Supplement A, Quad Cities Special Report No. 16 – Supplement A, 'Reactor Building Crane and Cask Yoke Assembly Modifications,' NRC Dckts. 50-237, 50-249, 50-254 and 50-265," dated June 3, 1975

3. Letter from K. R. Jury (Exelon Generation Company, LLC) to U. S. NRC, "Request for License Amendment Related to Heavy Loads Handling," dated September 26, 2002

Attachment 2
Request for License Amendment Related to Heavy Loads Handling

Proposed Revisions to the UFSAR

9.1.4.3.2 Reactor Building Overhead Crane

for loads less than or equal to 110 tons.

The 125-ton capacity reactor building overhead crane main hoist is single failure proof. Within the dual load path, the design criteria are such that all dual elements comply with the CMAA Specification No. 70 for allowable stresses, except for the hoisting rope which is governed by more stringent job specification criteria. With several approved exceptions, single element components within the load path (i.e. the crane hoisting system) have been designed to a minimum safety factor of 7.5, based on the ultimate strength of the material. Components critical to crane operation, other than the hoisting system, have been designed to a minimum safety factor of 4.5, based on the ultimate strength of the material. Table 9.1-3 lists the results of the crane component failure analysis.

The reactor building overhead crane and spent fuel cask yoke assemblies meet the intent of NUREG-0554 for loads less than or equal to 110 tons.

All analyses for handling spent fuel casks, performed relative to the overhead crane handling system loads have been based on the National Lead (NL) 10/24 spent fuel shipping cask which weighs 100 tons (Figure 9.1-18) and the HI-TRAC 100 transfer cask which weighs less than 100 tons (Section 9.1.2.2.4). If larger casks are used, additional analyses will be required to assure safety margins are maintained.

Administrative controls and installed limit switches restrict the path of travel of the crane to a specific controlled area when moving the spent fuel cask. The controls are intended to assure that a controlled path is followed in moving a cask between the decontamination and hatchway area and the spent fuel pool. Administrative controls also ensure movement of other heavy loads such as the drywell head, reactor vessel head, and dryer separator assembly is over preapproved pathways.

Technical Specifications state refueling requirements. Station procedures prohibit movement of heavy loads over the spent fuel pools or open reactor cavity except under Special Procedures

The crane reeving system does not meet the recommended criteria of Branch Technical Position APCSB 9-1 (now incorporated into NUREG-0554) for wire rope safety factors and fleet angles. The purpose of these criteria is to assure a design which minimizes wire rope stress wear and thereby provides maximum assurance of crane safety under all operating and maintenance conditions. Because the crane reeving system does not meet these recommended criteria, there is a possibility of an accelerated rate of wire rope wear occurring. Accordingly, to compensate in these design areas, a specific program of wire rope inspection and replacement is in place.

The inspection and replacement program assures that the entire length of the wire rope will be maintained as close as practical to original design safety factors at all times. This inspection and replacement program provides an equivalent level of protection to the methods suggested in wire rope safety and crane fleet angle criteria and will assure that accelerated wire rope wear will be detected before crane use.

"Two blocking" is an inadvertently continued hoist which brings the load and head block assemblies into physical contact, thereby preventing further movement of the load block and creating shock loads to the rope and reeving system. A mechanically operated power limit switch in the main hoist motor power circuit on the load side of all hoist motor power circuit controls provides adequate protection.

against "two blocking" in the event of a fused contactor in the main hoist control circuitry. This power limit switch will interrupt power to the main hoist motor and cause the holding brakes to set prior to "two blocking."

The reactor building refueling floor has been designed for a live load of 1000 lb/ft². The entire reactor building refueling floor (with the exception of the fuel pool and open reactor cavity) is considered a safe load path zone.

A 9-ton load drop has been analyzed. The results show that the refueling floor can survive a drop from 7 feet without scabbing damage. Procedures limit the 9-ton lift height to a maximum of 7 feet. Existing procedural controls limit both the height of a lift to clear obstacles and require the use of the most direct path to laydown areas.

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The reactor building overhead crane meets the single-failure criteria stated in NUREG-0612 for loads less than or equal to 110 tons. As required by CMAA-70, the maximum crane load weight plus the weight of the bottom block, divided by the number of parts of rope does not exceed 20% of the manufacturer's published breaking strength.

The reactor building overhead crane main hook has:

A rated load capacity	=	250,000 lb
Block and rope weight	=	20,500 lb
Total weight lifted	=	270,500 lb

This weight is supported by 12 parts of wire rope with a published breaking strength of 175,800 pounds.

$$\frac{\text{Total weight lifted/Number of parts of rope}}{\text{Breaking strength of rope}} = \frac{270,500}{12 \times 175,800} = 12.8\% \qquad (1)$$

As can be seen by Equation 1, this is less than the 20% CMAA-70 requirement.

A detailed analysis of the possibility of horizontal displacement of the cask in the event one of the redundant rope trains fails has been conducted. It has been confirmed that the horizontal load displacement will not exceed 2 1/2 inches throughout the critical elevations of lift. At the high point of the lift, with the cask above the operating floor, the static displacement of the load is approximately 1/2 inch with a total static plus dynamic displacement of approximately 1 inch. The total horizontal displacement of the load when the cask is submerged in the spent fuel pool is approximately 2 1/2 inches. A larger total horizontal displacement, approximately 9 inches, can occur with the load at its lowest elevation, that is with the load at the grade elevations. However, it should be noted that the NL 10/24 100-ton cask and the HI-TRAC 100 cask, which are the heaviest loads to be lifted through the equipment hatchway, are 7.33 feet in diameter and 7.83 feet across the cask yoke and approximately 8.25 feet in diameter and 8.5 feet across the cask yoke respectively. The equipment hatchway has a minimum 20.08 foot square opening (See Figure 9.1-20). Local protrusions of ductwork along the vertical path of the cask through the hatchway reduce the cross section to approximately 19.5 feet. Since the path of the cask is controlled by limit switches which restrict the position of the cask during lifting to ±6 inches from the center line of the hatchway, lateral clearances in excess of 4 feet are available.

Insert A

A load drop analysis has been performed for handling the Units 2 and 3 reactor cavity shield blocks weighing greater than 110 tons for the designated safe load path to show that a postulated load drop will not affect any safety-related equipment, as there will be no scabbing or perforation of the concrete under the refueling floor, and the overall response of the floor system is acceptable. This load drop analysis was performed in accordance with the guidelines of NUREG-0612, Appendix A. The load drop analysis methodology was reviewed and approved by the NRC. The designated safe load path, hoisting height restrictions, and the weight of the load on which the analysis was based are described in station procedures. When handling shield plugs weighing greater than 110 tons, crane controls incorporate travel limits and hoisting height restrictions.

Attachment 3
Request for License Amendment Related to Heavy Loads Handling

Calculation DRE02-0064, Rev. 0 and Rev. 0A,
"D2/3 Load Drop Evaluation of the Reactor Shield Plugs"

ATTACHMENT 1
Design Analysis Approval

DESIGN ANALYSIS NO.: DRE02-0064 **PAGE NO.** 1
Major REV Number: 0 **Minor Rev Number:**

[ ] BRAIDWOOD STATION
[ ] BYRON STATION
[ ] CLINTON STATION
[X] DRESDEN STATION
[ ] LASALLE CO. STATION
[ ] QUAD CITIES STATION
Unit: [X] 0 [ ] 1 [ ] 2 [ ] 3

DESCRIPTION CODE:(C018) S03
DISCIPLINE CODE: (C011) S
SYSTEM CODE: (C011) 00

TITLE: "D2/3 Load Drop Evaluation of the Reactor Shield Plugs

[X] Safety Related [ ] Augmented Quality [ ] Non-Safety Related

ATTRIBUTES (C016)

TYPE	VALUE	TYPE	VALUE
PROJ	11331-015		

COMPONENT EPN: (C014 Panel)

EPN	TYPE

DOCUMENT NUMBERS: (C012 Panel) (Design Analyses References)

Type/Sub	Document Number	Input (Y/N)
CALC / ENG	DRE98-0020	Y

REMARKS:

ATTACHMENT 1
Design Analysis Approval

DESIGN ANALYSIS NO. **DRE02-0064** **REV: 0** **PAGE 2**

Revision Summary: Initial Issue

Added Pages: 1 - 99, A1 - A9, B1 - B4, C1 - C3

Electronic Calculation Data Files: (Program Name, Version, File Name extension/size/date/hour/min)
slabCap.xls, 9/27/02, 24 kB, 5:56PM;
dre020064.mcd, 9/30/02, 466 kB, 12:58PM;
dre020064.doc, 9/30/02, 166 kB, 1:32PM

Design impact review completed? [ ] Yes [X] N/A, Per EC#:____________
(If yes, attach impact review sheet)

Prepared by: Kurt Koesser / Kurt Koesser / 9/30/02
Print / Sign / Date

Reviewed by: Adam Al-Dabbagh / Adam Al-Dabbagh / 9/30/02
Mohammad Amin / M. Amin / 9/30/02
Print / Sign / Date

Method of Review: [X] Detailed [ ] Alternate [ ] Test

This Design Analysis supersedes: N/A **in its entirety.**

Supplemental Review Required? [ ] Yes [ ] No

[ ] Additional Review [ ] Special Review Team

Additional Reviewer or Special Review Team Leader: ________ / ________ / ________
Print / Sign / Date

Special Review Team: (N/A for Additional Review)

Reviewers: 1) ________ / ________ / ________ 2) ________ / ________ / ________
Print / Sign / Date Print / Sign / Date
3) ________ / ________ / ________ 4) ________ / ________ / ________
Print / Sign / Date Print / Sign / Date

Supplemental Review Results:

Approved by: C. N. Petropoulos / C. N. Petropoulos / 9/30/02
Print / Sign / Date

A COPY MADE

External Design Analysis Review (Attachment 3 Attached)

Reviewed by: KISHIN CHHABLANI / Kishin Chhablani / 10-2-2-02
Print / Sign / Date

Approved by: T. Loch / T. Loch / 10-2-02
Print / Sign / Date

Do any ASSUMPTIONS / ENGINEERING JUDGEMENTS require later verification? [ ] Yes [X] No
Tracked By: AT#, EC# etc.)________________

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PURPOSE / OBJECTIVE

Purpose / Objective

The purpose of this calculation is to determine the maximum lifting height for the heavy load movement of each of the three layers of the Unit 3 Reactor Shield Plugs. The maximum lifting height will be evaluated for the entire travel path, from the initial location over the Unit 3 Reactor Cavity to the specified Laydown Area for each layer of shield plugs. The lifting height is controlled by the ability of the concrete structure below to survive a postulated shield plug drop from the maximum lifting height.

The objective of the calculation is to determine a safe load path for the removal of the Reactor Shield Plugs. The removal of the shield plugs is one of the first steps in the refueling process.

Background

The Reactor Building crane is rated at 125 tons. However, based on certain documentation, the NRC has indicated to the Station that the crane is presently rated as Single Failure Proof (SFP) for up to 110 tons.

There are three (3) layers of Reactor shield plugs. There are two (2) shield plugs in each layer and each layer is 2 feet thick. Each shield plug has the shape of a semi-circular disc. The diameter of the top layer shield plugs is approximately 43 feet, with the successive layers of plugs having smaller diameters. The shield plugs that form the top layer are the heaviest shield plugs. Exelon has determined (via an actual weighing process) that the top shield plugs and their lifting apparatus weigh slightly less than 116 Tons (232 kips).

This calculation is performed to assess the existing concrete structure for postulated load drops resulting from the movement of the shield plugs.

METHODOLOGY AND ACCEPTANCE CRITERIA

Methodology

NUREG-0612, Sections 5.1.4 (2) and 5.1.5 (1) (c) (Reference 2), requires that a load drop analysis should conform to the guidelines of Appendix A of the NUREG, as applicable, if the crane is not Single Failure Proof (for the specific load movement). NUREG-0612 Appendix A guidelines will be followed as applicable to the present load drop analysis.

Reference 1 provides general guidelines and formulations for the evaluation of impactive and impulsive loads. Reference 4 provides ductility requirements for reinforced concrete structures. Reference 6 provides structural design criteria for Dresden Station.

The methodology used in this calculation to evaluate the postulated load drops is described below.

1. Due to the anticipated small lifting height and large contact surface of impact (the plug is a half circle disc with a diameter close to 43 feet), and the corresponding low impact velocity, perforation of the floor will not occur. For an asymmetrical load drop (side drop of plug when only one or two of the three lift points of each plug fail), the potential for scabbing is possible. Therefore, scabbing will be investigated. This calculation will be based on the local damage equations given in Reference 1.

2. The overall adequacy of the impacted structural elements (beams, slabs, columns, and walls) will be determined by calculating the total strain energy in the impacted elements corresponding to an allowable ductility limit, and comparing this energy to the impact energy imparted to the impacted elements.

3. The yield resistance of the elements resisting the impact in flexure will be determined using an acceptable approach. The approach described in Reference 1, modified as described in this section of the calculation, will be used in this calculation.

4. The moment of inertia of the section will be determined using Reference 1, Section 3.1.8 and Figure 3.1.10. The energy absorption of the impacted elements will be calculated using constructed elasto-plastic load-deflection diagrams for elements. The ductility limit will be determined using Reference 4, Appendix C, Section C.3, and the area under the load-deflection diagram up to the applicable ductility limit will be used as the measure of energy absorption capacity of elements.

5. The shear failure load is estimated using ACI 318-99 (Reference 5). The shear failure load shall be at least 1.20 times the flexural resistance load in order to use the flexural mode of failure to calculate the strain energy. Otherwise, the ductility ratios given in Reference 4, Sections C.3.7 or C.3.9 shall be used. This requirement is stated in Reference 4, Appendix C, Section C.3.6.

6. Failure at the sling attachment or lug failure will result in a tilted drop of the shield plug. This drop usually results in half the impact energy of the full drop (when a failure at the crane hook occurs) due to the smaller travel distance of the shield plug center-of-gravity. In this calculation, most load drop scenarios are evaluated for full drop, unless noted.

7. The impact energy to be absorbed by the overall deflection of the impacted structural elements is less than the total kinetic energy of load drop. Some kinetic energy is dissipated during impact. This loss, which can be computed by equating the momentum of the entire system before and after impact, is most conveniently taken into account by multiplying the available kinetic energy by a factor. The value of this factor is dependent on the mass of the falling object and the effective mass of the impacted structural elements. The value of this factor is calculated by using equations from Section 15-4 of Reference 3.

8. This calculation will use the actual in-place concrete compressive strength, as specified in Reference 6.

Acceptance Criteria

After local damage effects are ruled out, the acceptable drop height is based on the ability of the impacted structural elements to absorb the remaining kinetic energy after loss due to impact is taken into account. This energy absorption limit is determined using elastic - perfectly plastic load-deflection diagram for the affected elements up to the allowable ductility limit applicable to these elements.

The ductility limits are determined from Table 5.1 of Reference 1 and Appendix C of Reference 4.

Computer Software Used in the Calculation

1. Microsoft Word
   Microsoft Word 97 SR-1
   Product ID: 53491-419-5449024-21064

2. Microsoft Excel
   Microsoft Excel 97 SR-1
   Product ID: 53491-419-5449024-21064

3. MathCad
   MathSoft Mathcad 2001 Professional
   S&L Program No. 03.7.548.10.2/O

The computer software listed above was used to prepare these calculations. These programs, accessed on the S&L LAN, have been validated per S&L Software Verification and Validation procedures for the program functions used in the calculation.

This calculation was prepared using the following S&L PCs:

PC No. 8334

ASSUMPTIONS / ENGINEERING JUDGMENTS

Assumptions

1. Hard missile impact is assumed. Energy lost in the deformation of the dropped plug itself is ignored, which is conservative.

2. For yield deflection calculation, the moment of inertia of the reinforced concrete structural elements is the average of the cracked and uncracked moments of inertia, in accordance with Reference 1.

3. The dead load of the impacted structural element is considered in determining the strain energy of the element.

4. The shield plug weight, including the lifting apparatus, is considered to be 116 Tons (232 kips). This is conservative.

No unverified assumptions are used.

Additional minor assumptions are made and justified in the body of this calculation.

Engineering Judgments

Minor engineering judgments are made and justified in the body of this calculation.

DESIGN INPUTS

1. The in-place concrete compressive strength is taken from Reference 6.

2. Beam, slab, and column sizes and reinforcement are obtained from References 11, 12, 13, 18, 19 and 20.

3. The shield plug size and reinforcement are obtained from References 14 and 21.

4. The shield plug weight of 116 Tons (232 kips) is based on actual weighing of the top layer of shield plugs by Dresden Station on September 20, 2002.

5. The rebar strength is obtained from Reference 6.

6. The movement and laydown areas of the six (6) shield plugs are specified by Dresden Station. Figures 1 through 3 in Attachment B are constructed based on this information. The lifting heights specified in the figures are the result of this calculation.

7. Per Attachment C of this calculation, the requirements described in Item 3 of Section 1 of Appendix A of NUREG-0612 will be satisfied by Dresden Station through administrative control of the plug movements in the evaluated areas shown in Attachment B, Figures 1 through 3.

REFERENCES

Ref. No.	Document No. / Title	Rev. No. or Date	Remarks
1	Second ASCE Conference on "Civil Engineering and Nuclear Power, Volume V: Report of the ASCE Committee on Impactive and Impulsive Loads", September 1980, Knoxville, Tennessee	September 1980	
2	NUREG-0612 "Control of Heavy Loads at Power Plants"	July 1980	
3	"Roark's Formulas for Stress and Strain", 6th Edition, by W. C. Young	6th Edition	
4	ACI 349-97 "Code Requirements for Nuclear Safety Related Concrete Structures"	1997	
5	ACI 318-99 "Building Code Requirements for Structural Concrete"	1999	
6	TDBD-DQ-01 "Topical Design Basis Document - Quad Cities Units 1 & 2 and Dresden Units 2 & 3 - Structural Design Criteria"	1	
7	S&L Evaluation No. SL-007347 "Evaluation of a Postulated Drop of the Top Two Reactor Shield Plugs (Cookies) During the Weighing Operation"	0	Attachment A
8	Drawing B-200	AF	
9	Drawing B-208	S	
10	Drawing B-209	N	
11	Drawing B-235	E	
12	Drawing B-236	H	
13	Drawing B-237	H	
14	Drawing B-242	C	
15	Drawing B-630	AA	
16	Drawing B-638	H	

Ref. No.	Document No. / Title	Rev. No. or Date	Remarks
17	Drawing B-639	D	
18	Drawing B-665	C	
19	Drawing B-666	D	
20	Drawing B-667	B	
21	Drawing B-672	000 4/1/1968	
22	S&L Form 1715 "Standard Specification for Concrete Work"	Q 8/2/1965	
23	Calculation DRE98-0020 "Evaluation of Reactor Building Superstructure"	2	
24	Drawing B-206	Q	
25	"Yield Line Formulae for Slabs" by K. W. Johansen	1972	
26	Sargent & Lundy Engineering Evaluation, Control No. D-1298M, "Evaluation of Drywell Shield Plug Drop While the Units Are Operating"	10/26/98	
27	Drawing B-257	F	
28	Drawing B-687	A	

CALCULATIONS

Introduction

This calculation will determine the maximum lifting height for the heavy load movement of each of the three layers of the Unit 3 Reactor Shield Plugs. The maximum lifting height will be evaluated for the entire travel path, from the initial location over the Unit 3 Reactor Cavity to the specified Laydown Area for each layer of shield plugs. The lifting height is controlled by the ability of the concrete structure below to survive a postulated shield plug drop from the maximum lifting height.

The objective of the calculation is to determine a safe load path for the removal of the Reactor Shield Plugs. The removal of the shield plugs is one of the first steps in the refueling process.

Travel Path of Reactor Shield Plugs

The travel path of the three layers of shield plugs must be evaluated because the weight of the top and middle layers exceeds the 110 Ton SFP rating for the Reactor Building Crane.

The travel path of the shield plugs is limited by the allowable reach of the Reactor Building Crane hook. The allowable reach of the crane hook for the top, middle, and bottom layers of shield plugs was determined in Calculation DRE98-0020 (Reference 23).

For the top and middle layer shield plugs, the maximum lifted load is 116 Tons (232 kips), and the maximum crane hook reach is 27'-3" north of Column Row N (equivalent to 1'-6" north of Column Row M).

For the bottom layer shield plugs, the maximum lifted load is 108 Tons (216 kips), and the maximum crane hook reach is 15'-9" north of Column Row N.

The travel path of the Unit 3 shield plugs are briefly described below.

Unit 3 Reactor Shield Plugs

The six Unit 3 concrete Reactor Shield plugs are situated at the intersection of Column Lines K and 47, over the Reactor cavity.

The four shield plugs from the top two layers (top layer and middle layer) will be moved south along Column Line 47. The shield plugs are then moved east between Column Lines L and M to Column Line 41. The shield plugs will then be moved north along Column Line 41 to the intersection of Column Lines K and 41, over the Unit 2 shield plugs. The two Unit 3 top layer shield plugs will be placed on top of the Unit 2 top layer shield plugs. The two Unit 3 middle layer shield plugs will then be placed on top of the Unit 3 top layer shield plugs.

The two bottom layer Unit 3 shield plugs will be moved south along Column Line 47. The shield plugs are then moved east between Column Lines L and M to Column Line 41. The shield plugs will then be moved south and east to the area between Column Lines 39 and 40. At this time, the shield plug is orientated in the north-south direction. The shield plug will be laid down between the two column lines within the specified area of Figure 3 of Attachment B. The Unit 3 shield plugs will be stacked on top of each other.

Refer to figures in Attachment B for plan of load paths for these shield plugs.

A detailed step-by-step description of the travel path is provided on the following pages. The purpose of this description is not to delineate exact movements to be followed. The description is provided as a guide to area descriptions in the figures of Attachment B that form the basis for evaluation of elements to define the height limits that are provided in Attachment B.

Detailed Movement Sequence on the Travel Path

The orientation of the in-place Reactor Shield Plugs are shown on Drawing B-242 (Ref. 14) and Drawing B-672 (Ref. 21). The diameter ("chord") of the in-place shield plugs are positioned as follows:

Top Layer Shield Plugs: Chord along the north-south axis.
Middle Layer Shield Plugs: Chord along the east-west axis.
Bottom Layer Shield Plugs: Chord along the north-south axis.

The detailed movement sequence of the Unit 3 shield plugs from the in-place position to the Laydown area is described below. Each of the two semi-circular shield plugs in each layer is moved separately.

Later in this calculation, the underlying concrete structures at each point along the load path will be evaluated.

Unit 3 Top Layer Shield Plugs

1. Lift the shield plug to a maximum height of 1'-0" above the top of the floor at Elevation 613'-0" (shield plug chord orientated in north-south direction) from the in-place location (with center-of-gravity along Column Row K) and move the center-of-gravity of the shield plug east or west to Column Row 47.

2. Move the shield plug south (shield plug chord orientated in north-south direction) along Column Row 47 up to the limit of the crane hook (about 1'-6" north of Column Row M).

3. With center-of-gravity of shield plug along Column Row 47, rotate the shield plug 45 degrees (until chord is orientated in northwest-southeast or northeast-southwest direction).

4. With center-of-gravity of shield plug along Column Row 47, continue to rotate the shield plug another 45 degrees until chord is orientated in east-west direction. Position the shield plug between Column Lines L and M as practically possible.

5. Move the shield plug east (shield plug chord orientated in east-west direction), north of and parallel to Column Row M, to the midway point between Column Rows 47 and 46.

6. Continue to move the shield plug east to Column Row 44.

7. Continue to move the shield plug east to Column Row 43.

8. Continue to move the shield plug east to Column Row 42.

9. Continue to move the shield plug east to Column Row 41.

10. With center-of-gravity of shield plug along Column Row 41, rotate the shield plug until chord is orientated in north-south direction (if required).

11. Move the shield plug north (shield plug chord orientated in north-south direction) along Column Row 41 until the center-of-gravity of the shield plug is over Column Row K.

12. Move the center-of-gravity of the shield plug in the desired direction and lower the shield plug on top of the Unit 2 top layer shield plugs.

Unit 3 Middle Layer Shield Plugs

13. Lift the shield plug to a maximum height of 1'-0" above the top of the floor at Elevation 613'-0" (shield plug chord orientated in east-west direction) from the in-place location (with center-of-gravity along Column Row 47) (apex of shield plug faces either north or south).

14. Move the shield plug south (shield plug chord orientated in east-west direction) along Column Row 47 up to the limit of the crane hook (about 1'-6" north of Column Row M). Position the shield plug between Column Lines L and M as practically possible.

15. Move the shield plug east (shield plug chord orientated in east-west direction), north of and parallel to Column Row M, to the midway point between Column Rows 47 and 46.

16. Continue to move the shield plug east, north of and parallel to Column Row M, to Column Row 44.

17. Continue to move the shield plug east, north of and parallel to Column Row M, to Column Row 43.

18. Continue to move the shield plug east, north of and parallel to Column Row M, to Column Row 42.

19. Continue to move the shield plug east, north of and parallel to Column Row M, to Column Row 41.

20. Lift the shield plug from current height of 1'-0" above the floor to a height of 2'-6" above the floor.

21. With center-of-gravity of shield plug along Column Row 41, move the shield plug north (shield plug chord orientated in east-west direction) along Column Row 41 until the center-of-gravity of the shield plug is over Column Row K.

22. Move the shield plug in the desired direction and lower the shield plug on top of the previously placed Unit 3 top layer shield plug.

Unit 3 Bottom Layer Shield Plugs

23. Lift the shield plug to a maximum height of 1'-0" above the top of the floor at Elevation 613'-0" (shield plug chord orientated in north-south direction) from the in-place location (with center-of-gravity along Column Row K) and move the center-of-gravity of the shield plug east or west to Column Row 47.

24. Move the shield plug south (shield plug chord orientated in north-south direction) along Column Row 47 up to the limit of the crane hook (about 1'-6" north of Column Row M).

25. With center-of-gravity of shield plug along Column Row 47, rotate the shield plug until chord is orientated in east-west direction. Position the shield plug between Column Lines L and M as practically possible.

26. Move the shield plug east (shield plug chord orientated in east-west direction), north of and parallel to Column Row M, to the midway point between Column Rows 47 and 46.

27. Continue to move the shield plug east, north of and parallel to Column Row M, to Column Row 44.

28. Continue to move the shield plug east, north of and parallel to Column Row M, to Column Row 43.

29. Continue to move the shield plug east, north of and parallel to Column Row M, to Column Row 42.

30. Continue to move the shield plug east, north of and parallel to Column Row M, to Column Row 41.

31. For the second Unit 3 bottom layer shield plug at this location, lift the shield plug from current height of 1'-0" above the floor to a height of 2'-6" above the floor.

32. Move the shield plug southeast (shield plug chord orientated in east-west direction) to the area between Column Lines 39 and 40. At this time, the shield plugs will be rotated to be orientated in the north-south direction. The shield plug will be laid down between the two column lines within the specified area of Figure 3 of Attachment B.

33. For the first Unit 3 bottom layer shield plug that is moved, set the shield plug on the floor at Elevation 613'-0". For the second Unit 3 bottom layer shield plug that is moved, lower the shield plug on top of the previously placed Unit 3 bottom layer shield plug.

Structural Elements Affected by Travel Path

The structural elements affected by the travel path are discussed below.

Columns

The concrete columns affected by the travel path are listed below (see B-208 [Ref. 9] and B-638 [Ref. 16]), along with the column sizes (see B-235 [Ref. 11] and B-665 [Ref. 18]).

M-49 (33" x 33")
M-48 (36" x 36")
M-47 (33" x 33")
M-46 (36" x 36")
M-45 (36" x 36")
M-43 (36" x 36")
M-42 (36" x 36")
M-41 (33" x 33")
M-40 (36" x 36")
M-39 (33" x 33")

Columns M-49, M-47, M-41, and M-39 are smaller than the other columns and therefore will control. Column M-47 will be evaluated in this calculation as representative of the critical case.

Beams

The concrete beams affected by the travel path are listed below (see B-208 [Ref. 9] and B-638 [Ref. 16]), along with the beam sizes (see B-236 [Ref. 12] and B-666 [Ref. 19]).

North-South Beams:

5B8 (33" x 54") (Column Rows 39 & 49 / L-M)
5B7 (27" x 54") (Column Rows 39 & 49 / M-N)
5B23 (36" x 54") (Column Rows 40 & 48 / L-M)
5B22 (36" x 54") (Column Rows 40 & 48 / M-N)
5B6 (27" x 54") (Column Rows 41 & 47 / L-M)
5B5 (27" x 54") (Column Rows 41 & 47 / M-N)
5B4 (36" x 69") (Column Rows 42 & 46 / L-M)
5B3 (36" x 69") (Column Rows 42 & 46 / M-N)
5B2 (33" x 66") (Column Rows 43 & 45 / L-M)

East-West Beams:

5B21 (30" x 54") (Column Row M / 49-48)
5B10 (24" x 48") (Column Row M / 39-48)

Beams 5B7, 5B6, and 5B5 control for north-south beams and Beam 5B10 controls for east-west beams. Beams 5B6 and 5B10 will be evaluated in this calculation. The reason for this selection is detailed in the section of this calculation dealing with "Selection of Critical Structural Elements".

Slabs

The concrete slabs affected by the travel path are listed below (see B-208 [Ref. 9] and B-638 [Ref. 16]), along with the slab thickness and designations (see B-237 [Ref. 13] and B-667 [Ref. 20]).

Slab "R" (18") (Column Rows 39 - 41 / L-N)
Slab "T" (18") (Column Rows 41 - 42 / L-M)
Slab "R" (18") (Column Rows 41 - 42 / M-N)
Slab "S" (18") (Column Rows 42 - 43 / L-M)
Slab "V" (24") (Column Rows 43 - 44 / L-M)
Slab "R" (18") (Column Rows 44 - 45 / L-M)
Slab "R" (18") (Column Rows 45 - 46 / L-M)
Slab "R" (18") (Column Rows 46 - 48 / L-N)
Slab "B1" (24") (Column Rows 48 - 49 / L-M)
Slab "R" (18") (Column Rows 48 - 49 / L-M)

Removable slab panels between Column Rows 42 - 43 & 45 - 46 / M-N) are not in place and will not be evaluated.

Slab "R" controls. Slab "R" will be evaluated in this calculation. For additional discussion, see the section of this calculation titled "Selection of Critical Structural Elements".

Walls

The concrete walls affected by the travel path are listed below (see B-208 [Ref. 9] and B-638 [Ref. 16]).

Column Row L Wall (Reactor Wall) (60" thick)
Column Row 44 Wall (24" thick)

The Column Row 44 Wall controls. The Column Row 44 Wall will be evaluated in this calculation.

Material Properties and Constants

The actual and in-place concrete compressive strength for the Unit 2 and Unit 3 Reactor Building are listed below.

This calculation will utilize the in-place concrete compressive strength. The minimum in-place concrete strength for Units 2 and 3 will be used in the calculation.

$f_{c\_nominal} := 4000 \cdot psi$ (Ref. 6, 8, 15) (Nominal strength)

$f_{c\_actual\_u2} := 4700 \cdot psi$ (Ref. 6) (In-Place Strength - Unit 2)

$f_{c\_actual\_u3} := 5000 \cdot psi$ (Ref. 6) (In-Place Strength - Unit 3)

$f_c := \min(f_{c\_actual\_u2}, f_{c\_actual\_u3})$

$f_c = 4700\, psi$ (Controlling strength)

The reinforcing bar yield strength for the Unit 2 and Unit 3 Reactor Building is listed below.

$f_y := 60000 \cdot psi$ (Ref. 6, 8, 15)

This calculation will determine the effect of a postulated load drop using the methodology given in Reference 1. Reference 1 provides Dynamic Increase Factors (DIF) for concrete and steel structures under various loadings. The DIFs for concrete and steel are tabulated below. The DIF's tabulated below are from Reference 1, Table 5.4.

$DIF_c := 1.25$ (DIF for concrete compression)

$DIF_s := 1.10$ (DIF for tension and compression in concrete reinforcing steel with fy = 60 ksi)

This calculation will (conservatively) not include DIFs, unless the use of DIFs is absolutely necessary to determine if a successful load path and lifting height are achievable.

$DIF_c := 1.00$

$DIF_s := 1.00$

Selection of Critical Structural Elements

Introduction

The description of the travel path of the Reactor Shield Plugs and a list of the structural elements affected by the travel path were given in previous sections of this calculation.

This section of the calculation will determine the critical structural elements that must be evaluated for the potential load drops. The selection of the critical structural elements is based on the size of the structural elements and the required orientation of the shield plugs at points along the load path.

The critical cases that must to be evaluated will be enveloped into several potential load drop scenarios. These scenarios will be evaluated in detail in this calculation.

Selection of Critical Structural Elements for Detailed Analysis

1. Beams within the Load Path

Unit 2 beams are Marks 5B2, 5B4, 5B6, 5B5, 5B22, 5B23 & 5B10 - Drawings B-208 (Ref. 9), B-209 (Ref. 10) & B-236 (Ref. 12).

Unit 3 beams are Marks 5B6, 5B4, 5B5 & 5B10 - Drawing B-638 (Ref. 16), B-639 (Ref. 17) & B-666 (Ref. 19).

Review of above drawings shows that the size and reinforcement of beams with similar mark number are identical for the two units. The review also indicates that Beam 5B6 is the weakest of the north/south beams within the travel path (smallest size of beam, smallest size of stirrup with larger spacing while the rebars are comparable). Therefore, Beam 5B6 is selected as a typical north/south beam.

In the east/west direction, all the beams within the travel path are 5B10, therefore, Beam 5B10 is selected as the typical east/west beam.

2. Slabs within the Load Path

Unit 2 slabs are Marks R, S, T & V - Drawings B-208 (Ref. 9), B-209 (Ref. 10) & B-237 (Ref. 13).

Unit 3 slab is Mark R - Drawing B-638 (Ref. 16), B-639 (Ref. 17) & B-667 (Ref. 20).

Review of above drawings shows that the reinforcement of slabs with similar mark number are identical for the two units. The flexural capacities of these slabs are calculated on the Excel spreadsheet shown on the next page.

Per Ref. 1, the flexural resistance of a slab for a concentrated force (R) is expressed as:

$$R = (2)(\pi)(Mu_{pos} + Mu_{neg})$$

where

Mu_{pos} is the average of positive moment capacities at midspan in both directions.

Mu_{neg} is the average of negative moment capacities at all supports in both directions.

2. Slabs within the Load Path (continued)

Moment Capacities of Slab at Elev. 613'-0" (Ref Dwgs B-208, B-209, B-237)

Parameters		
fc	4700	psi
fy	60000	psi
phi	0.9	
b	12	in

Slab No	t (in)	Ave d (in)	North / South Direction							
			Reinforcement		Reinf Area (sq in/ft)		Pos Moment		Neg Moment	
			Bottom	Top	Bottom	Top	a	Cap (k ft)	a	Cap (k ft)
R	18	16	#8 at 12	#8 at 6	0.79	1.58	0.99	55.12	1.98	106.73
S	18	16	#8 at 18	#8 at 15	0.527	0.632	0.66	37.16	0.79	44.38
T	18	16	#8 at 12	#8 at 6	0.79	1.58	0.99	55.12	1.98	106.73
V	24	22	#8 at 12	#8 at 12	0.79	0.79	0.99	76.45	0.99	76.45
V	24	22	#8 at 12	#8 at 9	0.79	1.053	0.99	76.45	1.32	101.12

Slab No	t (in)	Ave d (in)	East / West Direction							
			Reinforcement		Reinf Area (sq in/ft)		Pos Moment		Neg Moment	
			Bottom	Top	Bottom	Top	a	Cap (k ft)	a	Cap (k ft)
R	18	16	#8 at 9	#8 at 6	1.053	1.58	1.32	72.69	1.98	106.73
S	18	16	#8 at 8	#8 at 5	1.185	1.896	1.48	81.37	2.37	126.39
T	18	16	#8 at 9	#8 at 6	1.053	1.58	1.32	72.69	1.98	106.73
T	18	16	#8 at 9	#8 at 5	1.053	1.896	1.32	72.69	2.37	126.39
V	24	22	#8 at 6	#8 at 6	1.58	1.58	1.98	149.39	1.98	149.39
V	24	22	#8 at 6	#8 at 5	1.58	1.896	1.98	149.39	2.37	177.58

See Excel spreadsheet on previous page for the flexural capacities.

Slab R:

$$R_R := 2\cdot\pi\left[\left[\frac{(55.12+72.69)\cdot kip\cdot ft}{2}\right]+\frac{(106.73\cdot 4)\cdot kip\cdot ft}{4}\right]$$

$$R_R = 1.072\times 10^3\, ft\, kip$$

Slab S:

$$R_S := 2\cdot\pi\left[\left[\frac{(37.16+81.37)\cdot kip\cdot ft}{2}\right]+\frac{(44.38\cdot 2+126.39\cdot 2)\cdot kip\cdot ft}{4}\right]$$

$$R_S = 908.863\, ft\, kip$$

Slab T:

$$R_T := 2\cdot\pi\left[\left[\frac{(55.12+72.69)\cdot kip\cdot ft}{2}\right]+\frac{(106.73\cdot 2+106.73+126.39)\cdot kip\cdot ft}{4}\right]$$

$$R_T = 1.103\times 10^3\, ft\, kip$$

Slab V:

$$R_V := 2\cdot\pi\left[\left[\frac{(76.45+149.39)\cdot kip\cdot ft}{2}\right]+\frac{(76.45+101.12+149.3+177.58)\cdot kip\cdot ft}{4}\right]$$

$$R_V = 1.502\times 10^3\, ft\, kip$$

Along the travel path, the load drop will engage 2 adjacent slabs as shown:

V & S $\quad R_V + R_S = 2.411\times 10^3\, ft\, kip$

S & T $\quad R_S + R_T = 2.012\times 10^3\, ft\, kip$

T & R $\quad R_T + R_R = 2.175\times 10^3\, ft\, kip$

R & R $\quad R_R + R_R = 2.144\times 10^3\, ft\, kip$

The resistance of Slabs S & T is approximately 6.5% less than the resistance of Slabs R & R and T & R. The beam located between Slabs R & R is 5B6 (27" x 54") while the beam located between Slabs S & T is 5B2 (36" x 69"). Beam 5B2 will have larger resistance compared to Beam 5B6 due to the larger beam size and higher beam reinforcement. Therefore, as a result of the review, the total resistance of Slab S & T and Beam 5B2 is larger than the total resistance of Slab R & R and Beam 5B6 by engineering judgment.

On the above basis, the controlling structural elements in the path towards Unit 2 when the plug orientation is in the east-west direction, consist of two adjacent Mark R slabs and the north/south beam 5B6 located between the two slabs.

Selection of Critical Scenarios for Detailed Analysis

The evaluation of the following load drop scenarios envelopes all potential load drops of the Reactor Shield Plugs on to the Reactor Cavity and on to the floor at Elevation 613'-0". The load movements are limited to the areas shown in Figures 1 through 3 (Attachment B).

The description of structural elements in the load path and the size of the 43' diameter semi-circular discs being moved have guided the selection of Scenarios 2 through 5 below. Scenario 1 is needed to address a potential drop on the Reactor cavities.

The controlling load drop scenarios are described below:

1. All cases of the drop of a shield plug on to the Reactor cavity (Units 2 and 3).

2. Full drop of a shield plug on a single column (Drop Height = 1'-0").

3. Full drop of a shield plug on a system of two adjacent slabs with a beam in between the slabs (Drop Height = 2'-6").

4. Full drop of a shield plug on two adjacent columns (Drop Height = 2'-6").

5. Full drop of a shield plug on wall at Column Row 44 (Drop Height = 2'-6").

Notes:

a. Scenario 2 covers the case of a full drop of 1 foot on a wall

b. Scenario 4 covers the case of a full drop of 2'-6" on a wall and a column.

Scenario 1

Scenario 1

All cases of the drop of a shield plug on to the Reactor cavity (Units 2 and 3).

Shield Plug Drop During Initial Lift from Unit 3 Cavity and During Laydown on Top of Unit 2 Shield Plugs

Capacities of Plugs

The plug is reinforced per Section 13-13 on Ref. 21:

#9 at 12" oc (top and bottom) in the short direction
#11 at 4.5" oc to # 11 at 6" oc (bottom) in the long direction
#11 at 12" oc to # 9 at 12" oc (top) in the long direction

Determine shear capacity of the plugs (See Section 11 on Ref. 5)

$R_{plug_1} := 21.5\cdot ft - 2\cdot in - 0.5\cdot in - 0.5(4\cdot in)$ top layer plugs

$R_{plug_2} := R_{plug_1} - 2\cdot in - 0.5\cdot in - 0.5(4\cdot in)$ middle layer plugs

$R_{plug_3} := R_{plug_2} - 2\cdot in - 0.5\cdot in - 0.5(4.5\cdot in)$ bottom layer plugs

$$R_{plug} = \begin{pmatrix} 21.125 \\ 20.75 \\ 20.354 \end{pmatrix} ft \qquad t_{plug} := 24\cdot in$$

$$d_{ave} := t_{plug} - 1.5\cdot in - \left[\frac{1.41\cdot in}{2} + \left[\frac{1\cdot in^2 \cdot \left(\frac{1.128\cdot in + 1.41\cdot in}{2} \right)}{1\cdot in^2 + \left(1.56\cdot in^2\right)\cdot \frac{12\cdot in}{5.5\cdot in}} \right] \right] \qquad d_{ave} = 21.507\,in$$

$\phi_v := 0.85$ $\qquad f_c := 4700\cdot psi$

Shear plane at "t_{plug}" dimension out

$$\phi V_c := \phi_v \cdot \left(2 \cdot \sqrt{f_c \cdot psi}\right) \cdot d_{ave} \cdot \left[\pi \cdot \left(R_{plug} - d_{ave}\right)\right] \qquad \phi V_c = \begin{pmatrix} 1.827 \times 10^3 \\ 1.791 \times 10^3 \\ 1.754 \times 10^3 \end{pmatrix} kip$$

$$UL_{total\_v} := \overrightarrow{\left[\phi V_c \cdot \frac{\left[\frac{\pi \cdot \left(R_{plug}\right)^2}{2}\right]}{\left[\frac{\pi \cdot \left(R_{plug} - d_{ave}\right)^2}{2}\right]} \right]} \qquad UL_{total\_v} = \begin{pmatrix} 2.181 \times 10^3 \\ 2.146 \times 10^3 \\ 2.109 \times 10^3 \end{pmatrix} kip$$

Determine Bearing Capacity of Top Shield Plug - Bearing Bars are 1" x 24" (12" apart)

$\phi_{br} := 0.7$ ....strength reduction factor for concrete bearing

$$\phi P_{br} := \phi_{br} \cdot \left[0.85 \cdot f_c \cdot \left[1 \cdot in \cdot \left(\pi \cdot R_{plug} \cdot \frac{2}{3}\right)\right]\right] \cdot 2$$

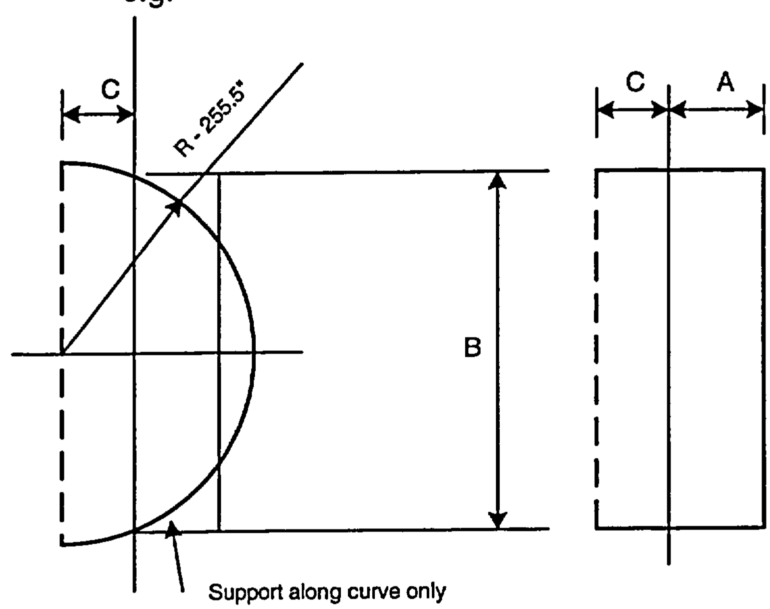
.... 2/3 factor is to account for the spaces between the bearing bars

$$\phi P_{br} = \begin{pmatrix} 2.969 \times 10^3 \\ 2.917 \times 10^3 \\ 2.861 \times 10^3 \end{pmatrix} kip$$

....bearing strength of concrete of the top plug (half circle)

$$UL_{total\_br} := \phi P_{br} \qquad UL_{total\_br} = \begin{pmatrix} 2.969 \times 10^3 \\ 2.917 \times 10^3 \\ 2.861 \times 10^3 \end{pmatrix} kip$$

Determine flexural capacity of plugs



Actual Configuration

Idealized Configuration

$$C := \frac{4 \cdot R_{plug}}{3 \cdot \pi}$$ $$C = \begin{pmatrix} 8.966 \\ 8.807 \\ 8.639 \end{pmatrix} ft$$

$$A := 1.1 \cdot C$$ $$A = \begin{pmatrix} 9.862 \\ 9.687 \\ 9.502 \end{pmatrix} ft$$ ....considered

$$A + C = \begin{pmatrix} 18.828 \\ 18.494 \\ 18.141 \end{pmatrix} ft$$ $$A + C = \begin{pmatrix} 18.828 \\ 18.494 \\ 18.141 \end{pmatrix} ft$$

$$B := 2\sqrt{R_{plug}^{2} - C^{2}}$$ $$B = \begin{pmatrix} 38.256 \\ 37.577 \\ 36.86 \end{pmatrix} ft$$

$$\overrightarrow{\frac{A+C}{B}} = \begin{pmatrix} 0.492 \\ 0.492 \\ 0.492 \end{pmatrix}$$

Calculate cracked moment of inertia using average of #11 at 5.5" oc in the long direction and #9 at 12" oc in the short direction

$$As := \frac{\left(1.56 \cdot in^2\right) \cdot \frac{12 \cdot in}{5.5 \cdot in} + 1 \cdot in^2}{2} \qquad As = 2.202\,in^2$$

$$b := 12 \cdot in$$

$$d := d_{ave} \qquad d = 21.507\,in$$

$$p := \frac{As}{b \cdot d} \qquad p = 8.531 \times 10^{-3}$$

$$E_c := 57000 \cdot \sqrt{f_c \cdot psi} \qquad E_c = 3.908 \times 10^3\,ksi$$

$$n := \frac{29000 \cdot ksi}{E_c} \qquad n = 7.421$$

$$p \cdot n = 0.063$$

Per Ref. 1, Figure 3.1.10 $F := 0.043$

$$I_{ct} := F \cdot b \cdot d^3 \qquad I_{ct} = 5.133 \times 10^3\,in^4$$

$$I_{gross} := \frac{b \cdot t_{plug}{}^3}{12} \qquad I_{gross} = 1.382 \times 10^4\,in^4$$

$$I_{ave} := \frac{I_{ct} + I_{gross}}{2}$$

$$t_{homo} := \left(\frac{12 \cdot I_{ave}}{b}\right)^{\frac{1}{3}} \qquad t_{homo} = 21.163\,in$$

Per Ref. 3, Table 26, Case 2, for a/b = 0.49

$t_{eq} := t_{homo}$ $\quad q_{unit} := 1\,ksf$

$$\sigma := \frac{0.36 \cdot q_{unit} \cdot B^2}{t_{eq}^{\;2}} \qquad \sigma = \begin{pmatrix} 1.176 \\ 1.135 \\ 1.092 \end{pmatrix} ksi$$

$$y_{max} := \frac{-0.08 \cdot q_{unit} \cdot B^4}{E_c \cdot t_{eq}^{\;3}} \qquad y_{max} = \begin{pmatrix} -0.666 \\ -0.62 \\ -0.574 \end{pmatrix} in$$

$$M_{unit} := \sigma \cdot \frac{b\, t_{eq}^{\;2}}{6} \qquad M_{unit} = \begin{pmatrix} 87.812 \\ 84.722 \\ 81.52 \end{pmatrix} kip \cdot ft$$

Based on Ultimate Strength Design

$\phi_M := 0.9$ $\quad fy_{rebar} := 60 \cdot ksi$

$$a := \frac{As \cdot fy_{rebar}}{0.85 \cdot f_c \cdot b} \qquad a = 0.23\,ft$$

$$\phi Mc := \phi_M \cdot As \cdot fy_{rebar} \cdot \left(d - \frac{a}{2}\right) \qquad \phi Mc = 199.441\,ft\,kip$$

Per "Yield-Line Formulae for Slabs" (Ref. 25)

$$qu := \frac{6 \cdot \phi Mc}{b\left(R_{plug}\right)^2} \qquad qu = \begin{pmatrix} 2.681 \\ 2.779 \\ 2.888 \end{pmatrix} ksf$$

$$UL_{total\_f} := \overrightarrow{\left[qu \cdot \frac{\pi \cdot \left(R_{plug}\right)^2}{2}\right]} \qquad UL_{total\_f} = \begin{pmatrix} 1.88 \times 10^3 \\ 1.88 \times 10^3 \\ 1.88 \times 10^3 \end{pmatrix} kip$$

$$\begin{pmatrix} \min(UL_{total\_v}) \\ \min(UL_{total\_br}) \\ \min(UL_{total\_f}) \end{pmatrix} = \begin{pmatrix} 2.109 \times 10^3 \\ 2.861 \times 10^3 \\ 1.88 \times 10^3 \end{pmatrix} kip$$

$$UL_{total} := \begin{pmatrix} \min(UL_{total\_v}) \\ \min(UL_{total\_br}) \\ \min(UL_{total\_f}) \end{pmatrix} \quad \begin{matrix} \text{Shear} \\ \text{Bearing} \\ \text{Flexure} \end{matrix}$$

$UL_{min} := \min(UL_{total})$ $\quad UL_{min} = 1.88 \times 10^3 kip$ (Flexure controls)

$R := UL_{min}$ $\quad R = 1.88 \times 10^3 kip$

From above results, the flexural mode of failure will govern. Note that the shear resistance of the plug is considerably higher than the flexural resistance.

Review of Shield Plug Ledge Shear Resistance

Reference Drawings:
B-242 (Ref. 14), B-257 (Ref. 27), B-672 (Ref. 21), B-687 (Ref. 28).

The ledges where the shield plugs are being seated are part of the 5'-0" thick circular wall that forms the top part of the shield structure. Each ledge is 5" wide and 2'-1" deep. The entire surface covering the three ledges is reinforced by a 1/2" thick stainless steel plate that is anchored to the concrete wall by steel straps. For each ledge there are two sets of straps. One set is horizontal and the second set is diagonal and both sets are spaced at 12" on center.

By comparison to the shear resistance of the shield plug seated on the ledge, the ledge shear resistance is substantially larger. In addition, all the ledges are continuous and all are part of the top of the shield wall.

Based on the above facts, by engineering judgment, the shear resistance of each ledge is higher than the shear resistance of the shield plug. Therefore, the shear check of the shield plug governs.

The following conditions will be addressed in the calculation below:

1. The drop of a lower layer plug onto the cavity and dry well head
2. The drop of a middle layer plug on the top of the 2 bottom layer plugs (all plugs from Unit 3).
3. The drop of a top layer plug on the top of the 2 middle layer plugs (all plugs from Unit 3).
4. The drop of a Unit 3 top layer plug on the top of the 2 Unit 2 top layer plugs.
5. The drop of a Unit 3 middle layer plug on a Unit 3 top layer plug, which is sitting on a Unit 2 top layer plug.

1. The Drop of a Bottom Layer Plug onto the Cavity and Dry Well Head

Per Ref. 26, the energy demand was calculated to be 636 kip-in for a plug weight of 184.5 kips and a lifting height of 6" above the floor. Readjust the energy demand for the higher plug weight and an additional 6" lifting height.

$W_{bottom} := 216 \cdot kip$ ....weight of the bottom layer plug per Ref. 23

$$\alpha_{2\_new} := \frac{W_{bottom}}{184.5 \cdot kip} \qquad \alpha_{2\_new} = 1.171$$

To account for the higher lift height of 12" (instead of 6") above the floor, the parameter $\alpha 1$ as calculated in Ref. 23 must be adjusted as follows:

$\Delta h_{drop} := 6 \cdot in$

$$\alpha_{1\_new} := \frac{(6.67 \cdot ft + \Delta h_{drop}) + 2.73 \cdot ft}{6.67 \cdot ft + 1.88 \cdot ft} \qquad \alpha_{1\_new} = 1.158$$

$$E_{new} := \alpha_{1\_new} \cdot \alpha_{2\_new} \cdot (540 \cdot in \cdot kip) \qquad E_{new} = 732.015 \, kip \cdot in$$

Based on Ref. 23, the drywell head is capable of absorbing 1800 kip-in of energy within the allowable strains. Therefore, the drop of 1'-0" above the floor is acceptable.

2. The Drop of a Middle Layer Plug on the Top of the 2 Bottom Layer Plugs
(All plugs from Unit 3).

$$\Delta_e := \left[\left(\frac{|y_{max_3}|}{q_{unit}}\right) \cdot qu_3\right] \qquad \Delta_e = 0.138\,ft \qquad \Delta_e = 1.658\,in$$

Per Ref. 4, a ductility of 10 may be used. Conservatively, ductility of 5.0 is used.

$$\mu := 5.0$$

$$E_s := R \cdot (\mu - 0.5) \cdot \frac{\Delta_e}{2} \qquad E_s = 584.47\,ft\,kip$$

$M := 224 \cdot kip$ ....weight of the middle layer plug

$M_1 := 2 \cdot (216 \cdot kip)$ ....weight of the 2 plugs resisting the drop (bottom layer plug)

Per Ref. 3, page 718, Case 2

$$k := \frac{1 + \frac{17}{35} \cdot \frac{M_1}{M}}{\left(1 + \frac{5}{8} \cdot \frac{M_1}{M}\right)^2} \qquad k = 0.398$$

Impact energy to be absorbed by the impacted plugs = M * h * k. The drop will engage two lower plugs.

$$h := \frac{2E_s}{M \cdot k} \qquad h = 13.105\,ft$$

Maximum height of drop of the shield plug on the middle layer plugs

3. <u>The Drop of a Top Layer Plug on the Top of the 2 Middle Layer Plugs</u>
(All plugs from Unit 3).

The drop of a top layer plug on the middle layer plugs will have less drop height and more in-place plugs to resist the drop. Therefore, the lifting height of 1'-0" is acceptable.

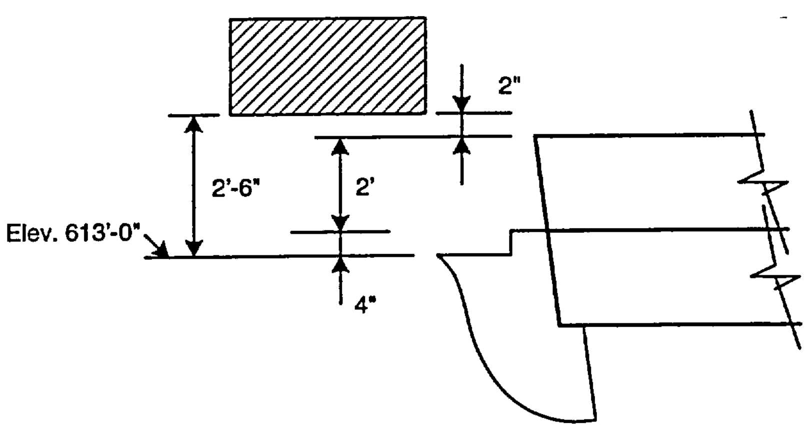
Summary of Unit 3 Plugs over Unit 3 Reactor Cavity

Based on the determined height of drop above, the Unit 3 shield plugs can be safely lifted up to 1'-0" above the floor at Elev. 613'-0" above the Reactor Cavity.

4. The Drop of a Unit 3 Top Layer Plug on the Top of the 2 Unit 2 Top Layer Plugs

The drop of a Unit 3 top layer plug on the Unit 2 top layer plugs will have less drop height and more in-place plugs to resist the drop than Item 2 above. Therefore, it is not critical to determine the safe maximum lifting height.

5. The Drop of a Unit 3 Middle Layer Plug on a Unit 3 Top Layer Plug, which is sitting on a Unit 2 Top Layer Plug



Based on the load path and laydown process, the laydown area of the Unit 3 top and middle plugs will be on top of the Unit 2 top plugs. The Unit 3 middle plugs will be 2" above the in-place Unit 3 top plugs during stacking.

Calculate the impacted energy which will be realized by the two plugs being impacted.

$M := 232\cdot kip$ ....weight of a top layer plug

$M_1 := 2\cdot(232\cdot kip)$ ....weight of the plugs resisting the drop (2 top layer plugs)

$$k := \frac{1 + \frac{17}{35}\cdot\frac{M_1}{M}}{\left(1 + \frac{5}{8}\cdot\frac{M_1}{M}\right)^2}$$

$k = 0.389$ ....Ref. 3, page 718, Case 2

$E_{lo} := 2 \cdot in \cdot (232 \cdot kip) \cdot k$ $\qquad E_{lo} = 180.69\,kip \cdot in$

Calculate the strain energy of the two plugs at flexural yield

$$\Delta_e := \left[\left(\frac{\left|y_{max_1}\right|}{q_{unit}}\right) \cdot qu_1\right] \qquad \Delta_e = 0.149\,ft \qquad \Delta_e = 1.786\,in$$

$$UL_{min} = 1.88 \times 10^3\,kip$$

$$E_{se} := \frac{2 \cdot UL_{min}}{2} \cdot \frac{\Delta_e}{2} \qquad E_{se} = 1.679 \times 10^3\,kip \cdot in \quad > \quad E_{lo} = 180.69\,kip \cdot in$$

Therefore, the impact is acceptable and the impacted plugs will remain elastic due to the impact.

Calculation to Assess Potential Perforation

This section of the calculation will address the potential of perforation of the floor slab at Elev. 613' due to impact by the dropped shield plug.

Perforation is not possible due to the following reasons:

1. Both plug and slab are made of reinforced concrete and may suffer local crushing at the impacted surfaces during impact
2. The impact area is large which reduces the impact intensity.
3. The impact velocity is very low considering that the maximum drop is 2.5 ft.

Additionally, the calculation on the next page confirms that scabbing is not expected to occur. Non-occurrence of scabbing implies that perforation can be ruled out.

Calculation to Assess Potential for Scabbing

This section of the calculation will address the potential of scabbing of the floor slab at Elev. 613' due to impact by the dropped shield plug.

To address the scabbing of the bottom face of the slab, Equations 4.1.1.1.2-1a and 4.1.1.1.2-5 of Ref. 1 are used. Note that scabbing will not occur if the failure is above the crane hook. Failure of a lug may cause the impact at the plug corner with a smaller impact area. Consider that an impact area of one square foot as the plug drops and impact the floor surface.

$d := \sqrt{\frac{1ft^2}{\pi} \cdot 2}$ $d = 13.541\,in$ ....diameter of an equivalent circular missile

$N := 0.72$ ....flat nosed body

$k := \frac{180}{\sqrt{\frac{f_c}{psi}}}$ $k = 2.626$

$h := 2.5 \cdot ft$ ....drop height considered

$V := \sqrt{2 \cdot \left(32.2 \cdot \frac{ft}{sec^2}\right) \cdot h}$ $V = 12.689\frac{ft}{sec}$

Conservatively consider that the weight of the plug corner which may break away during impact is 10000 lb.

$W_{corner} := 10000 \cdot lb$

$$x := \left[4 \cdot k \cdot N \cdot \frac{W_{corner}}{lb} \cdot \frac{d}{in} \cdot \left(\frac{V \cdot \frac{sec}{ft}}{1000 \cdot \frac{d}{in}}\right)^{1.8}\right]^{0.5} \cdot in$$

$x = 1.904\,in$ $\frac{x}{d} = 0.141 < 0.65$

$t_s := d \cdot \left[7.91 \cdot \left(\frac{x}{d}\right) - 5.06 \cdot \left(\frac{x}{d}\right)^2\right]$ $t_s = 13.708\,in < 18"$ (minimum slab thickness)
Scabbing is not likely to occur.

Scenario 2

Scenario 2

Full drop of a shield plug on a single column (Drop Height = 1'-0").

COLUMN M-47 (ELEVATION 613'-0")

The column at Column Row M-47 controls. Compute the maximum axial capacity of the concrete column at Column M-47, at refueling floor Elevation 613'-0".

$b_{col} := 33 \cdot in$ (Ref. 18)

$h_{col} := 33 \cdot in$ (Ref. 18)

$A_g := (b_{col}) \cdot (h_{col})$

$A_g = 1089\, in^2$

Longitudinal reinforcement consists of 12 #11 bars (Ref. 18).

$A_{s11} := 1.56 \cdot in^2$ (Ref. 5)

$d_{11} := 1.41 \cdot in$ (Ref. 5)

$N_{11} := 12$ (Ref. 18)

$A_{st} := (N_{11}) \cdot (A_{s11})$

$A_{st} = 18.72\, in^2$

Stirrups are #4 @ 18" (Type 2) (see B-665 [Ref. 18]):

$A_{st4} := .20 \cdot in^2$ (Ref. 5)

$d_4 := 0.5 \cdot in$ (Ref. 5)

Determine the controlling mode of failure for Column M-47. Two modes of failure will be investigated, based on the following parameters:

1. Buckling capacity of column
2. Crushing capacity of column based on modified ACI Code formula

Tabulate and compute the material properties of the column:

$$E_{steel} := 29000 \cdot ksi$$

$$E_{conc} := (57000) \cdot \sqrt{(f_c) \cdot (DIF_c) \cdot (psi)}$$

$$E_{conc} = 3907.7\,ksi$$

$$N := \frac{E_{steel}}{E_{conc}}$$

$$N = 7.421$$

Compute the gross moment of inertia of the column:

$$I_g := \left(\frac{1}{12}\right) \cdot (b_{col}) \cdot (h_{col})^3$$

$$I_g = 98826.8\,in^4$$

Compute the cracked moment of inertia of the column using the methodology given in Reference 1, Figure 3.1.10.

$cover_{col} := 1.5 \cdot in$ (Ref. 5)

$$d_{col} := h_{col} - cover_{col} - d_4 - (0.5) \cdot (d_{11})$$

$$d_{col} = 30.295\,in$$

$$As_{col\_pos} := (4) \cdot (A_{s11})$$

$$As_{col\_pos} = 6.24\,in^2$$

$\rho_{col} := \dfrac{As_{col\_pos}}{(b_{col}) \cdot (d_{col})}$ (Ref. 1, Figure 3.1.10)

$$\rho_{col} = 0.006242$$

$\rho n := (\rho_{col}) \cdot (N)$ (Ref. 1, Figure 3.1.10)

$$\rho n = 0.04632$$

$$As_{col\_neg} := (4)\cdot(A_{s11})$$

$$As_{col\_neg} = 6.24\,in^2$$

$$\rho_{col\_neg} := \frac{As_{col\_neg}}{(b_{col})\cdot(d_{col})}$$ (Ref. 1, Figure 3.1.10)

$$\rho_{col\_neg} = 0.006242$$

$$\rho n_{neg} := (\rho_{col\_neg})\cdot(N)$$ (Ref. 1, Figure 3.1.10)

$$\rho n_{neg} = 0.04632$$

$$\rho_{ratio} := \frac{\rho n_{neg}}{\rho n}$$ (Ref. 1, Figure 3.1.10)

$$\rho_{ratio} = 1$$

Determine the coefficient "F" from Reference 1, Figure 3.1.10 for the following values:

$\rho_{ratio} = 1$ $\rho n = 0.04632$

$F := 0.032$ (Ref. 1, Figure 3.1.10)

Compute the cracked moment of inertia for the column using Reference 1, Figure 3.1.10:

$$I_{cr} := (F)\cdot\left[(b_{col})\cdot(d_{col})^3\right]$$

$$I_{cr} = 29361.4\,in^4$$

Compute the average moment of inertia for the column using Reference 1:

$$I_a := \frac{I_g + I_{cr}}{2}$$

$$I_a = 64094.1\,in^4$$

Compute the clear length of the column between Elevation 613'-0" and Elevation 589'-0" (reduce gross column length by depth of shallowest beam framing into the column at Elevation 613'-0":

$$L_{col} := (613 - 589)\cdot ft - (4\cdot ft)$$

$$L_{col} = 20\,ft$$

Tabulate the value of "k" for the column (use k = 0.8):

$$k_{col} := 0.8$$

Determine Buckling Capacity of Column:

$$P_{crit\_col} := \frac{(\pi)^2\cdot(E_{conc})\cdot(I_a)}{\left[(k_{col})\cdot(L_{col})\right]^2}$$

$$P_{crit\_col} = 67056.2\,kip$$

Determine Crushing Capacity of Column:

Determine the crushing capacity of the column by modifying the ACI Code (Reference 5) formula for compression (at minimum eccentricity). The ACI formula will be modified by replacing the 0.8 factor in the numerator with 1.0 (The minimum eccentricity requirement that necessitates the 0.80 factor does not apply here).

$$\Phi_c := 0.70 \qquad \text{(Ref. 5)}$$

$$P_{crush\_col} := (1.00)\cdot(\Phi_c)\cdot\begin{bmatrix}(0.85)\cdot(f_c\cdot DIF_c)\cdot(A_g - A_{st}) \ldots \\ + (f_y\cdot DIF_s)\cdot(A_{st})\end{bmatrix} \qquad \text{(Ref. 5)}$$

$$P_{crush\_col} = 3779.3\,kip$$

Determine Controlling Column Capacity:

$$P_{col\_control} := \min(P_{crit\_col}, P_{crush\_col})$$

$$P_{col\_control} = 3779.3\,kip$$

Applied Energy of Load Drop and Energy Absorbing Capacity of Structure

Compute Area of Concrete Slab Affected by the Shield Plug Drop:

This is the area of the slab to be considered as effective mass sitting on top of the column.

$R_{plug} := (21 \cdot ft + 5 \cdot in)$ $R_{plug} = 21.417 ft$ (Plug Radius)

The slabs around Column Row M-47 are 18" thick. Compute the area of the slab to be considered as effective mass by adding the plug radius and the slab thickness.

$t_{slab} := 18 \cdot in$

$R_S := R_{plug} + t_{slab}$ (Effective radius)

$R_S = 22.917 ft$

$$A_{plug} := \frac{(\pi) \cdot (R_S)^2}{2}$$ (Effective area)

$A_{plug} = 824.941 ft^2$

Tabulate Weight of Upper Concrete Shield Plug:

$P_{plug} := 232 \cdot kip$

Compute Weight of Concrete Structures Near Column M-47 to be Considered Part of Effective Mass Sitting on Top of the Column:

The concrete shield plug will be moved from the area above the Reactor cavity (near Column Rows K-47) south along Column Row 47 to a location possibly above the concrete column at Column Row M-47. During this move, the center of gravity of the shield plug will be aligned with Column Row 47.

An investigation will be made to determine the effect of a load drop near Column Row M-47 of the shield plug. The weight (mass) of the existing concrete structure under the shield plug at Column Row M-47 must be determined. The existing concrete structure includes the concrete column, beams, and slabs. The effective length of the column will be taken down to the top of the next slab (at Elevation 589'-0").

The slabs around Column Row M-47 are 18" thick. The east-west beams (5B10) framing into Column M-47 are 24" wide x 48" deep (the depth includes the slab thickness). The north-south beams (5B5 and 5B6) framing into Column M-47 are 27" wide x 54" deep

Weight of Slab (Under Shield Plug):

$\gamma_{conc} := 150 \cdot pcf$

$M_{1a} := (A_{plug}) \cdot (\gamma_{conc}) \cdot (t_{slab})$

$M_{1a} = 185.61\,kip$

Weight of Beams (Under Shield Plug):

Determine net weight of beams framing into Column M-47.

This computation considers that the long axis of the shield plug (2 x plug radius) is orientated in the north-south direction.

$b_{NSbm} := 27 \cdot in$

$h_{NSbm} := 54 \cdot in$

$b_{EWbm} := 24 \cdot in$

$h_{EWbm} := 48 \cdot in$

$L_{net\_NSbm} := (2)(R_S) - h_{col}$

$L_{net\_NSbm} = 43.083\,ft$

$L_{net\_EWbm} := (R_S) - b_{col}$

$L_{net\_EWbm} = 20.167\,ft$

$M_{1b1} := (b_{NSbm}) \cdot (h_{NSbm} - t_{slab}) \cdot (L_{net\_NSbm}) \cdot (\gamma_{conc})$

$M_{1b1} = 43.622\,kip$

$M_{1b2} := (b_{EWbm}) \cdot (h_{EWbm} - t_{slab}) \cdot (L_{net\_EWbm}) \cdot (\gamma_{conc})$

$M_{1b2} = 15.125\,kip$

$M_{1b} := M_{1b1} + M_{1b2}$

$M_{1b} = 58.747\,kip$

Weight of Column (Under Shield Plug):

Determine net weight of Column M-47 between Elevations 589'-0" and 613'-0".

$$L_{net\_col} := (613 - 589)\cdot ft - t_{slab}$$

$$L_{net\_col} = 22.5\,ft$$

$$M_{1c} := (b_{col})\cdot(h_{col})\cdot(L_{net\_col})\cdot(\gamma_{conc})$$

$$M_{1c} = 25.52\,kip$$

Total Weight of Existing Concrete Structure (Under Shield Plug):

$$M_1 := M_{1a} + M_{1b} + M_{1c}$$

$$M_1 = 269.882\,kip$$

Determine Energy Losses if Shield Plug is Dropped on Top of Column M-47:

Refer to Roark & Young (6th Edition), Chapter 15, page 718 (Reference 3).

Determine K factor. Use Case 1 (moving body of mass "M" strikes axially one end of a bar of mass M1, the other end of which is fixed, with additional mass [slab + beams] at the end of the bar).

$$K := \frac{\left[1 + \left(\frac{1}{3}\right)\cdot\left(\frac{M_{1c}}{P_{plug}}\right) + \left(\frac{M_{1a} + M_{1b}}{P_{plug}}\right)\right]}{\left[1 + \left(\frac{1}{2}\right)\cdot\left(\frac{M_{1c}}{P_{plug}}\right) + \left(\frac{M_{1a} + M_{1b}}{P_{plug}}\right)\right]^2}$$

$$K = 0.47$$

$$drop := 1.00\cdot ft$$

$$E_{final} := (P_{plug})\cdot(drop)\cdot(K)$$

$E_{final} = 109.085\,kip\cdot ft$ (Impact Energy)

Maximum strain in concrete column:

$\varepsilon := 0.002$

$L_{net\_col} = 22.5\,ft$

Compute deflection of column at maximum strain:

$\Delta_e := (\varepsilon)\cdot(L_{net\_col})$

$\Delta_e = 0.54\,in$

Tabulate the allowable ductility ratio of the concrete column for impulse and impact load (Reference 1, Table 5.1, page 2-112):

$\mu_{duct} := 1.3$ (Ref. 1, Table 5.1, page 2-112)
(Ref. 4, Appendix C)

Compute the energy absorbing capacity of the concrete column (by computing the area under the load-deflection curve):

$E_s := (P_{col\_control})\cdot\left[(\Delta_e)\cdot(0.5) + \left[(\mu_{duct})\cdot(\Delta_e) - (\Delta_e)\right]\right]$

$E_s = 136.054\,kip\cdot ft$ (Strain Energy)

Compare the applied energy with the energy absorbing capacity of the structure.

$E_{final} = 109.085\,kip\cdot ft$ < $E_s = 136.054\,kip\cdot ft$ OK

Modify the energy absorbing capacity of the concrete column (computed above) by including the effect of the dead load carried by the column.

$P_{col\_DL} := M_{1a} + M_{1b}$

$P_{col\_DL} = 244.359\,kip$

Compute Reduction in Energy Absorbing Capacity of Column from Dead Load:

$E_{col\_DL} := (P_{col\_DL})\cdot(\mu_{duct})\cdot(\Delta_e)$

$E_{col\_DL} = 14.295\,kip\cdot ft$ (Dead Load Strain Energy)

Compute Modified Energy Absorbing Capacity of Column from Dead Load:

$E_{sf} := E_s - E_{col\_DL}$

$E_{sf} = 121.759\,kip\cdot ft$ (Net Available Strain Energy)

Compare the applied energy with the energy absorbing capacity of the structure.

$E_{final} = 109.085\,kip\cdot ft$ < $E_{sf} = 121.759\,kip\cdot ft$ OK

Summary

The concrete column at Column Row M-47 is capable of withstanding a postulated load drop for the drop height tabulated below:

$drop = 1\,ft$

for the load tabulated below:

$P_{plug} := 232\cdot kip$

using the DIFs tabulated below:

$DIF_c = 1$

$DIF_s = 1$

Scenario 3

Scenario 3

Full Drop of a Shield Plug on a System of Two Adjacent Slabs with a Beam in between the Slabs (Drop Height = 2'-6").

BEAMS

Beams 5B7, 5B6, and 5B5 control for north-south beams and Beam 5B10 controls for east-west beams. Beams 5B6 and 5B10 will be evaluated in this calculation.

Beam 5B6:

Beam Flexural Capacity:

This calculation considers that the postulated drop occurs when the shield plug diameter is parallel to the east-west direction with the shield plug centered over the beam. For this configuration, the impact of the postulated drop will be resisted by the beam and the slabs on both sides of the beam.

From previous computations:

$f_c = 4700\,\text{psi}$

$DIF_c = 1$

$DIF_s = 1$

Compute the flexural capacity of north-south beam 5B6.

Beam properties and reinforcing are tabulated on drawing B-666 (Ref. 19).

$b_{NSbm} = 27\,\text{in}$

$h_{NSbm} = 54\,\text{in}$

Flexural Reinforcement:

Bottom Bars (Positive Moment):
5 #11 "A" Bars, no "B" Bars

Top Bars (Negative Moment) (North End):
"C" Bars: 6 #11; Cut-Off: 4 #11 @ 9'-0"
"D" Bars: 4 #11; Cut-Off: 4 #11 @ 9'-0"

Top Bars (Negative Moment) (South End):
"C" Bars: 6 #11; Cut-Off: 4 #11 @ 12'-0"
"D" Bars: None

$x_{sp} := 3.5 \cdot in$ Spacing of rebar layers "A" & "B" and "C" & "D"

$cover := 1.50 \cdot in$ (Ref. 12 and 19)

$cover_n := 3.00 \cdot in$ (Ref. 12 and 19)

$d_{stirrup} := 0.5 \cdot in$ (Ref. 12 and 19)

$d_{11} = 1.41\,in$ (Ref. 4)

Compute effective depth for positive and negative moment reinforcement:

$$d_{pos} := h_{NSbm} - cover - d_{stirrup} - (0.5) \cdot (d_{11})$$

$$d_{pos} = 51.295\,in$$

$$d_{neg\_N} := h_{NSbm} - cover_n - d_{stirrup} - (0.5) \cdot (d_{11}) - (0.5) \cdot (x_{sp})$$

$$d_{neg\_N} = 48.045\,in$$

$$d_{neg\_S} := h_{NSbm} - cover_n - d_{stirrup} - (0.5) \cdot (d_{11})$$

$$d_{neg\_S} = 49.795\,in$$

Compute the average value of "d" for negative moment reinforcement in the beam:

$$d_{n\_av} := \frac{d_{neg\_N} + d_{neg\_S}}{2}$$

$$d_{n\_av} = 48.92\,in$$

Compute Flexural Capacity of Beam:

Tabulate properties and constants:

$\Phi_m := 0.90$

$N_{pos} := 5$

$N_{neg\_N} := 10$ (Maximum)

$N_{neg\_S} := 6$ (Maximum)

$A_{s11} = 1.56\,in^2$

$A_{s\_pos} := (N_{pos})\cdot(A_{s11})$ $A_{s\_pos} = 7.8\,in^2$

$A_{s\_neg\_N} := (N_{neg\_N})\cdot(A_{s11})$ $A_{s\_neg\_N} = 15.6\,in^2$ (Maximum)

$A_{s\_neg\_S} := (N_{neg\_S})\cdot(A_{s11})$ $A_{s\_neg\_S} = 9.36\,in^2$ (Maximum)

Flexural Strength of Beam:

$f_c = 4700\,psi$

Compute the value of β_1 for the specified concrete strength (between 4000 psi and 5000 psi).

$$\beta_1 := 0.85 - (0.05)\cdot\left[\frac{f_c - (4000\cdot psi)}{1000\cdot psi}\right]$$

$\beta_1 = 0.815$

Positive Moment Flexural Capacity:

$T_{pos} := (A_{s\_pos})\cdot(f_y)\cdot(DIF_s)$

$T_{pos} = 468\,kip$

$$a_{pos} := \frac{T_{pos}}{(\beta_1)\cdot(f_c)\cdot(DIF_c)\cdot(b_{NSbm})}$$

$a_{pos} = 4.525\,in$

$$\Phi M_{n\_pos} := (\Phi_m)(T_{pos}) \cdot \left(d_{pos} - \frac{a_{pos}}{2}\right)$$

$$\Phi M_{n\_pos} = 1721\,\text{ft} \cdot \text{kip}$$

Negative Moment Flexural Capacity (North End):

$$T_{neg\_N} := (A_{s\_neg\_N}) \cdot (f_y) \cdot (DIF_s)$$

$$T_{neg\_N} = 936\,\text{kip}$$

$$a_{neg\_N} := \frac{T_{neg\_N}}{(\beta_1) \cdot (f_c) \cdot (DIF_c) \cdot (b_{NSbm})}$$

$$a_{neg\_N} = 9.05\,\text{in}$$

$$\Phi M_{n\_neg\_N} := (\Phi_m)(T_{neg\_N}) \cdot \left(d_{neg\_N} - \frac{a_{neg\_N}}{2}\right)$$

$$\Phi M_{n\_neg\_N} = 3055.1\,\text{ft} \cdot \text{kip}$$

Negative Moment Flexural Capacity (South End):

$$T_{neg\_S} := (A_{s\_neg\_S}) \cdot (f_y) \cdot (DIF_s)$$

$$T_{neg\_S} = 561.6\,\text{kip}$$

$$a_{neg\_S} := \frac{T_{neg\_S}}{(\beta_1) \cdot (f_c) \cdot (DIF_c) \cdot (b_{NSbm})}$$

$$a_{neg\_S} = 5.43\,\text{in}$$

$$\Phi M_{n\_neg\_S} := (\Phi_m)(T_{neg\_S}) \cdot \left(d_{neg\_S} - \frac{a_{neg\_S}}{2}\right)$$

$$\Phi M_{n\_neg\_S} = 1983\,\text{ft} \cdot \text{kip}$$

Compute Average Value of Negative Moment Flexural Capacity (North and South Ends):

$$\Phi M_{n\_neg\_average} := \frac{\left(\Phi M_{n\_neg\_N} + \Phi M_{n\_neg\_S}\right)}{2}$$

$$\Phi M_{n\_neg\_average} = 2519.1\,ft\cdot kip$$

Compute Resistance Factors for Beam

Compute the resistance factors for the beam.

(Reference 1, Table 5.2, page 2-113).

Use formulation for a multi-span beam.

Moment Resistance Factors:

$L_{bm} := 25.75\cdot ft$ (Total Length of Beam 5B6)

Moment Resistance Factor:

$$R_M := \frac{(4)\cdot\left(\Phi M_{n\_neg\_average} + \Phi M_{n\_pos}\right)}{L_{bm}}$$

$$R_M = 658.655\,kip$$

$$R_{beam} := R_M$$

$$R_{beam} = 658.655\,kip$$

Note that this is the resistance based on a concentrated load, which is more likely representative of impact due to lug failure instead of failure above the crane hook.

Therefore, this application is conservative since the impact energy due to lug failure (side drop resulting in concentrated loading) is half that resulting from full mass drop through the total drop height.

As a result of this conservative approach, this evaluation covers both types of impact.

Compute Allowable Ductility Ratio:

Compute the allowable ductility using Reference 4, Section C.3.

Based on a review of the number of positive and negative reinforcing bars at the middle and ends of the beam, it is concluded that the middle of the beam is a reasonable location to apply the equation of Reference 4, Section C.3.3.

Note that the Reference 4 Commentary indicates that, in the equation for the permissible ductility ratio, the coefficient of 0.05 was chosen instead of 0.10 to provide additional margin of safety against overestimating ductility.

$N_{pos} = 5$

$N_{neg\_N} = 10$ (Maximum)

$N_{neg\_S} = 6$ (Maximum)

$N_{C\_cutoff} := 4$ (Ref. 12 and 19)

$N_{D\_cutoff} := 4$ (Ref. 12 and 19)

Compute number of negative moment rebars at beam mid-span.

$$N_{neg\_midspan} := N_{neg\_N} - N_{C\_cutoff} - N_{D\_cutoff}$$

$$N_{neg\_midspan} = 2$$

Compute permissible ductility per Reference 4, Section C.3.3:

$$\rho_{p\_bm} := \frac{(N_{pos}) \cdot (A_{s11})}{(b_{NSbm}) \cdot (d_{pos})}$$

$$\rho_{p\_bm} = 0.005632$$

$$\rho_{n\_bm} := \frac{(N_{neg\_midspan}) \cdot (A_{s11})}{(b_{NSbm}) \cdot (d_{pos})}$$

$$\rho_{n\_bm} = 0.002253$$

$$\mu_{duct\_limit\_bm\_midspan} := \frac{0.05}{\left(\rho_{p\_bm} - \rho_{n\_bm}\right)}$$

$\mu_{duct\_limit\_bm\_midspan} = 14.797$ > 10 (Max. per Ref. 4, Sect. C.3.3)

Use $\mu = 10$ as a maximum based on Reference 4, Section C3.3.

Beam Shear Capacity:

Compute the shear capacity of north-south beam 5B6.

Beam properties and reinforcing are tabulated on drawing B-666 (Ref. 19).

Beam 5B6:

Shear Reinforcement:

Stirrups (# 4 bars) are placed as follows:

18 bars @ 6" spacing from North end.
Balance of bars are placed @ 12" spacing.

$d_{stirrup} = 0.5\,in$

$A_{stirrup} := 0.20 \cdot in^2$

$A_v := (2) \cdot (A_{stirrup})$

$A_v = 0.4\,in^2$

$sp_{stir\_N} := 6 \cdot in$ (Stirrup spacing at north end [to 9'-0" from north supp.])

$sp_{stir\_S} := 12 \cdot in$ (Stirrup spacing at south end)

Use an average stirrup spacing of 9 inches:

$sp_{stir\_average} := 9 \cdot in$

Compute Shear Strength:

$\Phi_S := 0.85$

Use average "d" for shear strength computation:

$d_{n\_av} = 48.92\,in$

$\Phi V_{c\_bm\_aver} := (\Phi_S) \cdot (2) \cdot \sqrt{f_c} \cdot (\sqrt{psi}) (b_{NSbm}) \cdot (d_{n\_av})$

$\Phi V_{c\_bm\_aver} = 153.9\,kip$

$$\Phi V_{s\_bm\_aver} := \frac{(A_v)\cdot(f_y)\cdot(d_{n\_av})}{sp_{stir\_average}}$$

$$\Phi V_{s\_bm\_aver} = 130.5\,kip$$

$$\Phi V_{n\_bm\_aver} := \Phi V_{c\_bm\_aver} + \Phi V_{s\_bm\_aver}$$

$$\Phi V_{n\_bm\_aver} = 284.4\,kip$$

Compute Shear Capacity and Shear Resistance Factors Including Adjacent Slab

The concrete beam cannot fail without also failing the slab.

Compute the shear capacity and shear resistance of the slab itself. For this computation, the effective width of the slab will be taken as the total clear length around the outer perimeter of the two slabs adjacent to Beam 5B6. Note that the length of the sides of the slabs directly adjacent to Beam 5B6 are not included in the total length computation, because Beam 5B6 and the two adjacent slabs are postulated to fail as a single system.

$t_{slab} = 18\,in$ (Slab thickness)

$a_{slab} := 24.5\cdot ft$ (Minimum E-W center-center slab dimension)

$b_{slab} := 25.75\cdot ft$ (Maximum N-S center-center slab dimension)

$b_{adj\_bm\_min} := 2.00\cdot ft$ (Minimum width of adjacent beams)

Tabulate widths of adjacent beams:

$b_{5B4} := 36\cdot in$

$b_{NSbm} = 27\,in$ (Beam 5B6)

$b_{5B23} := 36\cdot in$

$b_{5B10} := 24\cdot in$

$b_{wall} := 30\cdot in$ (North wall)

$$L_{eff\_slab} := (2)\cdot\left[(2)\cdot(a_{slab}) - (0.5)\cdot(b_{5B4} + b_{5B23}) - b_{NSbm}\right] \cdots + (2)\cdot\left[b_{slab} - (0.5)\cdot(b_{5B10}) - b_{wall}\right]$$

$$L_{eff\_slab} = 1584\,in$$

Slab Reinforcement (Drawings B-237 and B-667 [Ref. 13 and 20]):

Slab "R" (Ref. 13 and 20):

Flexural Reinforcement:

All bars are # 8

Bottom Bars (Positive Moment): N-S # 8 @ 12"; E-W # 8 @ 9"

Top Bars (Negative Moment): N-S # 8 @ 6"; E-W # 8 @ 6"

$d_8 := 1.00 \cdot in$

$A_{s8} := 0.79 \cdot in^2$

$cover_{slab} := 1.00 \cdot in$ (Ref. 13 and 20)

$d_{slab} := t_{slab} - cover_{slab} - (0.5) \cdot (d_8)$

$d_{slab} = 16.5\,in$

Slab Shear Capacity:

$\Phi V_{n\_slab} := (\Phi_S) \cdot (2) \cdot \sqrt{f_c} \cdot (\sqrt{psi}) (L_{eff\_slab}) \cdot (d_{slab})$

$\Phi V_{n\_slab} = 3046\,kip$

Compute Shear Resistance Factors Including Effect of Slab

Compute the shear resistance factor for a concentrated load applied to the beam and slab at mid-span. For this case, the reaction at each end is one half the applied load.

Shear Resistance Factor:

$R_{v\_combined} := (2) \cdot (\Phi V_{n\_bm\_aver}) + \Phi V_{n\_slab}$

$R_{v\_combined} = 3614.8\,kip$

Compute Slab Section Properties

Compute the section properties of the slab.

$b_{eff\_slab} := 1 \cdot in$ (Unit width of slab)

The slab unit width of 1 inch is used so that the formulations are consistent with Ref. 1 Table 5.3, which are given in terms of capacity per inch unit width of slab.

$t_{slab} = 18\,in$

$cover_{slab} = 1\,in$

$d_8 = 1\,in$

$d_{slab} = 16.5\,in$

$E_{steel} = 29000\,ksi$

$E_{conc} = 3907.7\,ksi$

$N = 7.421$

Compute the gross moment of inertia of the slab:

$$I_{g\_slab} := \left(\frac{1}{12}\right) \cdot \left(b_{eff\_slab}\right) \cdot \left(t_{slab}\right)^3$$

$I_{g\_slab} = 486\,in^4$

Compute the cracked moment of inertia of the slab:

Use the methodology given in Reference 1, Figure 3.1.10.

The cracked moment of inertia will be computed in the negative moment region of the slab, adjacent to Beam 5B6. Therefore, the top bars (# 8 @ 6") will be used to compute ρ.

$As_{slab\_pos} := (2) \cdot \left(A_{s8}\right)$ N-S # 8 @ 6"

$As_{slab\_pos} = 1.58\,in^2$

$b_{bar\_slab} := 12 \cdot in$ (Effective width of slab for rebar spacing)

$$\rho_{p\_slab} := \frac{As_{slab\_pos}}{(b_{bar\_slab}) \cdot (d_{slab})}$$ (Ref. 1, Figure 3.1.10)

$\rho_{p\_slab} = 0.0079798$

$\rho n := (\rho_{p\_slab}) \cdot (N)$ (Ref. 1, Figure 3.1.10)

$\rho n = 0.05922$

Determine the coefficient "F" from Reference 1, Figure 3.1.10 for the following values:

$\rho n = 0.05922$

$F := 0.037$ (Ref. 1, Figure 3.1.10)

Compute the cracked moment of inertia for the slab using Reference 1, Figure 3.1.10:

$$I_{cr\_slab} := (F) \cdot \left[(b_{eff\_slab}) \cdot (d_{slab})^3 \right]$$

$I_{cr\_slab} = 166.2\, in^4$

Compute the average moment of inertia for the slab using Reference 1:

$$I_{a\_slab} := \frac{I_{g\_slab} + I_{cr\_slab}}{2}$$

$I_{a\_slab} = 326.1\, in^4$

Compute Slab Stiffness:

Use Reference 1, Table 5.3. Consider the slab to be fixed on all four sides with load at center of slab.

$\upsilon_c := 0.17$ (Poisson ratio for concrete)

$$ba_{ratio} := \frac{b_{slab}}{a_{slab}}$$

$ba_{ratio} = 1.051$

$\alpha_{slab} := 0.0671$ (Reference 1, Table 5.3)

$$K_{slab} := \frac{(12)\cdot(E_{conc})\cdot\left(\frac{I_{a\_slab}}{in}\right)}{(\alpha_{slab})\cdot(a_{slab})^2\cdot\left(1-\upsilon_c{}^2\right)}$$

$a_{slab} = 24.5\,ft$

$$K_{slab} = 2715.1\,\frac{kip}{in}$$

$$K_{slab} = 32580.8\,\frac{kip}{ft}$$

Compute Slab Average Reinforcement:

Determine the average slab reinforcement for positive and negative moment.

For positive moment reinforcement (bottom rebars), take the average of the reinforcement in the north-south and east-west directions. The bottom reinforcement is: N-S # 8 @ 12"; E-W # 8 @ 9"

For negative moment reinforcement (top rebars), take the average of the reinforcement in the north-south and east-west directions. The top reinforcement is: N-S # 8 @ 6"; E-W # 8 @ 6"

$$A_{pos\_slab} := (0.5)\cdot\left[A_{s8} + \left(\frac{12\cdot in}{9\cdot in}\right)\cdot(A_{s8})\right]$$

$$A_{pos\_slab} = 0.922\,in^2$$

$$A_{neg\_slab} := (0.5)\cdot\left[(2)(A_{s8}) + (2)\cdot(A_{s8})\right]$$

$$A_{neg\_slab} = 1.58\,in^2$$

Positive Moment Flexural Capacity:

$$T_{pos} := (A_{pos\_slab})\cdot(f_y)\cdot(DIF_s)$$

$$T_{pos} = 55.3\,kip$$

$$a_{pos} := \frac{T_{pos}}{(\beta_1)\cdot(f_c)\cdot(DIF_c)\cdot(b_{bar\_slab})}$$

$$a_{pos} = 1.203\,in$$

$$\Phi M_{n\_pos\_slab} := (\Phi_m)(T_{pos})\cdot\left(d_{slab} - \frac{a_{pos}}{2}\right)$$

$$\Phi M_{n\_pos\_slab} = 65.9\,ft\cdot kip$$

Negative Moment Flexural Capacity:

$$T_{neg} := (A_{neg\_slab})\cdot(f_y)\cdot(DIF_s)$$

$$T_{neg} = 94.8\,kip$$

$$a_{neg} := \frac{T_{neg}}{(\beta_1)\cdot(f_c)\cdot(DIF_c)\cdot(b_{bar\_slab})}$$

$$a_{neg} = 2.062\,in$$

$$\Phi M_{n\_neg\_slab} := (\Phi_m)(T_{neg})\cdot\left(d_{slab} - \frac{a_{neg}}{2}\right)$$

$$\Phi M_{n\_neg\_slab} = 109.98\,ft\cdot kip$$

Compute Slab Moment Resistance Factor

Compute the slab resistance factor using Reference 1, Table 5.3, for a slab with fixed supports on 4 sides and load applied at the center.

$$R_{M\_slab} := \frac{2\cdot\pi\cdot\left(\Phi M_{n\_pos\_slab} + \Phi M_{n\_neg\_slab}\right)}{b_{bar\_slab}}$$

$$R_{M\_slab} = 1105.4\,\text{kip}$$

Compute Beam Stiffness:

$$b_{NSbm} = 27\,\text{in}$$

$$h_{NSbm} = 54\,\text{in}$$

Compute the gross moment of inertia of the beam:

$$I_{g\_bm} := \left(\frac{1}{12}\right)\cdot\left(b_{NSbm}\right)\cdot\left(h_{NSbm}\right)^3$$

$$I_{g\_bm} = 354294\,\text{in}^4$$

$$I_{g\_bm} = 17.086\,\text{ft}^4$$

Compute the cracked moment of inertia of the beam:

Use the methodology given in Reference 1, Figure 3.1.10.

$$N_{pos} = 5$$

$$d_{pos} = 51.295\,\text{in}$$

$$As_{bm\_pos} := \left(N_{pos}\right)\cdot\left(A_{s11}\right)$$

$$As_{bm\_pos} = 7.8\,\text{in}^2$$

$$\rho_{p\_bm} := \frac{As_{bm\_pos}}{\left(b_{NSbm}\right)\cdot\left(d_{pos}\right)}$$ (Ref. 1, Figure 3.1.10)

$$\rho_{p\_bm} = 0.0056319$$

$$\rho n := (\rho_{p\_bm}) \cdot (N)$$ (Ref. 1, Figure 3.1.10)

$$\rho n = 0.0418$$

Determine the coefficient "F" from Reference 1, Figure 3.1.10 for the following values:

$$\rho n = 0.0418$$

$$F := 0.027$$ (Ref. 1, Figure 3.1.10)

Compute the cracked moment of inertia for the beam using Reference 1, Figure 3.1.10:

$$I_{cr\_bm} := (F) \cdot \left[(b_{NSbm}) \cdot (d_{pos})^3 \right]$$

$$I_{cr\_bm} = 98390.4\, in^4$$

$$I_{cr\_bm} = 4.745\, ft^4$$

Compute the average moment of inertia for the beam using Reference 1:

$$I_{a\_bm} := \frac{I_{g\_bm} + I_{cr\_bm}}{2}$$

$$I_{a\_bm} = 226342.2\, in^4$$

$$I_{a\_bm} = 10.915\, ft^4$$

Compute Combined Beam and Slab Moment Resistance Factor

Compute the combined beam and slab moment resistance factor by combining the moment resistance of Beam 5B6 with the moment resistance of the two (2) adjacent slabs.

$$R_M = 658.655\,\text{kip}$$

$$R_{beam} = 658.655\,\text{kip}$$

The beam moment resistance is reduced by a factor that accounts for the area of the two adjacent slabs that are tributary to the beam.

$$R_{M\_combined} := \left[R_{beam} - (2)\cdot\left(\frac{R_{M\_slab}}{4}\right)\right] + (2)\cdot\left(R_{M\_slab}\right)$$

$$R_{M\_combined} = 2316.7\,\text{kip}$$

Compute Ratio of Moment and Shear Resistance Factors

Compute the ratio of the shear resistance factor to the moment resistance factor of the combined beam and slab section.

$$Ratio_{MV} := \frac{R_{v\_combined}}{R_{M\_combined}}$$

$Ratio_{MV} = 1.56$ > 1.20 (Flexure Inelastic Behavior is applicable)

Compute Stiffness of Combined Beam and Slab Section:

Compute the stiffness of the beam / slab using the formula from the following reference.

(Reference 1, Table 5.2, page 2-112).

Use stiffness formulation for a multi-span beam.

Stiffness Using Nominal Concrete Strength (f'c = 4700 psi):

$$E_{conc} = 3907.7\,ksi$$

$$L_{bm} = 25.75\,ft$$

$$K_{e\_bm} := \frac{(92)\cdot(E_{conc})\cdot(I_{a\_bm})}{(L_{bm})^3}$$

$$K_{e\_bm} = 33096.6\,\frac{kip}{ft}$$

Compute Stiffness of Combined Beam and Slab Section

$$K_{slab} = 32580.8\,\frac{kip}{ft}$$

$$K_{combined} := (2)\cdot(K_{slab}) + K_{e\_bm}$$

$$K_{combined} = 98258.2\,\frac{kip}{ft}$$

Compute Deflection of Combined Beam and Slab Section:

$$\Delta_{e\_combined} := \frac{R_{M\_combined}}{K_{combined}}$$

$$\Delta_{e\_combined} = 0.283\,in$$

$$\Delta_{e\_combined} = 0.02358\,ft$$

Compute Energy Absorbing Capacity of Beam / Slab

Compute the total energy absorbing capacity under the Load - Deflection curve for the beam / slab system. The energy absorbing capacity will be based on an upper limit of (10)*($\Delta_{e\_combined}$) on the Load - Deflection curve.

At a deflection of 10 times the elastic deflection, the energy absorbing capacity of the beam / slab will be:

$$E_s := (R_{M\_combined}) \cdot (\Delta_{e\_combined}) \cdot (10 - 0.5)$$

$E_s = 518.9 \text{kip} \cdot \text{ft}$ (Strain Energy)

Determine Energy Losses if Shield Plug is Dropped on Beam / Slab:

Refer to Roark & Young (6th Edition), Chapter 15, page 718 (Reference 3).

Determine K factor. Use Case 4 (moving body of mass "M" strikes transversely the center of a beam with fixed ends and a mass of M11).

From previous computations:

$P_{plug} = 232\,kip$ (Weight of shield plug)

$M_{1a} = 185.612\,kip$ (Weight of slab under shield plug)

$$K_{bm} := \frac{\left[1 + \left(\frac{13}{35}\right)\cdot\left(\frac{M_{1a}}{P_{plug}}\right)\right]}{\left[1 + \left(\frac{1}{2}\right)\cdot\left(\frac{M_{1a}}{P_{plug}}\right)\right]^2}$$

$K_{bm} = 0.662$

Compute Kinetic Energy Applied to Beam / Slab:

Compute the kinetic energy applied to the beam / slab for the drop height given below:

$drop = 1\,ft$

Applied Kinetic Energy:

$E_{final\_bm} := (P_{plug})\cdot(drop)\cdot(K_{bm})$

$E_{final\_bm} = 153.536\,kip\cdot ft$ (Impact Energy)

Compare Applied Kinetic Energy to Energy Absorbing Capacity:

Compute the kinetic energy applied to the beam / slab for the drop height given below:

$drop = 1\,ft$

$E_{final\_bm} = 153.536\,kip\cdot ft$ < $E_s = 518.9\,kip\cdot ft$ OK

Compute Reduction in Energy Absorbing Capacity of from Dead Load:

Dead load of combined beam / slab system:

$M_{1a} = 185.612\,kip$

$W_{DL\_comb} := M_{1a}$

$W_{DL\_comb} = 185.612\,kip$

Conservatively compute the strain energy of the dead load.

$E_{DL\_comb} := \left(W_{DL\_comb}\right)\cdot(10)\cdot\left(\Delta_{e\_combined}\right)$

$E_{DL\_comb} = 43.763\,kip\cdot ft$

Compute Modified Energy Absorbing Capacity of Beam / Slab:

$E_{S\_final\_comb} := E_s - E_{DL\_comb}$

$E_{S\_final\_comb} = 475.1\,kip\cdot ft$ (Strain Energy)

<u>Compare Applied Kinetic Energy to Energy Absorbing Capacity:</u>

Compute the kinetic energy applied to the beam / slab for the drop height given below:

$drop = 1\,ft$

$E_{final\_bm} = 153.536\,kip\cdot ft$ < $E_{S\_final\_comb} = 475.1\,kip\cdot ft$ OK

<u>Compute Kinetic Energy Applied to Beam / Slab for Alternate Drop Height:</u>

Compute the kinetic energy applied to the beam / slab for the alternate drop height given below:

$drop_{alternate} := 2.5\cdot ft$

Applied Kinetic Energy:

$E_{final\_bm\_alt} := \left(P_{plug}\right)\cdot\left(drop_{alternate}\right)\cdot\left(K_{bm}\right)$

$E_{final\_bm\_alt} = 383.84\,kip\cdot ft$ (Impact Energy)

Compare Applied Kinetic Energy (Alternate) to Energy Absorbing Capacity:

Compute the kinetic energy applied to the beam / slab for the alternate drop height given below:

$$drop_{alternate} = 2.5\,ft$$

$E_{final\_bm\_alt} = 383.84\,kip \cdot ft$ < $E_{S\_final\_comb} = 475.1\,kip \cdot ft$ OK

Summary:

The system consisting of 2 Mark "R" slabs with beam type 5B6 in between is adequate for the most severe drop of the plug from a height of 2.5 feet.

Beam 5B10:

Drop Assessment When The Plug Orientation in The North-South Direction and Centered Between Two Columns With The C.G. at The Center Of Beam Type 5B10

This calculation section is added to address the potential of drop if the plug happens to be orientated in the north-south direction and located between two column lines of slabs type R and potentially centered over beam Type 5B10.

The slab's resistance in bending and shear has been evaluated earlier in this Scenario, where two adjacent type R slabs with beam type 5B6 were considered for plug drop on the beam and the two slabs. The drop height was determined to be 2.5 feet.

Since the slabs are identical, we will calculate beam 5B10 resistance in flexure and shear and compare these resistances to those of beam 5B6. If the comparison yields comparable resistances, it would be concluded that the drop height for this system will also be 2.5 feet.

Beam 5B10 Flexural And Shear Resistance

Beam Flexural Capacity:

Compute the flexural capacity of north-south beam 5B10.

Beam properties and reinforcing are tabulated on drawing B-666.

Beam 5B10:

$b_{NSbm} := 24 \cdot in$

$h_{NSbm} := 48 \cdot in$

Flexural Reinforcement:

Bottom Bars (Positive Moment): 7 #11 "A" Bars, no "B" Bars

Top Bars (Negative Moment) (North End): 7 #11 "C" Bars

Top Bars (Negative Moment) (South End): 7 #11 "C" Bars

$x_{sp} := 3.5 \cdot in$ Spacing of rebar layers "A" & "B" and "C"

$cover_p := 1.5 \cdot in$

$cover_n := 3.0 \cdot in$

$d_{stirrup} := 0.5 \cdot in$

$d_{11} := 1.41 \cdot in$

Compute effective depth for positive and negative moment reinforcement:

$d_{pos} := h_{NSbm} - cover_p - d_{stirrup} - (0.5) \cdot (d_{11})$

$d_{pos} = 45.295\, in$

$d_{neg\_N} := h_{NSbm} - cover_n - d_{stirrup} - (0.5) \cdot (d_{11}) - (0.5) \cdot (x_{sp})$

$d_{neg\_N} = 42.045\, in$

Compute Flexural Capacity of Beam:

Tabulate properties and constants:

$\Phi_m = 0.9$

$f_y = 60\, ksi$

$N_{pos} := 7$

$N_{neg\_N} := 7$

$N_{neg\_S} := 7$

$A_{s11} := 1.56 \cdot in^2$

$A_{s\_pos} := (N_{pos}) \cdot (A_{s11})$ $A_{s\_pos} = 10.92\, in^2$

$A_{s\_neg\_N} := (N_{neg\_N}) \cdot (A_{s11})$ $A_{s\_neg\_N} = 10.92\, in^2$

$A_{s\_neg\_S} := (N_{neg\_S}) \cdot (A_{s11})$ $A_{s\_neg\_S} = 10.92\, in^2$

Flexural Strength:

$\beta_1 = 0.815$ $f_c = 4700\,\text{psi}$

Positive Moment Flexural Capacity:

$T_{pos} := (A_{s\_pos}) \cdot (f_y)$ $T_{pos} = 655.2\,\text{kip}$

$$a_{pos} := \frac{T_{pos}}{(\beta_1) \cdot (f_c) \cdot (b_{NSbm})} \qquad a_{pos} = 7.127\,\text{in}$$

$$\Phi M_{n\_pos} := (\Phi_m)(T_{pos}) \cdot \left(d_{pos} - \frac{a_{pos}}{2}\right)$$

$\Phi M_{n\_pos} = 2050.7\,\text{ft} \cdot \text{kip}$

Negative Moment Flexural Capacity (North and South Ends):

$T_{neg\_N} := (A_{s\_neg\_N}) \cdot (f_y)$ $T_{neg\_N} = 655.2\,\text{kip}$

$$a_{neg\_N} := \frac{T_{neg\_N}}{(\beta_1) \cdot (f_c) \cdot (b_{NSbm})} \qquad a_{neg\_N} = 7.127\,\text{in}$$

$$\Phi M_{n\_neg\_N} := (\Phi_m)(T_{neg\_N}) \cdot \left(d_{neg\_N} - \frac{a_{neg\_N}}{2}\right)$$

$\Phi M_{n\_neg\_N} = 1891\,\text{ft} \cdot \text{kip}$

Compute Resistance Factors for Beam

Compute the resistance factors for the beam.

(Reference 1, Table 5.2, page 2-113).

Use formulation for a multi-span beam.

Moment Resistance Factors:

$L_{bm} := 24.5 \cdot ft$ (Total Length of Beam 5B10)

Moment Resistance Factor (North End):

$$R_{M\_N} := \frac{(4) \cdot (\Phi M_{n\_neg\_N} + \Phi M_{n\_pos})}{L_{bm}}$$

$R_{M\_N} = 643.537\,kip$

$R_m := R_{M\_N}$

$R_m = 643.5\,kip$ **vs 658.6 kip for beam 5B6** (See page 56)

Beam Shear Capacity:

Compute the shear capacity of north-south beam 5B10.

Beam properties and reinforcing are tabulated on drawing B-666.

Beam 5B10:

Shear Reinforcement:

Stirrups (# 4 bars) are placed as follows:

10 bars @ 5", 6 @ 6" Balance of bars are placed at 12" spacing.

$\Phi_S = 0.85$

$d_{stirrup} = 0.5\,in$

$A_{stirrup} := 0.20 \cdot in^2$

$A_v := (2) \cdot (A_{stirrup})$ $A_v = 0.4\,in^2$

$sp_{stir\_N} := 5 \cdot in$

$sp_{stir\_S} := 5 \cdot in$

Shear Strength:

North End:

$$\Phi V_{c\_N} := (\Phi_S)\cdot(2)\cdot\sqrt{f_c}\cdot(\sqrt{psi})(b_{NSbm})\cdot(d_{neg\_N})$$

$$\Phi V_{c\_N} = 117.604\,kip$$

$$\Phi V_{s\_N} := \frac{(A_v)\cdot(f_y)\cdot(d_{neg\_N})}{sp_{stir\_N}}$$

$$\Phi V_{s\_N} = 201.816\,kip$$

$$\Phi V_{n\_N} := \Phi V_{c\_N} + \Phi V_{s\_N}$$

$$\Phi V_{n\_N} = 319.42\,kip$$

$$\Phi V_n := \Phi V_{n\_N}$$

$$\Phi V_n = 319.42\,kip$$

Compute Shear Resistance Factors

Compute the shear resistance factor for a concentrated load applied to the beam at mid-span. For this case, the reaction at each end is one half the applied load.

$$R_v := (2)\cdot(\Phi V_n)$$

$R_v = 638.8\,kip$ **vs 568.8 kips for beam 5B6**

(See Page 60)

$$(2)\cdot(\Phi V_{n\_bm\_aver}) = 568.8\,kip$$

Summary

Since both the flexural and shear resistances for beam 5B10 are comparable to those of beam 5B6 (flexural resistance is 2% lower and shear resistance is 12% higher), the analysis for Scenario 3 is also applicable when the shield plug is orientated in the north-south direction and dropped over Beam 5B10.

Scenario 4

Scenario 4

Full drop of a shield plug on two adjacent columns (Drop Height = 2'-6")

TWO COLUMNS IMPACT (SCENARIO 4)

Introduction

Column M-47 is a 33' x 33" column that was evaluated in Scenario 2 for a drop height of 1'-0". Column M-41 is a 33' x 33" column identical to Column M-47.

Scenario 2 used a very conservative approach, where the entire mass of the shield plug was considered to be dropped on a single column. This approach is conservative because at the worst orientation of the plug over one column, the wall on Column Row L will share the effect of the drop with the column.

Scenario 4 addresses a more realistic situation.

This scenario addresses the case where the shield plug is lifted to a height of 2'-6" with the shield plug located above two columns. The total strain energy of the two adjacent columns is compared to the applied energy.

The columns to be considered in this scenario are Columns M-41 and M-40. These columns are selected because the shield plugs will need to be lifted to a height of 2'-6" in the area above these two columns.

Tabulate Material Properties:

$\gamma_{conc} = 150\,\text{pcf}$

$\gamma := \gamma_{conc}$ $\qquad\qquad$ $\gamma = 150\,\text{pcf}$

Tabulate Resistance of Column M-41 (33' x 33")

Column M-41 is a 33" x 33" column and is identical in size to Column M-47 that was addressed in Scenario 2.

From Scenario 2:

$P_{col\_control} = 3779.3\,\text{kip}$

$A1 := (33\cdot\text{in})^2$ $\qquad\qquad$ $A1 = 1089\,\text{in}^2$

$R_1 := P_{col\_control}$ $\qquad\qquad$ $R_1 = 3779.3\,\text{kip}$

Compute Properties of Column M-40 (36" x 36")

Calculate Resistance of adjacent column M-40:

$A2 := (36\cdot in)^2$ $A2 = 1296\,in^2$

Compute Combined Resistance of Two Columns:

Since the column number of bars and size are the same in both columns, the resistance in compression of the larger column can be calculated as follows:

$R_2 := R_1 + 0.85\cdot 0.7\cdot f_c\cdot (A2 - A1)$

$R_2 = 4358.2\,kip$

The combined resistance of the two columns can be calculated as follows:

$R := R_1 + R_2$

$R = 8137.4\,kip$

Applied Energy of Load Drop and Energy Absorbing Capacity of Structure

Compute Area of Concrete Slab Affected by the Shield Plug Drop:

This is the area of the slab to be considered as effective mass sitting on top of the column.

This area was previously computed in Scenario 2. From Scenario 2:

$R_{plug} = 21.417\,ft$ (Plug Radius)

The slabs around Columns M-40 and M-41 are 18" thick. Compute the area of the slab to be considered as effective mass by adding the plug radius and the slab thickness. From Scenario 2:

$t_{slab} = 18\,in$

$R_S = 22.917\,ft$ (Effective radius)

$A_{plug} = 824.941\,ft^2$

Tabulate Weight of Upper Concrete Shield Plug:

$P_{plug} = 232\,kip$

Compute Weight of Concrete Structures Above Columns M-41 and M-40:

An investigation will be made to determine the effect of a load drop of the shield plug between Columns M-41 and M-40.

The weight (mass) of the existing concrete structure under the shield plug during the rotation of the shield plug or during the diagonal move toward the laydown area of the lower shield plugs must be determined. The existing concrete structure includes the concrete columns, beams, and slabs. The effective length of the column will be taken down to the top of the next slab (at Elevation 589'-0").

The slabs around the columns are 18" thick. The east-west beams (5B10) framing into the columns are 24" wide x 48" deep (the depth includes the slab thickness). The north-south beams (5B5 and 5B6) framing into Column M-41 are 27" wide x 54" deep. The north-south beams (5B22 and 5B23) framing into Column M-40 are 36" wide x 54" deep.

Calculate Masses of columns and masses supported by the columns:

$WC1 := 22.5\cdot ft\cdot(3\cdot ft)^2\cdot\gamma$ $WC1 = 30.375\,kip$ (Column M-40)

$WC2 := 22.5\cdot ft\cdot(33\cdot in)^2\cdot\gamma$ $WC2 = 25.523\,kip$ (Column M-41)

Weight of Slab (Under Shield Plug):

$t_{slab} := 18\cdot in$

$M_{1a} := (A_{plug})\cdot(\gamma_{conc})\cdot(t_{slab})$

$M_{1a} = 185.61\,kip$

Weight of Beams (Under Shield Plug):

Determine net weight of beams framing into Columns M-41 and M-40.

This computation considers that the long axis of the shield plug (2 x plug radius) is orientated in the north-south direction.

Beams

$$W_{beams} := [\,24.5\cdot ft\cdot 2\cdot ft\cdot 2.5\cdot ft + 16\cdot ft\cdot (3\cdot ft\cdot 2.5\cdot ft + 2.25\cdot ft\cdot 2.5\cdot ft)\,]\cdot \gamma$$

$$W_{beams} = 49.875\,kip$$

Determine Energy Losses if Shield Plug is Dropped on Top of Columns M-41 and M-40:

Refer to Roark & Young (6th Edition), Chapter 15, page 718 (Reference 3).

Determine K factor. Use Case 1 (moving body of mass "M" strikes axially one end of a bar of mass M1, the other end of which is fixed, with additional mass [slab + beams] at the end of the bar).

$M2 := W_{beams} + M_{1a}$ $\quad$ $M2 = 235.487\,kip$ $\quad$ (Beams and Slabs)

$WC := WC1 + WC2$ $\quad$ $WC = 55.898\,kip$ $\quad$ (Columns)

$$K := \frac{\left[1 + \left(\frac{1}{3}\right)\cdot\left(\frac{WC}{P_{plug}}\right) + \left(\frac{M2}{P_{plug}}\right)\right]}{\left[1 + \left(\frac{1}{2}\right)\cdot\left(\frac{WC}{P_{plug}}\right) + \left(\frac{M2}{P_{plug}}\right)\right]^2}$$

$$K = 0.459$$

$$drop := 2.50\cdot ft$$

$$P_{plug} = 232\,kip$$

$$E_{final} := \left(P_{plug}\right)\cdot(drop)\cdot(K)$$

$E_{final} = 266.5\,kip\cdot ft$ $\quad$ (Impact Energy)

Maximum strain in concrete column:

$\varepsilon := 0.002$

$L_{net\_col} := 22.5 \cdot ft$

Compute deflection of column at maximum strain:

$\Delta_e := (\varepsilon) \cdot (L_{net\_col})$

$\Delta_e = 0.54\,in$

Tabulate the allowable ductility ratio of the concrete column for impulse and impact load (Reference 1, Table 5.1, page 2-112):

$\mu_{duct} := 1.3$ (Ref. 1, Table 5.1, page 2-112)
(Ref. 4, Appendix C)

Compute the energy absorbing capacity of the concrete column (by computing the area under the load-deflection curve):

$E_s := (R) \cdot \left[(\Delta_e) \cdot (0.5) + \left[(\mu_{duct}) \cdot (\Delta_e) - (\Delta_e) \right] \right]$

$E_s = 292.9\,kip \cdot ft$ (Strain Energy)

Compare the applied energy with the energy absorbing capacity of the structure.

$E_{final} = 266.492\,kip \cdot ft$ < $E_s = 292.948\,kip \cdot ft$ OK

Modify the energy absorbing capacity of the concrete column (computed above) by including the effect of the dead load carried by the column.

$P_{col\_DL} := M2$

$P_{col\_DL} = 235.487\,kip$

Compute Reduction in Energy Absorbing Capacity of Column from Dead Load:

$E_{col\_DL} := (P_{col\_DL}) \cdot (\mu_{duct}) \cdot (\Delta_e)$

$E_{col\_DL} = 13.776\,kip \cdot ft$ (Dead Load Strain Energy)

Compute Modified Energy Absorbing Capacity of Column from Dead Load:

$E_{sf} := E_s - E_{col\_DL}$

$E_{sf} = 279.172\,kip \cdot ft$ (Net Available Strain Energy)

Compute the applied energy with the energy absorbing capacity of the columns.

$E_{final} = 266.5\,kip \cdot ft$ < $E_{sf} = 279.172\,kip \cdot ft$ OK

Summary

The two concrete columns at Column Rows M-41 and M-40 are capable of withstanding a postulated load drop for the drop height tabulated below:

$drop = 2.5\,ft$

for the load tabulated below:

$P_{plug} := 232 \cdot kip$

using the DIFs tabulated below:

$DIF_c = 1$

$DIF_s = 1$

Scenario 5

Scenario 5

Full drop of a shield plug on wall at Column Row 44 (Drop Height = 2'-6")

WALL ON ROW 44

Evaluate the wall along Column Row 44. Compute the maximum axial capacity of the concrete wall at refueling floor Elevation 613'-0".

$b_{wall} := 12 \cdot in$ (Ref. 18)

$h_{wall} := 24 \cdot in$ (Ref. 18)

$A_{g\_wall} := (b_{wall}) \cdot (h_{wall})$

$A_{g\_wall} = 288\, in^2$

Longitudinal reinforcement consists of # 6 bars @ 12" spacing (Ref. 24).

$A_{s6} := 0.44 \cdot in^2$ (Ref. 5)

$d_6 := 0.75 \cdot in$ (Ref. 5)

$N_6 := 2$ (Ref. 24) (Total number of bars in 12" length of wall)

$A_{st\_wall} := (N_6) \cdot (A_{s6})$

$A_{st\_wall} = 0.88\, in^2$

Determine the controlling mode of failure for the wall. Two modes of failure will be investigated, based on the following parameters:

1. Buckling capacity of wall
2. Crushing capacity of wall based on modified ACI Code formula

Tabulate and compute the material properties of the wall:

$E_{steel} = 29000\, ksi$

$E_{conc} = 3907.7\, ksi$

$N = 7.421$

Compute the gross moment of inertia of the wall:

$$I_g := \left(\frac{1}{12}\right)\cdot(b_{wall})\cdot(h_{wall})^3$$

$$I_g = 13824\,in^4$$

Compute the cracked moment of inertia of the wall using the methodology given in Reference 1, Figure 3.1.10.

$cover_{wall} := 1.5\cdot in$ (Ref. 5) (Conservative)

$$d_{wall} := h_{wall} - cover_{wall} - (0.5)\cdot(d_6)$$

$$d_{wall} = 22.125\,in$$

$$As_{wall\_pos} := \frac{(N_6)\cdot(A_{s6})}{2}$$

$$As_{wall\_pos} = 0.44\,in^2$$

$\rho_{wall} := \dfrac{As_{wall\_pos}}{(b_{wall})\cdot(d_{wall})}$ (Ref. 1, Figure 3.1.10)

$$\rho_{wall} = 0.001657$$

$\rho n := (\rho_{wall})\cdot(N)$ (Ref. 1, Figure 3.1.10)

$$\rho n = 0.0123$$

$$As_{wall\_neg} := \frac{(N_6)\cdot(A_{s6})}{2}$$

$$As_{wall\_neg} = 0.44\,in^2$$

$\rho_{wall\_neg} := \dfrac{As_{wall\_neg}}{(b_{wall})\cdot(d_{wall})}$ (Ref. 1, Figure 3.1.10)

$$\rho_{wall\_neg} = 0.001657$$

$\rho n_{neg} := (\rho_{wall\_neg})\cdot(N)$ (Ref. 1, Figure 3.1.10)

$$\rho n_{neg} = 0.0123$$

$$\rho_{ratio} := \frac{\rho n_{neg}}{\rho n}$$ (Ref. 1, Figure 3.1.10)

$$\rho_{ratio} = 1$$

Determine the coefficient "F" from Reference 1, Figure 3.1.10 for the following values:

$\rho_{ratio} = 1$ $\rho n = 0.0123$

$F := 0.01$ (Ref. 1, Figure 3.1.10)

Compute the cracked moment of inertia for the wall using Reference 1, Figure 3.1.10:

$$I_{cr} := (F)\cdot\left[(b_{wall})\cdot(d_{wall})^3\right]$$

$$I_{cr} = 1299.7\,in^4$$

Compute the average moment of inertia for the wall using Reference 1:

$$I_a := \frac{I_g + I_{cr}}{2}$$

$$I_a = 7561.8\,in^4$$

Compute the clear length of the wall between Elevation 613'-0" and Elevation 589'-0" (reduce gross wall length by depth of thinnest slab framing into the wall at Elevation 613'-0":

$$L_{wall} := (613 - 589)\cdot ft - (1.5\cdot ft)$$

$$L_{wall} = 22.5\,ft$$

Tabulate the value of "k" for the wall (use k = 0.8):

$$k_{wall} := 0.8$$

Determine Buckling Capacity of Wall:

$$P_{crit\_wall} := \frac{(\pi)^2\cdot(E_{conc})\cdot(I_a)}{\left[(k_{wall})\cdot(L_{wall})\right]^2}$$

$$P_{crit\_wall} = 6250.9\,kip$$

Determine Crushing Capacity of Wall:

Determine the crushing capacity of the wall by modifying the ACI Code (Reference 5) formula for column compression (at minimum eccentricity). The ACI formula will be modified by replacing the 0.8 factor in the numerator with 1.0.

$$\Phi_c := 0.70 \qquad \text{(Ref. 5)}$$

$$P_{crush\_wall} := (1.00)\cdot(\Phi_c)\cdot\begin{bmatrix}(0.85)\cdot(f_c\cdot DIF_c)\cdot(A_{g\_wall} - A_{st\_wall}) \ldots \\ +(f_y\cdot DIF_s)\cdot(A_{st\_wall})\end{bmatrix} \qquad \text{(Ref. 5)}$$

$$P_{crush\_wall} = 839.9\,\text{kip}$$

Determine Controlling Wall Capacity:

$$P_{wall\_control} := \min(P_{crit\_wall}, P_{crush\_wall})$$

$$P_{wall\_control} = 839.9\,\text{kip}$$

Applied Energy of Load Drop and Energy Absorbing Capacity of Structure

Compute Area of Concrete Slab Affected by the Shield Plug Drop:

This is the area of the slab to be considered as effective mass sitting on top of the column.

This area was previously computed in Scenario 2. From Scenario 2:

$R_{plug} = 21.417\,ft$ (Plug Radius)

The slabs adjacent to Column Line 44 between Column Rows L and M are 18" and 24" thick. Compute the area of the slab to be considered as effective mass by adding the plug radius and the slab thickness (use minimum slab thickness of 18").

From Scenario 2:

$t_{slab} = 18\,in$

$R_S = 22.917\,ft$ (Effective radius)

$A_{plug} = 824.941\,ft^2$

Tabulate Weight of Upper Concrete Shield Plug ("Cookie"):

$P_{plug} := 232 \cdot kip$

Compute Weight of Concrete Structures Near Column M-47:

The concrete shield plug will be moved east from the Unit 3 side to the Unit 2 side along a path north of Column Row M. During this move, the center of gravity of the shield plug will be over the wall at Column Row 44.

An investigation will be made to determine the effect of a load drop of the shield plug over the wall at Column Row 44 . The weight (mass) of the existing concrete structure under the shield plug at Column Row 44 must be determined. The existing concrete structure includes the concrete wall and slabs. The effective length of the wall will be taken down to the top of the next slab (at Elevation 589'-0").

The slabs around Column Row 44 are 18" and 24" thick.

$\gamma_{conc} := 150 \cdot pcf$

Weight of Slab (Under Shield Plug):

$t_{slab\_min} := 18 \cdot in$

$t_{slab\_max} := 24 \cdot in$

$$M_{1aw} := (A_{plug}) \cdot (\gamma_{conc}) \cdot \left(\frac{t_{slab\_min} + t_{slab\_max}}{2} \right)$$

$M_{1aw} = 216.55\,kip$

Weight of Wall (Under Shield Plug):

Determine net weight of the wall between Elevations 589'-0" and 613'-0". Weight of wall is based on a length of wall equal to the radius of the shield plug.

$L_{eff\_wall} := R_{plug}$ $\qquad$ $L_{eff\_wall} = 21.417\,ft$

$L_{net\_wall} := (613 - 589) \cdot ft - t_{slab}$

$L_{net\_wall} = 22.5\,ft$

$M_{1cw} := (L_{eff\_wall}) \cdot (h_{wall}) \cdot (L_{net\_wall}) \cdot (\gamma_{conc})$

$M_{1cw} = 144.56\,kip$

Total Weight of Existing Concrete Structure (Under Shield Plug):

$M_{1w} := M_{1aw} + M_{1cw}$

$M_{1w} = 361.109\,kip$

Determine Energy Losses if Shield Plug is Dropped on Top of Column Row 44 Wall:

Refer to Roark & Young (6th Edition), Chapter 15, page 718 (Reference 3).

Determine K factor. Use Case 1 (moving body of mass "M" strikes axially one end of a bar of mass M1, the other end of which is fixed, with additional mass [slabs] at the end of the bar).

$$K := \frac{\left[1 + \left(\frac{1}{3}\right)\cdot\left(\frac{M_{1cw}}{P_{plug}}\right) + \left(\frac{M_{1aw}}{P_{plug}}\right)\right]}{\left[1 + \left(\frac{1}{2}\right)\cdot\left(\frac{M_{1cw}}{P_{plug}}\right) + \left(\frac{M_{1aw}}{P_{plug}}\right)\right]^2}$$

$K = 0.425$

$drop := 2.5\cdot ft$

$E_{final} := (P_{plug})\cdot(drop)\cdot(K)$

$E_{final} = 246.406\,kip\cdot ft$ (Impact Energy)

Maximum strain in concrete wall:

$\varepsilon := 0.002$

$L_{net\_wall} = 22.5\,ft$

Compute deflection of wall at maximum strain:

$\Delta_e := (\varepsilon)\cdot(L_{net\_wall})$

$\Delta_e = 0.54\,in$

Tabulate the allowable ductility ratio of the concrete wall for impulse and impact load (Reference 1, Table 5.1, page 2-112):

$\mu_{duct} := 1.3$ (Ref. 1, Table 5.1, page 2-112)
(Ref. 4, Appendix C)

Compute the energy absorbing capacity of the concrete wall (by computing the area under the load-deflection curve):

The value of $P_{wall\_control}$ must be multiplied by the effective length of the wall.

$$E_s := \left[(P_{wall\_control}) \cdot \left(\frac{L_{eff\_wall}}{ft}\right)\right] \cdot \left[(\Delta_e) \cdot (0.5) + \left[(\mu_{duct}) \cdot (\Delta_e) - (\Delta_e)\right]\right]$$

$E_s = 647.556\,kip \cdot ft$ (Strain Energy)

Compare the applied energy with the energy absorbing capacity of the structure.

$E_{final} = 246.406\,kip \cdot ft$ < $E_s = 647.556\,kip \cdot ft$ OK

Modify the energy absorbing capacity of the concrete wall (computed above) by including the effect of the dead load carried by the wall.

$P_{wall\_DL} := M_{1aw}$

$P_{wall\_DL} = 216.547\,kip$

Compute Reduction in Energy Absorbing Capacity of Wall from Dead Load:

$E_{wall\_DL} := (P_{wall\_DL}) \cdot (\mu_{duct}) \cdot (\Delta_e)$

$E_{wall\_DL} = 12.668\,kip \cdot ft$ (Dead Load Strain Energy)

Compute Modified Energy Absorbing Capacity of Wall from Dead Load:

$E_{sf} := E_s - E_{wall\_DL}$

$E_{sf} = 634.888\,kip \cdot ft$ (Net Available Strain Energy)

Compare the applied energy with the energy absorbing capacity of the structure.

$E_{final} = 246.406\,kip \cdot ft$ < $E_{sf} = 634.888\,kip \cdot ft$ OK

Summary

The concrete wall at Column Row 44 is capable of withstanding a postulated load drop for the drop height tabulated below:

$$drop = 2.5\,ft$$

for the load tabulated below:

$$P_{plug} := 232 \cdot kip$$

using the DIFs tabulated below:

$$DIF_c = 1$$

$$DIF_s = 1$$

SUMMARY AND CONCLUSIONS

This calculation determines values of maximum safe lifting height for movement of Unit 3 Reactor Shield Plugs (plugs) for storage in Unit 2 during outage of Unit 3 and the return of these plugs to Unit 3 at the end of Unit 3 outage. The lifting of the plugs takes place on Unit 3 and Unit 2 Reactor Cavities and above Reactor Building floor Elevation 613'-0".

There are three layers of plug. There are two plugs in each layer. Each plug has the shape of a semi-circular disc of thickness 2'-0". The diameter of the top layer of plugs is approximately 43' with the successive layers having smaller diameter.

Each of the top plugs weighs 116 Ton (232 kips). This exceeds the single failure proof (SFP) rating of the Reactor Building Crane, which is 110 Ton (220 kips). The load drop analyses performed in this calculation are performed to comply with Sections 5.1.4 (2) and 5.1.5 (c) of NUREG-0612 (Ref. 2), which requires a load drop analysis when the SFP requirements are not met.

The load drop analyses performed in this calculation to determine the maximum lifting heights meet the intent of the Appendix A of NUREG-0612. The portion of the appendix that is applicable to heavy load drop evaluation is Section 1 "General Considerations". This section has 10 items. The table below summarizes the applicability of the 10 items to the scope of this calculation, and when the item is applicable, the table states whether the intent of the requirement or the requirement itself is met.

NUREG-0612, Appendix A, Section 1, Items 1-10

Item No.	Applicability	Analysis
1	YES	Considered and Evaluated. Requirement Met.
2	NO	
3	YES	The RB Crane does not restrict the travel area within the designated load path via mechanical stops or electrical interlocks. However, Dresden Station will administratively control load movements to the evaluated areas (see Attachment C of this calculation). Therefore, the intent of this requirement will be met.
4	NO	
5	NO	
6	YES	Analysis is based on this requirement. Requirement Met.
7	YES	Analysis is based on this requirement. Requirement Met.
8	YES	Analysis is based on this requirement. Requirement Met.
9	NO	
10	NO	

Refer to Figures 1 through 3 (Attachment B) for the travel paths that are evaluated for each layer of plugs. These figures specify the maximum lift heights that are determined in this calculation. These evaluations consider effects of local damage and overall damage to the supporting reinforced concrete structure.

For local damage, it is shown that for the calculated maximum heights, impacted floor slabs will not suffer back face scabbing. Equations from Ref. 1 based on NDRC local damage equations are employed to obtain this conclusion. This is an expected result in view of the size of the impactor (plugs) and the slow velocity of impact corresponding to 2'-6" maximum drop height.

For overall damage, the energy balance is used to show that for the maximum heights calculated the ductility limits of the impacted structural elements as determined from Appendix C of the ACI 349 (Ref. 4) will not be exceeded. When flexure controls, the ductility limit for beams and slabs is determined to be 10. In order to conclude that flexure governs, it is verified that the shear strength exceeds the flexural strength by at least 20%. When axial compression governs, the limit of ductility is 1.3 from ACI 349.

In the overall damage evaluation, elasto-plastic load deflection diagrams developed from the component capacities from ACI 349, and stiffness calculations from Ref. 1 are used. The area under this diagram must exceed the energy of the fall reduced by the losses that take place at the instant of impact. These losses are calculated using equations from Chapter 15 of the book "Roark's Formulas for Stress and Strain" (Ref. 3).

Two factors of conservatism exist in the overall impact evaluations. These are:

- The effect of increase in yield and crushing strengths due to high strain rate effects encountered in impact loading are ignored, and
- The entire weight of the plug is assumed to drop the full amount of the drop height. If failure occurs at the lift points or slings, the center of gravity travels less due to ensuing rotation of the plug prior to impact.

In making these overall impact evaluations, due to the size of the plug more than one floor element may become engaged in the impact process. The following five load drop scenarios were considered to envelope all potential load drops of the plugs on to the Reactor Cavity and on to the floor at Elevation 613'-0". Note that the load movements are limited to the areas shown in Figures 1 through 3 (Attachment B). The scenarios considered are:

1. All cases of the drop of a shield plug on to the Reactor cavity (Units 2 and 3).
2. Full drop of a shield plug on a single column (Drop Height = 1'-0").
3. Full drop of a shield plug on a system of two adjacent slabs with a beam in between the slabs (Drop Height = 2'-6").
4. Full drop of a shield plug on two adjacent columns (Drop Height = 2'-6").
5. Full drop of a shield plug on wall at Column Row 44 (Drop Height = 2'-6").

The following notes apply to the five scenarios above.

a. Scenario 2 covers the case of a full drop of 1 foot on a wall

b. Scenario 4 covers the case of a full drop of 2'-6" on a wall and a column.

Based on above evaluations it was determined that the ductility limit of 1.3 for axial compression and 10 for flexure will not be exceeded. Therefore, maximum heights of 12" (1'-0") and 30" (2'-6") in Figures 1 through 3 are acceptable.

ATTACHMENT A

S&L Evaluation No. SL-007347

Sargent & Lundy

[X] SAFETY-RELATED [ ] NON-SAFETY-RELATED [ ] REGULATORY RELATED [ ] RELIABILITY RELATED	Attachment A Calculation No. DRE02-0064 Revision No. 0 Page A2	CONTROL NO. D-1298M PROJECT NO. 10338-021 PAGE 1 OF 8
ENGINEERING EVALUATION FORM SYSTEM CODE: DRE-1600, QC-0020		COMED DRESDEN/QUAD CITIES

PURPOSE/OBJECTIVE

This engineering evaluation provides the basis of an engineering judgment for accepting load drops associated with the removal of any one of the shield plugs in Dresden and Quad Cities stations. The objective is to consider only drops resulting from a shield plug dropping onto the lower layer shield plugs, or from the drop of the lowest layer shield plug onto the drywell head. Load drops on the operating floor are not within the scope of this evaluation. This evaluation was initiated to support the ongoing ComEd efforts to reduce the outage duration by removing the shield plugs while the unit is in coastdown.

ASSUMPTIONS

1. It is assumed that the bottom of the lifted shield plug will be no higher than 6 inches above the operating floor. This evaluation is performed by comparison to the critical parameters for the LaSalle Station. The detailed evaluation of the LaSalle load drop (Reference 3) has this 6 inch limitation, and this assumption is reasonable based on typical heavy load movement procedures. This 6" limitation shall be stated in any procedure that specifically relies on the result of this evaluation.

INTERFACING COMMENTS BY:

Division	Name of Commentor	Calc. No.	Signature of Commentor

	Signature	Position	Date
Prepared By:	M. Amin	Consultant	10-26-98
Reviewed By:	Thomas J. Behringer	Project Manager	10-26-98
Approved By:	M. Amin	Consultant	10-26-98

2. There are no unverified assumptions.

DESIGN INPUT

- For shield plugs of Dresden and Quad Cities, the geometry parameters are from Refs 1a through 1f.
- For shield plugs of Dresden and Quad Cities, F'c and Fy are from Refs. 1g and 1h.
- For Dresden and Quad Cities drywell head material, radius, and thickness information is from Refs. 2a, 2b, 2c and 2d.
- LaSalle shield plug and drywell head information is from Refs. 1a, 1b and 3.

Sargent & Lundy

[X] SAFETY-RELATED [ ] NON-SAFETY-RELATED [ ] REGULATORY RELATED [ ] RELIABILITY RELATED	CONTROL NO. D-1298M PROJECT NO. 10338-021 PAGE 3 OF 8
ENGINEERING EVALUATION FORM SYSTEM CODE: DRE-1600, QC-0020	COMED DRESDEN/QUAD CITIES

REFERENCES

1. Sargent & Lundy Drawings:

   a. LaSalle, S-768, Rev. D
   b. LaSalle, S-268, Rev. F.
   c. Quad Cities, B-252, Rev. G
   d. Quad Cities, B-234, Rev T
   e. Dresden, B-242, Rev. C
   f. Dresden, B-216, Rev. J
   g. Dresden, B-200, Rev. AF
   h. Quad Cities, B-188, Rev. F

2. CBI Drawings:

   a. Quad Cities, Drawing 7, Rev. 4 (Contract 9-6735)
   b. Quad Cities, Drawing 3, Rev. 4 (Contract 9-6735)
   c. Dresden, Drawing 7, Rev. 2 (Contract 9-4646)
   d. Dresden, Drawing 3, Rev. 0 (Contract 9-4646)

3. Calculation L-000061, "Reactor Shield Plugs-Heavy Load Assessment," Rev. 1, January 17, 1996 (LaSalle)

4. Sargent & Lundy Calculation PS-0288, Project No. 09936-002, Rev. 0, June 24, 1996

5. Sargent & Lundy Calculation LS0165, Project No. 09936-002, Rev. 0, July 2, 1996

EVALUATION

The evaluation is made by comparing the shield plug and drywell head relevant parameters for Dresden, Quad Cities and LaSalle stations. Reference 3 is a detailed evaluation of a similar load drop evaluation for LaSalle. Based on similarities to the LaSalle evaluation, certain drops for Dresden and Quad Cities can be considered acceptable. Where a key parameter varies such that acceptability by direct comparison to Reference 3 is not possible for Dresden and Quad Cities, information from References 4 and 5 is used to judge the acceptability of the particular load drop. The description of the postulated load drops and bases for acceptability follows with the aid of relevant parameters summarized in Table 1(at the end of this evaluation).

Description of Drop Scenarios

Considering the geometry of the drywell cavity and the three layers of the shield plugs, clearly, the critical type of load drop will result when one of the two third layer (lowest level) plugs drops into the cavity. This type of drop has the potential for striking the drywell head. Several scenarios are possible as follows: (1) direct vertical drop into the cavity due to a failure in the crane operation, (2) failure of the lug on the symmetry axis of the half-circular shaped plug and rotation of the plug about the horizontal axis passing through the other two symmetrically located lugs, and (3) failure of one of the symmetrically located lugs and rotation about the horizontal axis passing through the remaining lugs.

Reference 3 has considered these three scenarios for LaSalle. A brief discussion of these and comparison to Dresden and Quad Cities is provided below.

1. Vertical Drop of a Third Layer Plug Into Cavity

In this case, the dropping plug impacts the lowest level ledge in the cavity and impact is absorbed by the flexure of the plug. LaSalle calculation (Ref.3) shows that the plug flexural strength is quite adequate to support the impact load. Any scabbing of the plug is not considered to be of consequence to cause leakage and loss of containment function.

Referring to Table 1, we conclude that since Dresden and Quad Cities plugs have the same thickness, slightly heavier bottom reinforcement, same concrete and steel strength values; this drop is also acceptable for Dresden and Quad Cities.

2. Drop Due to Failure of Lug on Line of Symmetry of Plug

In this case the rotation about the horizontal axis through the two symmetrical lugs causes the falling plug to impact the lowest ledge. Based on Ref.3 discussion for LaSalle, this interruption of the motion either prevents any strike on the drywell head, or if the plug strikes the drywell head the energy will be less than the uninterrupted drop. Since case 3 below discusses the uninterrupted impact on the drywell head, Case 2 is considered bounded with Case 3.

Because Dresden and Quad Cities have similarly shaped plugs and similar lug configuration to LaSalle, the above Case 2 conclusion for LaSalle carries over to these stations, and Case 3 also bounds the Case 2 drop for these stations.

3. Drop Due to Failure of a Symmetrically Located Lug

In this case, because of the cavity and plug geometry, Ref. 3 concludes that an uninterrupted impact on the drywell head is possible. After considering the rotational motion of the plug and momentum transfer during impact, the energy required to be absorbed by the drywell head is calculated to be 540 in.kip (Ref.3) for LaSalle.

Reference 3 performs a nonlinear, large-displacement, elstoplastic analysis of the LaSalle drywell head under patch loading normal to the shell surface with the patch load applied at the likely location that plug impacts the drywell head. The computer program ADINA was used to calculate the deflection under the load and the values of maximum mid-thickness (i.e., membrane) and maximum surface strains. The area under the calculated load-deflection curve is the energy that shell absorbs as the applied load magnitude is increased. Reference 3 shows that at the maximum surface strain of 0.78%, the energy absorbed by the shell equals the energy demand of 540 in.kip. The solution is terminated at this level of deformation in the shell.

Reference 3 uses strain acceptance limits of 2% for maximum membrane strain and 6% for maximum surface strain. These values are conservative limits established based on

containment tests, as referenced and discussed in Ref.3. Based on the fact that energy demand for the LaSalle load drop can be accommodated at the maximum surface strain of 0.78%, the LaSalle load drop was considered acceptable in Ref. 3.

The energy demand is directly related to the drop height. By referring to Table 1 at the end of this evaluation, we note that relative to bottom of the third layer plugs the drywell head in Dresden and Quad Cities is lower than that at LaSalle. This effect of height difference can be estimated by increasing the LaSalle energy demand by the ratio.

$$\alpha 1 = (6.67 + 2.73) / (6.67 + 1.88) = 1.10.$$

(Note : 2.73' = 2'- 8 23/32", 1.88' = 1'-10 1/2" from Table 1, and 6.67' = three times the thickness of one layer of shield plugs plus 2" for gaps and 6" for the maximum height above the floor.)

Another amplification factor is needed to account for the slight difference in the weight of the plug in LaSalle and Dresden/ Quad Cities. By referring to Table 1, this weight factor is

$$\alpha 2 = 184.5 / 172 = 1.07$$

Consequently, the energy demand for Dresden/ Quad Cities becomes

$$E = 1.10 \times 1.07 \times 540 = 636 \text{ in.kip.}$$

Because the Ref. 3 calculation was terminated at the balance point for LaSalle, and because load-deflection curve is nonlinear, reference is made to the results of other S&L calculations for similar shells loaded similar to the LaSalle drywell head analysis. These are obtained from Refs.4 and 5. The drywell heads analyzed in these references were designed also by CBI and are for similar vintage BWRs. Reference 4 shell has a radius of 16'-2" and thickness of 1.5". Reference 5 shell has a radius of 18'-11" and thickness of 1.5". In these respects, these shells are considered comparable to the Dresden/ Quad Cities drywell heads.

The Ref. 4 analysis was carried out to the maximum surface strain of 2.7% (corresponding maximum membrane strain was 0.74%). The Ref. 5 analysis was carried out to the maximum surface strain of 2.3% (corresponding maximum membrane strain was 1.13%). For both

analyses, at the point of solution termination, it was possible to further load the model, indicating the absence of any cliff in the load-deflection curve.

The energy absorbed by the shells in Refs. 4 and 5, well exceeds 1800 in.kip. This energy is much more than the energy demand of 636 in.kip for Dresden/ Quad Cities. It is, therefore, concluded that the Case 3 load drop for Dresden/ Quad Cities is acceptable. Because of localized inelastic deformations at the location of impact, permanent deformation of the shell at the impact location in the form of a dent is expected; however, the containment function of the drywell head will not be impaired.

SUMMARY AND CONCLUSIONS

The consequence of a load drop into the drywell cavity during the removal of shield plugs has been evaluated for Dresden/ Quad Cities when the reactor is operating. By comparing the key input parameters to the parameters for LaSalle for which a similar load drop has been analyzed in detail (Ref. 3), and by comparing to the load-deformation and energy absorption capability of similar drywell heads calculated in Refs. 4 and 5, we conclude that the worst case load drop in Dresden/ Quad Cities is acceptable. Some localized denting of the drywell head is expected; however, containment function will be maintained.

The limitation of this evaluation result is that, in the vicinity of the reactor cavity where a plug may drop into the cavity, the bottom of the shield plug should, at no time, be raised higher than 6" above the operating floor. This 6" limitation shall be stated in any procedure that relies on the result of this evaluation.

This evaluation does not address any drop of the shield plugs onto the operating floor.

Sargent & Lundy

[X] SAFETY-RELATED [ ] NON-SAFETY-RELATED [ ] REGULATORY RELATED [ ] RELIABILITY RELATED	Attachment A Calculation No. DRE02-0064 Revision No. 0 Page A9 Final	CONTROL NO. D-1298M PROJECT NO. 10338-021 PAGE 8 OF 8
ENGINEERING EVALUATION FORM SYSTEM CODE: DRE-1600, QC-0020		COMED DRESDEN/QUAD CITIES

Table 1
Comparison of Shield Plug and Drywell Head Parameters in LaSalle, Dresden and Quad Cities

Item	:LaSalle	Dresden	Quad Cities
1.0 SHIELD PLUG			
1.1 Radius	19'-1 1/2"	19'-10"	19'-10"
1.2 Thickness	2'-0"	2'-0"	2'-0"
1.3 Weight	172 kips	185.4 kips	185.4 kips
1.4 Top Reinforcement	#8 @ 12" and #7 @ 12	#9 @ 12" and #9 @ 12"	#9 @ 12" and #9 @ 12"
1.5 Bottom Reinforcement	#11 @6" and #7 @ 12"	#11 @ 6" and #9 @ 12"	#11@ 6" and #9 @ 12"
1.6 F'_c	4 ksi	4 ksi	4 ksi
1.7 F_y	60 ksi	60 ksi	60 ksi
1.8 Distance from Bottom of Lowest Shield Plug to Top of Drywell Head	1'-10 1/2"	2'-8 23/32"	2'-8 23/32"
1.9 Designer	S&L	S&L	S&L
(Shield plug information from References 1a through 1h and 3.)			
2.0 DRYWELL HEAD			
2.1 Steel Type	SA-516 Gr 70	SA-212 Gr B	SA-516 Gr 70
2.2 Radius	15'-10"	17'-3 3/4"	17'-3 3/4"
2.3 Height	8'-0 3/4"	8'-10 9/32"	8'-10 9/32"
2.4 Thickness	1-3/8"	1-7/16"	1-7/16"
2.5 Designer	CBI	CBI	CBI
(Drywell Head Information from References 2a, 2b, 2c, 2d and 3)			

ATTACHMENT B

Figures 1, 2, and 3

ATTACHMENT C

Design Input for Administrative Control of Load Movements

CONSTANTINE N PETROPOULOS

09/28/02 11:12 AM

To: ADAM ALDABBAGH/Sargentlundy@SARGENTLUNDY, MOHAMMAD AMIN/Sargentlundy@SARGENTLUNDY, KURT H KOESSER/Sargentlundy@SARGENTLUNDY

cc:

Subject: Re: FW: Status of Dresden Crane LAR

----- Forwarded by CONSTANTINE N PETROPOULOS/Sargentlundy on 09/28/02 11:10 AM -----

CONSTANTINE N PETROPOULOS

09/27/02 02:33 PM

To: dean.galanis@exeloncorp.com

cc: allan.haeger@exeloncorp.com, kishin.chhablani@exeloncorp.com, timothy.loch@exeloncorp.com

Subject: Re: FW: Status of Dresden Crane LAR

Cookie load drop D3 2002.do

Dean:

The attached file responds to the question, and shows the applicability of the NUREG-0612 Appendix A items to the hravy load drop analysis.

Best Regards
Constantine

dean.galanis@exeloncorp.com

Attachment C
Calculation No. DRE02-0064
Revision No. 0 Page C2

Dresden Unit 3 Reactor Shield Plug Heavy Load Drop Evaluation
Compliance with NUREG-0612 Appendix A

The approach used in the evaluation of postulated heavy load drops for the movement of the reactor shield plugs meets the intent of Appendix A of NUREG-0612. The portion of Appendix A that is applicable to the heavy load drop evaluation is Section 1 "General Considerations". Section 1 of Appendix A has ten items that should be considered as appropriate. The table shown below discusses the applicability of each one of the ten items to the evaluation, and whether each applicable item is either fully met or its intent is being met in the analysis.

NUREG-0612, Appendix A, Section 1, Items 1-10

Item No.	Applicability	Analysis
1	YES	Considered and Evaluated. **Requirement Met.**
2	NO	
3	YES	The RB Crane does not restrict the travel area within the designated load path via mechanical stops or electrical interlocks. Per conversation with Mr. Tim Loch, the movement of the crane will be controlled administratively by reactor services based on input from engineering. **Therefore, the intent of this requirement will be met.**
4	NO	
5	NO	
6	YES	Analysis is based on this requirement. **Requirement Met.**
7	YES	Analysis is based on this requirement. **Requirement Met.**
8	YES	Analysis is based on this requirement. **Requirement Met.**
9	NO	
10	NO	

Exelon Nuclear

ATTACHMENT 2
Design Analysis Minor Revision Cover Sheet

CC-AA-309-1001
Revision 0

Analysis No.	DRE02-0064; Rev. 0A	**Last Page No.** 3
		Revision
EC/ECR No.	EC (Eval) 340053 Rev. 0	**Revision**
Title:	D2/3 Load Drop Evaluation of the Reactor Shield Plugs	

Station(s)	Dresden	**Is this Design Analysis Safeguards?**	Yes ☐ No ☒
Unit No.:	2&3	**Does this Design Analysis Contain Unverified Assumptions?**	Yes ☐ No ☒
Safety Class	Safety Related		
System Code	00	ATI/AR# None	

Description of Change

Calculation No. DRE02-0064, Rev. 0, addressed the load drop evaluation of the Reactor Shield Plugs. Though page 1 of the calculation Rev. 0, shows the calculation to be applicable for both Units 2 & 3, the conclusion (Page 96) only addresses the movement of the Unit 3 shield plugs.

This revision (minor) provides an evaluation to show that the Rev. 0 of the calculation is also applicable to the load drop of the Unit 2 Reactor Shield Plugs (Item "A"). Additionally, this minor revision also addresses the actual weights of the Unit 3 Concrete Shield Plugs (cookies) as determined after Rev. 0 of this calculation was approved (Item "B").

Note: On page 19 (Rev. 0), in the section noted "Slabs", the last slab listed should be Slab "R" (18") (Column Rows 48-49 / M-N). Column Rows L-M as shown in Rev. 0 is incorrect. This correction has no impact on the final conclusion of the load drop evaluation.

Calc.DRE02-0064_Minor Rev.Cover Sheet.doc

Disposition of Changes (include additional pages as required)

Item "A": Applicability of Rev. 0 of the Calculation for Unit 2 Shield Plug Drop.

The Reactor Shield Plugs are moved over the concrete slabs, beams and columns between column lines "L" and "N" of the refuel floor, elevation 613'-0" (Refer to Attachment "B"). The travel path for the top two layers of the Reactor Shield Plugs of Unit 2 are similar (Opposite Hand) to that evaluated for Unit 3 (Refer to Attachment B, Pages B2, B3 and B4). The bottom layer of the Unit 2 shield plugs are normally stacked on top of each other between column lines "46" to "49" and and "L" to "N".

The load path from Unit 2 to Unit 3 is basically opposite to the load path from Unit 3 to Unit 2 which was addressed in Rev. 0. Rev. 0 of this calculation evaluated the adequacy of the following concrete elements which are required for the load drop of the Unit 2 shield plugs and found them acceptable:

Columns (page 18): M-39, M-40, M-41, M-42, M-43, M-45, M-46, M-47, M-48 and M-49.
Beams (page 18): Column Rows 39-40-41-42 / L-M and M-N; Column Row 43-45 / L-M; Column Row M / 39-44 & Column Rows 46-49 /L-N.
Slabs (page 19): Column Rows 39-42 / L-N; Column Rows 42-46 / L-M; Column Rows 46-49 / L-N.
Walls (page 19): Column Row L / 39-49; Column Row 44 / Dryer-Separator Pool Wall to N.

The configuration, size and thickness of the shield plugs of Units 2 & 3 are the same (Reference drawings B-242 & B-672). The weights of the Unit 2 shield plugs are assumed to be the same as those considered for the Unit 3 plugs in the Rev. 0 analysis. The actual weights of the Unit 3 plugs were found to be less than the estimated weights (refer to Item "B" below). The five load drop scenarios considered for potential load drops of the Unit 3 plugs on the Reactor Cavity and on the floor at elevation 613'-0" (Page 98) are also applicable to the Unit 2 shield plugs since the configuration of the Unit 2 and Unit 3 refuel floors and the configuratuion of the Unit 2 and Unit 3 reactor shield plugs are similar.

Based on the above qualification of the Unit 2 concrete elements for load drops, the analysis performed in DRE02-0064 (Rev. 0) is also applicable for load drops of the Unit 2 concrete shield plugs. Refer to Attachment "D" for identification of the Unit 2 Concrete elements qualified in Rev. 0 for load drops.

Item "B": Record of the Estimated and the Actual Weights of the Unit 3 Shield Plugs:

	Estimated Weights (Calc. Rev. 0)	Actual Weights (EC 339901)
Top Layer Plugs:	116 Tons (Page 4)	114.9+0.36=115.26 Tons & 112.9+0.36=113.26 Tons
Middle Layer Plugs	112 Tons (Page 36)	105.0+0.36=105.36 Tons & 106.5+0.36=106.36 Tons
Bottom Layer Plugs:	108 Tons (Page 35)	100.0+0.36=100.36 Tons & 99.2+0.36= 99.56 Tons

Note: O.36 Tons is the weight of the rigging for the concrete shield plugs as weighed and reported by "Reactor Services Group" on 09/12/2002 (Reference email - Attachment page "D2").

The Reactor Building Crane has been designated by the NRC to be single-failure proof up to 110 tons. Hence, the lifted loads equal to and below 110 tons do not require "Load Drop Analysis". Therefore, for the Unit 3 plugs, based on the actual weights shown above, the middle layer and the bottom layer shield plugs can be lifted and moved around without any restrictions. Only the top layer shield plugs should be lifted and moved with the restrictions specified in the Rev. 0 of the calculation (Refer to Attachment B, Page B2). The Unit 2 plugs have not been accurately weighed. After their weights are determined, an evaluation will be performed to identify any load lifting/movement restrictions. Until then, the Unit 2 plugs will be lifted and moved with the same restrictions that were applicable to the Unit 3 plugs when they were moved before their actual weights were determined (refer to Attachment B).

Conclusion and Summary:
Calculation DRE02-0064 (Rev. 0) which was performed to address the "Load Drop Evaluation" of the Unit 3 Reactor Shield Plugs is also applicable for the "Load Drop Evaluation" of the Unit 2 Reactor Shield Plugs"

Additionally, based on the actual weights determined from the Unit 3 shield plugs, various restrictions described in the calculation (Rev. 0) for the movement of the middle and the bottom layers of the Unit 3 shield plugs are not required since their weights are below 110 tons, the single-failure proof capacity of the crane.

ATTACHMENT 2
Design Analysis Minor Revision Cover Sheet

CC-AA-309-1001
Revision 0

CALC. DRE02-0064 REV. 0A

Preparer	Kishin Chhablani	Kishin Chhablani 1/13/2003
	Print Name	Sign Name
Reviewer	Robert A. Koncel	Robert A Koncel 1/13/03
	Print Name	Sign Name
Method of Review	☒ Detailed Review	☐ Alternate Calculations ☐ Testing
~~Review Notes:~~		
Approver	Tim Loch	T Loch 1-16-03
	Print Name	Sign Name
(For External Analyses Only) Exelon Reviewer	N/A	
	Print Name	Sign Name
Approver	N/A	
	Print Name	Sign Name

Attachment : D
Calc. DRE02-0064
REV.0A Page D2 of D2
(FINAL)

Chhablani, Kishin

From: Purdy, Kenneth M.
Sent: Wednesday, November 20, 2002 10:00 AM
To: Chhablani, Kishin
Subject: RE: Weight of Strongback & Rigging for Shield Blocks

Kishin
The weighing of the Reactor Head strongback was performed as a pre outage activity there was no WO number

-----Original Message-----
From: Chhablani, Kishin
Sent: Wednesday, November 20, 2002 9:15 AM
To: Purdy, Kenneth M.
Cc: Loch, Timothy L.; Speroff, Randy D.
Subject: FW: Weight of Strongback & Rigging for Shield Blocks

Ken,

I sent a email earlier about back-up information regarding Unit 2 Vessel Head Weight. Please also verify and inform me the WO numbers for weight of the Strongback. Thanks,

Kishin

-----Original Message-----
From: Chhablani, Kishin
Sent: Tuesday, September 10, 2002 11:09 AM
To: Haeger, Allan R.
Subject: FW. Weight of Strongback & Rigging for Shield Blocks

FOR YOUR INFORMATION
KISHIN

-----Original Message-----
From: Chhablani, Kishin
Sent: Tuesday, September 10, 2002 9:19 AM
To: Speroff, Randy D ; Reda, Joseph S.
Subject: FW. Weight of Strongback & Rigging for Shield Blocks

If the following information is not correct please inform me. Thanks,

Kishin

-----Original Message-----
From: Chhablani, Kishin
Sent: Monday, September 09, 2002 3 32 PM
To: Chhablani, Kishin
Subject: Weight of Strongback & Rigging for Shield Blocks

THIS IS FOR RECORD PURPOSES:

JOE REDA GOT A CALL FROM RANDY SPEROFF TODAY AT 15:25 HOURS GIVING HIM FOLLOWING ACTUAL WEIGHTS OF THE STRONGBACK AND THE RIGGING FOR SHIELD BLOCKS:

STRONGBACK: 9400 LBS.

RIGGING FOR SHIELD BLOCKS: 710 LBS.

KISHIN CHHABLANI
09/09/2002