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Exelon Nuclear

RS-03-115

June 12, 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: Additional Information Regarding the Request for License Amendment Related to Heavy Loads Handling

References: (1) Letter from K. R. Jury (Exelon Generation Company, LLC) to U. S. NRC, "Request for License Amendment Related to Heavy Loads Handling," dated February 26, 2003

(2) Letter from U. S. NRC to J. L. Skolds (Exelon Generation Company, LLC), "Dresden Nuclear Power Station, Units 2 and 3 – Request for Additional Information Regarding Heavy Loads Handling Amendment Request," dated May 23, 2003

In Reference 1, Exelon Generation Company (EGC), LLC, requested 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 to include a description of a load drop analysis performed for handling reactor cavity shield plugs weighing greater than 110 tons with the Unit 2/3 reactor building crane during power operation.

In Reference 2, the NRC requested additional information regarding this proposed change. Attachment 1 to this letter provides the requested information.

In addition, Attachment 2 to this letter provides information discussed between Mr. L. W. Rossbach and other members of the NRC and Mr. A. R. Haeger and other members of EGC in a teleconference on June 10, 2003, regarding the planned initial movement of the top layer of reactor shield plugs.

Attachment 3 to this letter provides various references discussed in Attachments 1 and 2.

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

A001

I declare under penalty of perjury that the foregoing is true and correct. Executed on the 12th day of June, 2003.

Respectfully,

Patrick R. Simpson

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

Attachment 1: Responses to NRC Questions
Attachment 2: Description of Planned Initial Movement of Reactor Shield Plug Top Layer
Attachment 3: Copies of Selected Reference Materials

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

Attachment 1
Additional Information Regarding Request for License Amendment Related to Heavy Loads Handling

Responses to NRC Questions

Question 1
The load drop of a shield plug during lifting of the plug from the cavity in Unit 3 and laying it down on top of Unit 2 shield plugs is analyzed as Scenario 1 of the load drop analysis (Attachment 3 of the amendment request). The calculations for determining the bearing capacity of the top shield plug on page 29 of 95, utilize a strength reduction factor of 0.7 for the bearing strength of concrete and a 2/3 factor to account for the spaces between the bearing bars. Provide the basis and applicable reference material for the use of this strength reduction factor and 2/3 factor.

Response to Question 1

Strength Reduction Factor
Reference 1, in Section 9.3.1, "Design Strength," states that the design strength provided by a member shall be taken as the nominal strength multiplied by the strength reduction factor (Ø) in Sections 9.3.2 and 9.3.4.

Reference 1, in Section 9.3.2.4 specifies the value of the strength reduction factor (Ø) in "bearing on concrete" as 0.70. This value is used on page 29 of Reference 2 when calculating the bearing capacity of the top shield plug, which is a normal weight reinforced concrete.

2/3 Factor
As described on page 29 of Reference 2, the 2/3 factor is to account for the spaces between the bearing bars. It is also mentioned on page 29 that the bearing bars are 1" x 24" and the space between the bars is 12". This assumed spacing is conservative compared to the spacing of 1" between the bars shown in Section 12-12 on the attached drawings B-242 (Unit 2) and B-672 (Unit 3).

With 24" long bars located with 12" space between them, the center-to-center distance between the bars will be 12 + ½ (24 + 24) = 36".

With the 36" center-to-center distance between the bars, the length of bars available for bearing is ½ (24 + 24) = 24".

Hence, the ratio of bearing length to center-to-center length is 24/36 or 2/3.

Therefore, as the bearing bars are not continuous (for the full length) and the calculation considers a space of 12" between them, on page 29 the factor of 2/3 is used to arrive at the reduced bearing strength of the concrete of the top plug (half circle). This is a conservative factor, given that the spacing between the bars shown on the drawings is 1".

Question 2
Provide justification for not taking the concrete reinforcement into consideration in the determination of the weight of the slab, beams and column in the calculations related to the "Full drop of a shield plug on single column" which is Scenario 2 of the load drop analysis.

Attachment 1
Additional Information Regarding Request for License Amendment Related to Heavy Loads Handling

Responses to NRC Questions

Response to Question 2
The unit weight of concrete, considered as 150 pounds per cubic foot (pcf) on page 47 of Reference 2, is the unit weight normally taken for the design of reinforced concrete members.

Reference 3 on page 7 states, "Natural stone aggregates conforming to ASTM C33 are used in the majority of concrete construction, giving a unit weight for such concrete of about 145 pcf (pounds per cubic foot) or 2320 kg/m^3 (kilograms per cubic meter). When steel reinforcement is added, the unit weight of normal weight reinforced concrete is taken for calculation purposes as 150 pcf, or 2400 kg/m^3. Actual weights for concrete and steel are rarely, if ever, computed separately."

Reference 4 on page 6-9 provides the weights of building materials. For plain concrete 1" thick, the weight is shown as 12 pounds per square foot. This value, when converted to one-foot thickness, corresponds to a unit weight of plain concrete of 144 pcf. The weight of 1" thick reinforced concrete is given as 12.5 pounds per square foot. When converted to one-foot thickness, the unit weight of the reinforced concrete will be 150 pcf.

The two references quoted above provide the justification for considering the unit weight of the reinforced concrete as 150 pcf.

Question 3
The full drop of a shield plug on a system of two adjacent slabs with a beam in between the slabs, is analyzed as Scenario 3 of the load drop analysis. In this calculation the combined beam and slab moment resistance factor, as discussed on page 68 of 95, is obtained on the basis of an equation provided on this page. Provide an explanation of how this equation reduces the beam moment resistance that accounts for the area of the two adjacent slabs as indicated.

Response to Question 3
Beam 5B6 flexural resistance is calculated on pages 52-56 of Reference 2 (see attached drawing B-208). This beam supports two identical slabs, one on each side. The slab flexural resistance is calculated on pages 62-66. The slab flexural resistance is computed on the basis that beams exist on all the four sides to resist the reactions of the loaded slab. For each of the two slabs, the beams are available to resist the reactions from three sides. The fourth side of each of the two slabs is resisted by beam 5B6. Therefore, this part of the slab resistance must be subtracted from the beam total resistance. This part amounts to one-fourth of the resistance of each slab.

The above load resistance system can be expressed as follows:

$$\left\{\begin{matrix}\text{Two Slabs}\\ \text{Resistance}\end{matrix}\right\} + \left\{\begin{matrix}\text{Beam 5B6}\\ \text{Resistance}\end{matrix}\right\} - \left\{\begin{matrix}\text{One Fourth of Each of the Two}\\ \text{Slabs Supported by Beam 5B6}\end{matrix}\right\}$$

The above expression represents the equation on page 68 of Reference 2 that is used to compute the combined resistance of the two slabs and beam 5B6 located between them.

Question 4
The full drop of a shield plug on wall at column 44 is analyzed as Scenario 5 of the load drop analysis. Provide a definition of the factor "Kwall = 0.8" used on page 89 of 95 of this calculation and explain how it is obtained.

Response to Question 4
Reference 4 on page 5-134 states that "K" is a factor to equate the strength of a framed compression element of length "L" to an equivalent pin-ended member of length (also called effective length) "KL" subject to axial load only. Reference 3 on page 543 provides the equivalent pin-end lengths for various rotational and translation end restraint conditions.

Reference 4 on page 5-135 also provides Table C-C2.1, giving theoretical and recommended "K" values for various rotational and translation end restraint conditions.

Based on the sizes of the end restraint members for the wall at Column Row 44 (Reference 2, pages 87-90), and the fact that the wall ends at plant elevation 613'-0" (see attached drawing B-208) and is continuous at elevation 589'-0" (see attached drawing B-226), the wall was considered to have "one end unrestrained and the other end restrained."

For these end conditions, value of "K" on page 543 of Reference 3 is 0.7. This same value 0.7 is also shown as the theoretical "K" value in Reference 4 on page 5-135. However, this same table recommends using design value of 0.80 for "K" for these end conditions when ideal conditions are approximated. Accordingly, Reference 2 on page 89, conservatively considers the value of "Kwall" as 0.8.

Question 5
Provide a drawing or illustration of the maximum height that a top shield plug can drop and explain why this height is the maximum possible drop height.

Response to Question 5
Attached drawing B-216 (Section C-C) shows the three layers of the shield plugs located over the reactor cavity before the top shield plug is lifted.

Pages 28 thru 37 of Reference 2 provide the analysis for lifting of the shield plugs (including top layer plugs) above the reactor cavity, for a height of up to 1'-0" above the floor at elevation 613'-0". Therefore, the load drop of the top layer shield plugs, from a maximum height of 1'-0" above the floor level at the reactor cavity, has been analyzed.

Beyond the reactor cavity area, the top layer shield plugs will move along the load path identified on page B2 (attachment B) of Reference 2. The maximum height that the top layer shield plugs will be lifted above the floor level as noted on attachment page B2 is

1'-0". Pages 38 thru 85 of Reference 2 provide the load drop analysis for this height, for the various scenarios, as the top layer plug moves along the load path. Pages 96-99 summarize the results of the analysis of all plugs including the top layer shield plugs.

Pages 14-15 describe the movement of the Unit 3 top layer shield plugs and the maximum lift height mentioned is 1'-0". The movement of the Unit 2 top layer shield plugs will be in the opposite direction from the Unit 3 plugs.

Based on the travel path and lift height of the shield plugs explained above, the maximum lift height of the top layer shield plug will be 1'-0" above the floor at elevation 613'-0".

Question 6
Are the 1997 Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-97) and the 1999 Building Code Requirements for Structural Concrete (ACI 318-99) referenced in calculation DRE02-0064 attached to your submittal the current versions of these codes? If there are later versions, why weren't they used and how would they affect the calculation?

Response to Question 6
The 1999 edition of ACI 318 was subsequently revised in 2002 (i.e., ACI 318-02). The 1997 edition of ACI 349 was subsequently revised in 2001 (i.e., ACI 349-01).

DNPS has followed an internal controlled guidance document for DNPS and Quad Cities Nuclear Power Station. Topical Design Basis Document TDBD-DQ-1, "Structural Design Criteria," Section 3.3, "Codes and Standards," provides guidelines regarding this question.

ACI 318-99
Section 3.3.4 of TDBD-DQ-1 relates to ACI 318, the basic concrete design code. It states, "The latest issue of the ACI shall be used for new design of the Dresden and Quad Cities Stations."

Reference 2 does not design a new structure. It evaluates the existing concrete structure for various load drop conditions. Hence, internal guidance would not require use of the latest code.

In 1990, Sargent and Lundy had performed a generic code reconciliation to compare a previous version of the ACI code (i.e., 318-89) to the original code of construction for DNPS, ACI 318-63. This reconciliation recommended use of ACI 319-89 for new design and for reassessment as long as all pertinent structural safety provisions contained in the latest code revisions are met. ACI 318-99 was used as the code for calculation DRE02-0064, since ACI 318-99 has a similar technical basis as ACI 318-89. Therefore, it is acceptable to use ACI 318-99 for the reassessment of the existing concrete structure for load drop analysis.

It is also expected that if the ACI 318-02 code were used in the analysis, the end results of the calculation would not change. While the 2002 edition of the code did change the strength reduction factors (Ø) slightly, this would not affect the conclusion of the calculation.

ACI 349-97

Reference 2 used ACI 349-97, Appendix C to provide guidelines for "Impulsive and Impactive Effects" as a technical aid to ascertain ductility limits. Page 7 of the calculation states, "The ductility limits are determined from the Table 5.1 of Reference 1 and Appendix C of Reference 4." Reference 4 as shown on page 10 is ACI 349-97.

A general review of the relevant sections C.2, "Dynamic strength increase," and C.3, "Deformation," of both ACI 349-97 and ACI 349-01 shows no change in the corresponding technical parameters.

Based on above, the use of codes ACI 318-99 and ACI 349-97 is appropriate. The use of the latest codes is not required for reassessment of designs. If the latest codes are considered, the conclusions of the analysis, "Load Drop Evaluation of the Reactor Shield Plugs," (Reference 2) will not change.

References

1. "Building Code Requirements for Structural Concrete (ACI 318-99) and Commentary (318R-99)," 1999 Edition, ACI International

2. Calculation DRE 02-0064 (Revisions 0 and 0A), Attachment 3 to letter from K. R. Jury (Exelon Generation Company, LLC) to U. S. NRC, "Request for License Amendment Related to Heavy Loads Handling," dated February 26, 2003

3. "Reinforced Concrete Design," (text book) Fourth Edition, by Wang and Salmon

4. "Manual of Steel Construction," Ninth Edition, American Institute of Steel Construction

Attachment 2
Additional Information Regarding Request for License Amendment Related to Heavy Loads Handling

Description of Planned Initial Movement of Reactor Shield Plugs Top Layer

Background

The concrete shield plugs (plugs) over the reactor cavities of Unit 2 and Unit 3 are stacked in three layers as shown in the attached drawings B-242 (Unit 2) and B-672 (Unit 3). The plugs are semi-circular in shape with the uppermost plug having a diameter equal to 43'-0".

Currently, the top layer plugs are oriented with the diameter in the north-south direction as shown on the above referenced drawings. Page B2 (attachment B) of the calculation DRE 02-0064 in Reference 2 provides the analyzed load path for the movement of the top layer plugs.

Currently, when the Unit 2 top layer plugs are moved during reactor disassembly, they are lifted and rotated 90 degrees while being moved south and towards the center between columns 40 and 42. The plugs are rotated in order to have the diameter in the east-west direction. The plugs are then moved west between columns L and M until the plugs are located between columns 46 and 48. The plugs are then moved north and placed on the top of the Unit 3 plugs. When the plugs are to be reinstalled on Unit 2, the above steps are reversed.

The Unit 3 top layer plugs require same steps as the Unit 2 plugs, except that the plugs move from west to east.

A detailed review of the steps required to move the plugs was performed to design interlocks for the movement of the top layer plugs to ensure that the movement remains within the analyzed area, as discussed in Reference 2. In order to stay within the analyzed area between columns L and M, the plugs must be rotated to the east-west orientation from their installed north-south orientation. Further, in order to properly rig the plugs from their installed north-south orientation, the crane hook must be moved east or west of the center of the analyzed area between columns 40 and 42 for Unit 2 (columns 46 and 48 for Unit 3). It was noted that if all the top layer plugs were oriented in the east-west direction in their installed position, the plugs could be moved from the top of one unit reactor cavity to the top of the other unit reactor cavity without being rotated during the movement, and the crane hook could remain near the center of the analyzed area for all portions of the movement.

Changing the installed orientation of the top layer plugs to the east-west direction is also required in order to allow for a practical design for the interlocks, since the interlocks can then be designed to restrict crane movement to the center of the analyzed load path, given that rotation of the plugs will no longer be needed. If the interlocks allowed movement significantly beyond the center of the analyzed path, then rotation of the plugs could result in a portion of the plugs being outside the analyzed path. EGC has decided to design the interlocks with this in mind and revise the installed orientation of the plugs. This change will be performed under the engineering change process and the relevant drawings will be revised.

Attachment 2
Additional Information Regarding Request for License Amendment Related to Heavy Loads Handling

Description of Planned Initial Movement of Reactor Shield Plugs Top Layer

Effect on Commitments in Original License Amendment Request

In Reference 1, EGC stated that the electrical interlocks or mechanical stops would be activated whenever the top layer plugs weighing over 110 tons are moved. To accomplish the objectives discussed in the above section, on a one-time basis during the first time the top layer plugs of Units 2 and 3 are moved following approval of the Reference 1 request, it will be necessary to de-activate the interlocks for a short time until the plugs are lifted and brought within the path defined by the interlocks. During this one-time initial movement, the following will be performed.

- The plug will be lifted to a maximum height of 1 foot above the floor, as assumed in the load drop analysis.
- The plug will be moved towards the center of the intended travel path (i.e., center of the analyzed area between columns 40 and 42 for Unit 2 and between columns 46 and 48 for Unit 3) before being moved south or being rotated.
- The interlocks will be activated.
- The plug will be moved south.
- The plug will be rotated 90 degrees to an east-west orientation while being moved south. Given space limitations, it may not be possible to avoid rotating the plug until it has been moved far enough south to completely clear the reactor cavity. However, rotation will not begin until the plugs have been moved as far south as possible while remaining north of column M and the plug will always remain over the analyzed area.

EGC will perform these moves under administrative controls to ensure that the plugs remain within the analyzed load path shown in page B2 (attachment B) and that the lift height of the plugs is limited to a maximum of 1'-0", as assumed in the load drop analysis. These controls will be specified in the maintenance work package for the lift. The work package will specify the specific steps for the movement, as described above. The work package will also specify that a spotter be present with a drawing showing the analyzed area. The spotter will be instructed to ensure that the plugs remain within the analyzed area while the interlocks are deactivated.

Reference

1. Letter from K. R. Jury (Exelon Generation Company, LLC) to U. S. NRC, "Request for License Amendment Related to Heavy Loads Handling," dated February 26, 2003

Attachment 3
Additional Information Regarding Request for License Amendment Related to Heavy Loads Handling

Copies of Selected Reference Materials

References Supplied
ACI 318-99, Sections 9.3 and 9.3.2.4
"Reinforced Concrete Design," pages 7, 543
"Manual of Steel Construction," pages 5-134, 5-135, 6-9
Drawings B-208, B-216, B-226, B-242, B-672

ATTACHMENT - 3

REFERENCE NO. 1

ACI 318-99
ACI 318R-99

Building Code Requirements for Structural Concrete (318-99) and Commentary (318R-99)

Reported by ACI Committee 318

aci international®

american concrete institute
P.O. BOX 9094
FARMINGTON HILLS, MICHIGAN 48333-9094

CODE

9.2.8 — Load factors — For post-tensioned anchorage zone design a load factor of 1.2 shall be applied to the maximum tendon jacking force.

9.3 — Design strength

9.3.1 — Design strength provided by a member, its connections to other members, and its cross sections, in terms of flexure, axial load, shear, and torsion, shall be taken as the nominal strength calculated in accordance with requirements and assumptions of this code, multiplied by the strength reduction factors ϕ in 9.3.2 and 9.3.4.

9.3.1.1 — If the structural framing includes primary members of other materials proportioned to satisfy the load factor combinations in Section 2.3 of ASCE 7, it shall be permitted to proportion the concrete members using the set of strength reduction factors ϕ listed in Appendix C and the load factor combinations in ASCE 7.

9.3.2 — Strength reduction factor ϕ shall be as follows:

9.3.2.1 — Flexure, without axial load..................0.90

9.3.2.2 — Axial load, and axial load with flexure. (For axial load with flexure, both axial load and moment nominal strength shall be multiplied by appropriate single value of ϕ)

(a) Axial tension, and axial tension with flexure..0.90

(b) Axial compression, and axial compression with flexure:
Members with spiral reinforcement conforming to 10.9.3 ....0.75
Other reinforced members 0.70

except that for low values of axial compression ϕ shall

COMMENTARY

R9.2.8 — The load factor of 1.2 applied to the maximum tendon jacking force results in a design load of about 113% of the specified tendon yield strength but not more than 96% of the nominal ultimate strength of the tendon. This compares well with the maximum attainable jacking force, which is limited by the anchor efficiency factor.

R9.3 — Design strength

R9.3.1 — The term design strength of a member, refers to the nominal strength calculated in accordance with the requirements stipulated in this code multiplied by a strength reduction factor ϕ, which is always less than one.

The purposes of the strength reduction factor ϕ are (1) to allow for the probability of understrength members due to variations in material strengths and dimensions, (2) to allow for inaccuracies in the design equations, (3) to reflect the degree of ductility and required reliability of the member under the load effects being considered, and (4) to reflect the importance of the member in the structure.[9.2,9.3] For example, a lower ϕ is used for columns than for beams because columns generally have less ductility, are more sensitive to variations in concrete strength, and generally support larger loaded areas than beams. Furthermore, spiral columns are assigned a higher ϕ than tied columns since they have greater ductility or toughness.

R9.3.1.1 — Appendix C has been included to facilitate computations for buildings with substantial portions of their structural framing provided by elements other than concrete. If the strength reduction factors in Appendix C are used for the concrete elements, the required strengths are to be determined using the load factor combinations in Section 2.3 of ASCE 7.

R9.3.2.1 — In applying 9.3.2.1 and 9.3.2.2, the axial tensions and compressions to be considered are those caused by external forces. Effects of prestressing forces are not included.

R9.3.2.2 — For members subjected to axial load with flexure, design strengths are determined by multiplying both P_n and M_n by the appropriate single value of ϕ. For members subjected to flexure and relatively small axial compression loads, failure is initiated by yielding of the tension reinforcement and takes place in an increasingly more ductile manner as the ratio of axial load to moment decreases. At the same time the variability of the strength also decreases. For small axial loads, the value of ϕ may be increased from that for compression members to 0.90 permitted for flexure as the design axial load strength ϕP_n decreases from a specified value to zero.

For members meeting the limitations specified for $(h-d'-d_s)/h$ and f_y, the transition starts at a design axial

CODE

be permitted to be increased in accordance with the following:

For members in which f_y does not exceed 60,000 psi, with symmetric reinforcement, and with $(h-d'-d_s)/h$ not less than 0.70, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10 f_c' A_g$ to zero.

For other reinforced members, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10 f_c' A_g$ or ϕP_b, whichever is smaller, to zero.

9.3.2.3 — Shear and torsion 0.85

9.3.2.4 — Bearing on concrete (except for post-tensioning anchorage zones) 0.70

9.3.2.5 — Post-tensioned anchorage zones 0.85

9.3.3 — Development lengths specified in Chapter 12 do not require a ϕ-factor.

9.3.4 — In structures that rely on special moment resisting frames or special reinforced concrete structural walls to resist earthquake effects, the strength reduction factors ϕ shall be modified as follows:

(a) The strength reduction factor for shear shall be 0.60 for any structural member that is designed to resist earthquake effects if its nominal shear strength is less than the shear corresponding to the development of the nominal flexural strength of the member. The nominal flexural strength shall be determined considering the most critical factored axial loads and including earthquake effects;

(b) The strength reduction factor for shear in diaphragms shall not exceed the minimum strength reduction factor for shear used for the vertical components of the primary lateral-force-resisting system;

(c) The strength reduction factor for shear in joints and diagonally reinforced coupling beams shall be 0.85.

COMMENTARY

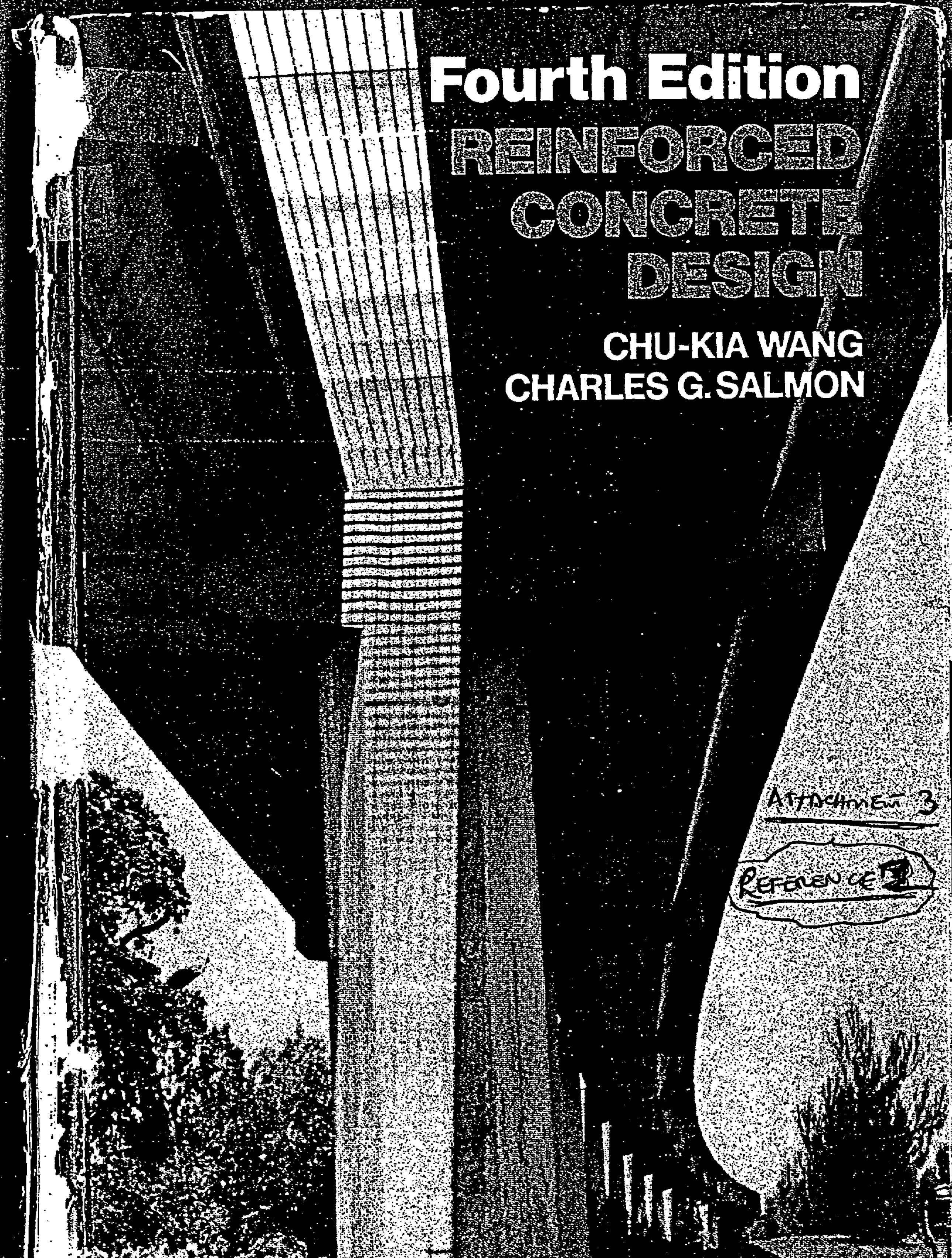
load strength, ϕP_n of $0.10 f_c' A_g$. For other conditions, P_b should be calculated to determine the upper value of design axial load strength ϕP_n (the smaller of $0.10 f_c' A_g$ and ϕP_b) below which an increase in ϕ can be made.

R9.3.2.5 — The ϕ-factor of 0.85 reflects the wide scatter of results of experimental anchorage zone studies. Since 18.13.4.2 limits the nominal compressive strength of unconfined concrete in the general zone to $0.7\lambda f_{ci}'$, the effective design strength for unconfined concrete is $0.85 \times 0.7\lambda f_{ci}' \approx 0.6\lambda f_{ci}'$.

R9.3.4 — Strength reduction factors in 9.3.4 are intended to compensate for uncertainties in estimation of strength of structural members in buildings. They are based primarily on experience with constant or steadily increasing applied load. For construction in regions of high seismic risk, some of the strength reduction factors have been modified in 9.3.4 to account for the effects of displacement reversals into the nonlinear range of response on strength.

Section 9.3.4(a) refers to brittle members such as low-rise walls, portions of walls between openings, or diaphragms that are impractical to reinforce to raise their nominal shear strength above nominal flexural strength for the pertinent loading conditions.

Short structural walls were the primary vertical elements of the lateral-force-resisting system in many of the parking structures that sustained damage during the 1994 Northridge earthquake. Section 9.3.4(b) requires the shear strength reduction factor for diaphragms to be 0.60 if the shear strength reduction factor for the walls is 0.60.


Fourth Edition
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CHU-KIA WANG
CHARLES G. SALMON
Attachment 3
Reference

added to the first three types of cement in ASTM C150 or to the blended hydraulic cements in ASTM C595 at the time the concrete is mixed.

Portland blast-furnace-slag cement has lower heat of hydration than ordinary Type I cement and is useful for mass concrete structures such as dams; and because of its high sulfate resistance, it is used in seawater construction. *Portland-pozzolan cement* is a blended mixture of ordinary Type I cement with pozzolan. *Pozzolan* is a finely divided siliceous or siliceous and aluminous material which possesses little or no inherent cementitious property, but in the powdery form and in the presence of moisture, will chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. Blended cements with pozzolan gain strength more slowly than cements without pozzolan, hence they produce less heat during hydration, and thus are widely used in mass concrete construction.

1.5 AGGREGATES

Since aggregate usually occupies about 75% of the total volume of concrete, its properties have a definite influence on the behavior of hardened concrete. Not only does the strength of the aggregate affect the strength of the concrete, its properties also greatly affect durability (resistance to deterioration under freeze-thaw cycles). Since aggregate is less expensive than cement, it is logical to try to use the largest percentage feasible. In general, for maximum strength, durability, and best economy, the aggregate should be packed and cemented as densely as possible. Hence aggregates are usually graded by size and a proper mix has specified percentages of both *fine* and *coarse* aggregates.

Fine aggregate (sand) is any material passing through a No. 4 sieve† [i.e., less than about $\frac{3}{16}$ in. (5 mm) diameter]. Coarse aggregate (gravel) is any material of larger size. The nominal maximum size of coarse aggregate permitted (ACI-3.3.3)‡ is governed by the clearances between sides of forms and between adjacent bars and may not exceed "(a) $\frac{1}{5}$ the narrowest dimension between sides of forms, nor (b) $\frac{1}{3}$ the depth of slabs, nor (c) $\frac{3}{4}$ the minimum clear spacing between individual reinforcing bars. . . ." Additional information concerning aggregate selection and use is to be found in a report of ACI Committee 621 [10].

Natural stone aggregates conforming to ASTM C33 [11] are used in the majority of concrete construction, giving a unit weight for such concrete of about 145 pcf (pounds per cubic foot) or 2320 kg/m^3 (kilograms per cubic meter). When steel reinforcement is added, the unit weight of *normal weight* reinforced concrete is taken for calculation purposes as 150 pcf, or 2400 kg/m^3. Actual weights for concrete and steel are rarely, if ever, computed separately. For special purposes, lightweight or extra heavy aggregates are used.

†4.75 mm according to ASTM Standard E11.

‡Numbers refer to sections in the "ACI Code," officially ACI 318–83, *Building Code Requirements for Reinforced Concrete* [9].

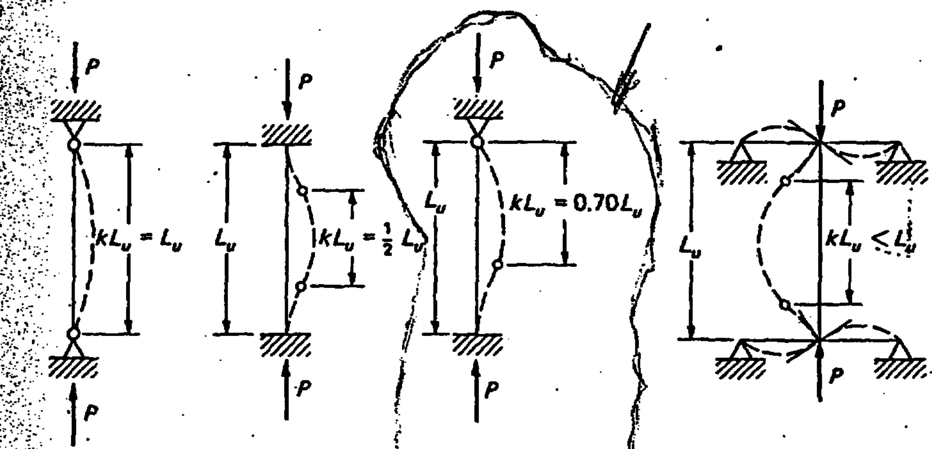
$$(kL_u)^2 = \frac{\pi^2(1008)(0.200bh^3)}{3.92bh} = 507h^2$$

$$\frac{kL_u}{h} = 22.5$$

$$\frac{kL_u}{r} = 22.5\sqrt{12} = 78 \qquad \text{(abscissa of point } C\text{)}$$

15.3 EQUIVALENT PIN-END LENGTHS

For conditions other than pin ends where the factor k in Eq. (15.2.1) is 1.0, the equivalent pin-end length (also called *effective* length) factor k must be determined for various rotational and translational end restraint conditions. Where translation at both ends is adequately prevented, the distance between points of inflection is shown in Fig. 15.3.1. For all such cases the equivalent pin-end length is less than the actual unbraced length (i.e., k is less than one).



(a) End rotations unrestrained (b) End rotations fully restrained (c) One end restrained, other unrestrained (d) Partially restrained at each end

Figure 15.3.1 Equivalent pin-end (i.e., effective) lengths; no joint translation.

If sidesway or joint translation is possible, as in the case of the unbraced frame, the equivalent pin-end length exceeds the actual unbraced length (i.e., k is greater than one), as shown in Fig. 15.3.2.

As reinforced concrete columns are in general part of a larger frame, it is necessary to understand the concepts of a *braced frame* (where joint translation is prevented by rigid bracing, shear walls, or attachment to an adjoining structure) and the *unbraced frame* (where buckling stability is dependent on the stiffness of the beams and columns that constitute the frame). As shown in Figs. 15.3.3(a) and (c), the effective length kL_u for cases where joint translation is prevented may never exceed the actual length L_u.

ATTACHMENT - 3

REFERENCE-4

Manual of

STEEL CONSTRUCTION

Allowable Stress Design

SIGN

Ninth Edition

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CHAPTER C

FRAMES AND OTHER STRUCTURES

C2. FRAME STABILITY

The stability of structures as a whole must be considered from the standpoint of the structure, including not only the columns, but also the beams, bracing system and connections. The stability of individual elements must also be provided. Considerable attention has been given in the technical literature to this subject, and various methods of analysis are available to assure stability. The SSRC *Guide to Design Criteria for Metal Compression Members* devotes several chapters to the stability of different types of members considered as individual elements, and then considers the effects of individual elements on the stability of the structure as a whole (Galambos, 1988).

The effective length concept is one method for estimating the interaction effects of the total frame on a column being considered. This concept uses K-factors to equate the strength of a framed compression element of length L to an equivalent pin-ended member of length KL subject to axial load only. Other methods are available for evaluating the stability of frames subject to gravity and lateral loading and individual compression members subject to axial load and moments. The effective length concept is one tool available for handling several cases which occur in practically all structures, and it is an essential part of many analysis procedures. Although the concept is completely valid for ideal structures, its practical implementation involves several assumptions of idealized conditions which will be mentioned later.

Two conditions, opposite in their effect upon column strength under axial loading, must be considered. If enough axial load is applied to the columns in an unbraced frame dependent entirely on its own bending stiffness for resistance to lateral deflection of the tops of the columns with respect to their bases (see Fig. C-C2.1), the effective length of these columns will exceed the actual length. On the other hand, if the same frame were braced to resist such lateral movement, the effective length would be less than the actual length, due to the restraint (resistance to joint rotation) provided by the bracing of other lateral support. The ratio K, effective column length to actual unbraced length, may be greater or less than 1.0.

The theoretical K-values for six idealized conditions in which joint rotation and translation are either fully realized or nonexistent are tabulated in Table C-C2.1. Also shown are suggested design values recommended by the Structural Stability Research Council for use when these conditions are approximated in actual design. In general, these suggested values are slightly higher than their theoretical equivalents, since joint fixity is seldom fully realized.

If the column base in Case f of Table C-C2.1 were truly pinned, K would actually exceed 2.0 for a frame such as that pictured in Fig. C-C2.1, because the flexibility of the horizontial member would prevent realization of full fixity at

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End condition cc

the top of the column. On the other hand, the restraining influence of foundations, even where these footings are designed only for vertical load, can be very substantial in the case of flat-ended column base details with ordinary anchorage (Stang and Jaffe, 1948). For this condition, a design K-value of 1.5 would generally be conservative in Case f.

While in some cases the existence of masonry walls provides enough lateral support for their building frames to control lateral deflection, the increasing use of light curtain wall construction and wide column spacing for high-rise structures not provided with a positive system of diagonal bracing can create a situation where only the bending stiffness of the frame itself provides this support.

Table C-C2.1

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code		Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free				

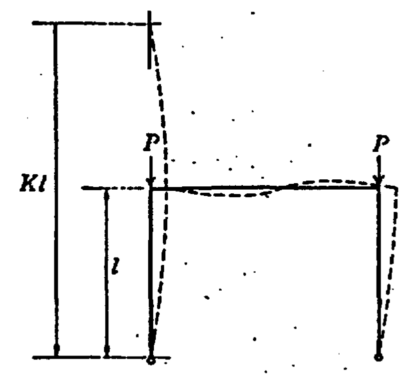

Figure C-C2.1

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GRAVITIES

istance	Weight Lb. per Cu. Ft	Specific Gravity
SEASONED ontent by		
mber 15 to 20% r up to 50%		
, red	40	0.62-0.65
ite, red	22	0.32-0.38
	41	0.66
	30	0.48
s spruce	32	0.51
	25	0.40
	45	0.72
	29	0.42-0.52
	49	0.74-0.84
	46	0.73
d	43	0.68
te	33	0.53
nut	54	0.86
	59	0.95
lack	41	0.65
	46	0.74
on	32	0.51
	30	0.48
	26	0.41
w, long-leaf	44	0.70
w, short-leaf	38	0.61
	30	0.48
California	26	0.42
ite, black	27	0.40-0.46
ck	38	0.61
ite	26	0.41
UIDS		
0%	49	0.79
iatic 40%	75	1.20
91%	94	1.50
huric 87%	112	1.80
66%	106	1.70
able	58	0.91-0.94
al, lubricants	57	0.90-0.93
, max density	62.428	1.0
°C.	59.830	0.9584
	56	0.88-0.92
w, fresh fallen	8	.125
water	64	1.02-1.03
0 mm.	.08071	1.0
	.0478	0.5920
xide	.1234	1.5291
noxide	.07821	0.9673
ating	.028-.036	0.35-0.45
al	.038-.039	0.47-0.48
	.00559	0.0693
	.0784	0.9714
	.0892	1.1056

e of gases to air at 0°C. and 760 mm.
c gravities, except where stated that

WEIGHTS OF BUILDING MATERIALS

Materials	Weight Lb. per Sq. Ft
CEILINGS	
Channel suspended system	1
Lathing and plastering	See Partitions
Acoustical fiber tile	1
FLOORS	
Steel Deck	See Manufacturer
Concrete-Reinforced 1 in.	
Stone	12½
Slag	11½
Lightweight	6 to10
Concrete-Plain 1 in.	
Stone	12
Slag	11
Lightweight	3 to 9
Fills 1 in.	
Gypsum	6
Sand	8
Cinders	4
Finishes	
Terrazzo 1 in.	13
Ceramic or Quarry Tile ¾ in.	10
Linoleum ¼ in.	1
Mastic ¾ in.	9
Hardwood ⅞ in.	4
Softwood ¾ in.	2½
ROOFS	
Copper or tin	1
Corrugated steel	See Manufacturer
3-ply ready roofing	1
3-ply felt and gravel	5½
5-ply felt and gravel	6
Shingles	
Wood	2
Asphalt	3
Clay tile	9 to 14
Slate ¼	10
Sheathing	
Wood ¾ in.	3
Gypsum 1 in.	4
Insulation 1 in.	
Loose	½
Poured-in-place	2
Rigid	1½

Materials	Weight Lb. per Sq. Ft
PARTITIONS	
Clay Tile	
3 in.	17
4 in.	18
6 in.	28
8 in.	34
10 in.	40
Gypsum Block	
2 in.	9½
3 in.	10½
4 in.	12½
5 in.	14
6 in.	18½
Wood Studs 2 × 4	
12–16 in. o.c.	2
Steel partitions	4
Plaster 1 in.	
Cement	10
Gypsum	5
Lathing	
Metal	½
Gypsum Board ½ in.	2
WALLS	
Brick	
4 in.	40
8 in.	80
12 in.	120
Hollow Concrete Block (Heavy Aggregate)	
4 in.	30
6 in.	43
8 in.	55
12½ in.	80
Hollow Concrete Block (Light Aggregate)	
4 in.	21
6 in.	30
8 in.	38
12 in.	55
Clay tile (Load Bearing)	
4 in.	25
6 in.	30
8 in.	33
12 in.	45
Stone 4 in.	55
Glass Block 4 in.	18
Windows, Glass, Frame & Sash	8
Curtain Walls	See Manufacturer
Structural Glass 1 in.	15
Corrugated Cement Asbestos ¼ in.	3

For weights of other materials used in building construction, see pages 6-7 and 6-8

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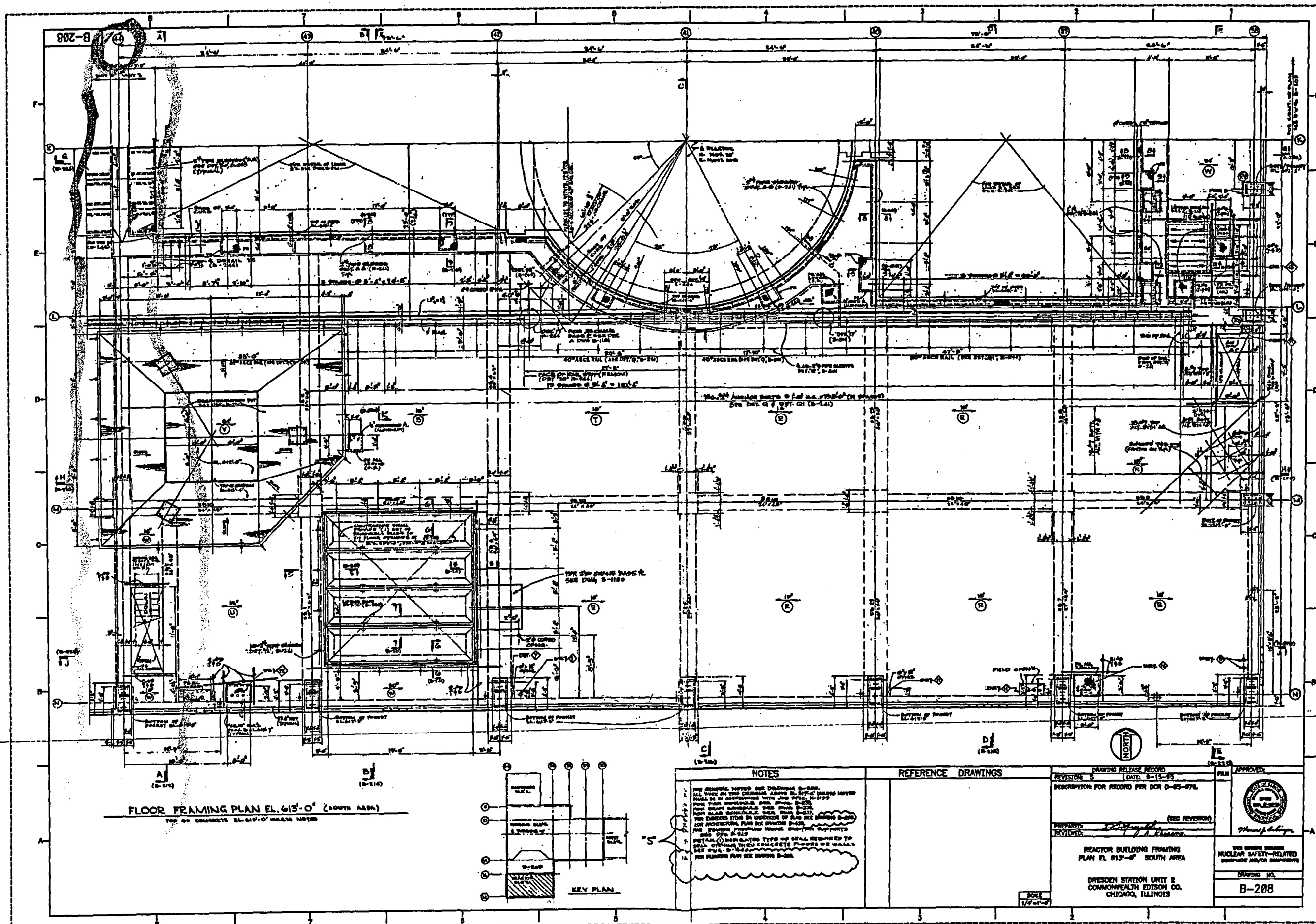
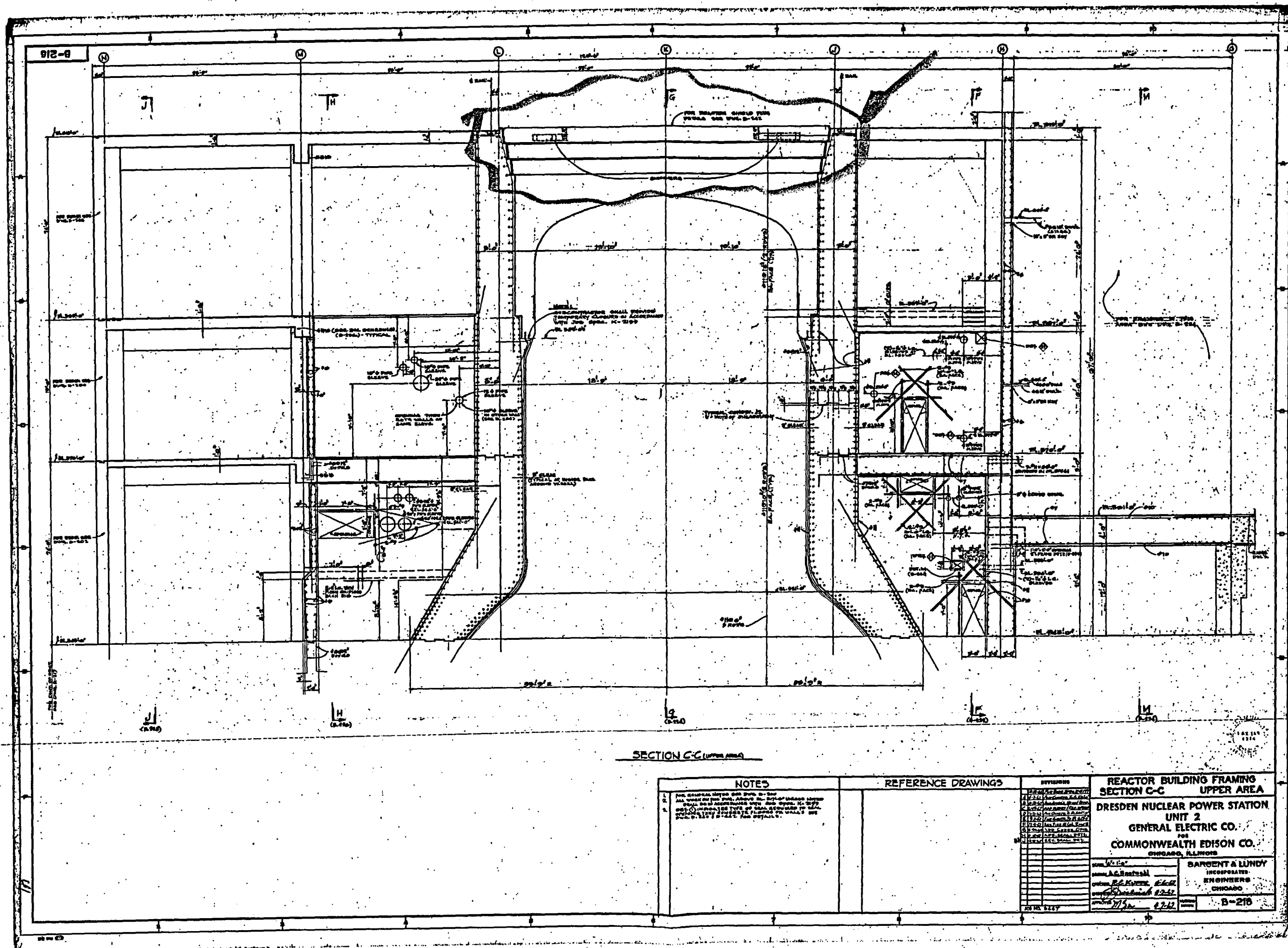
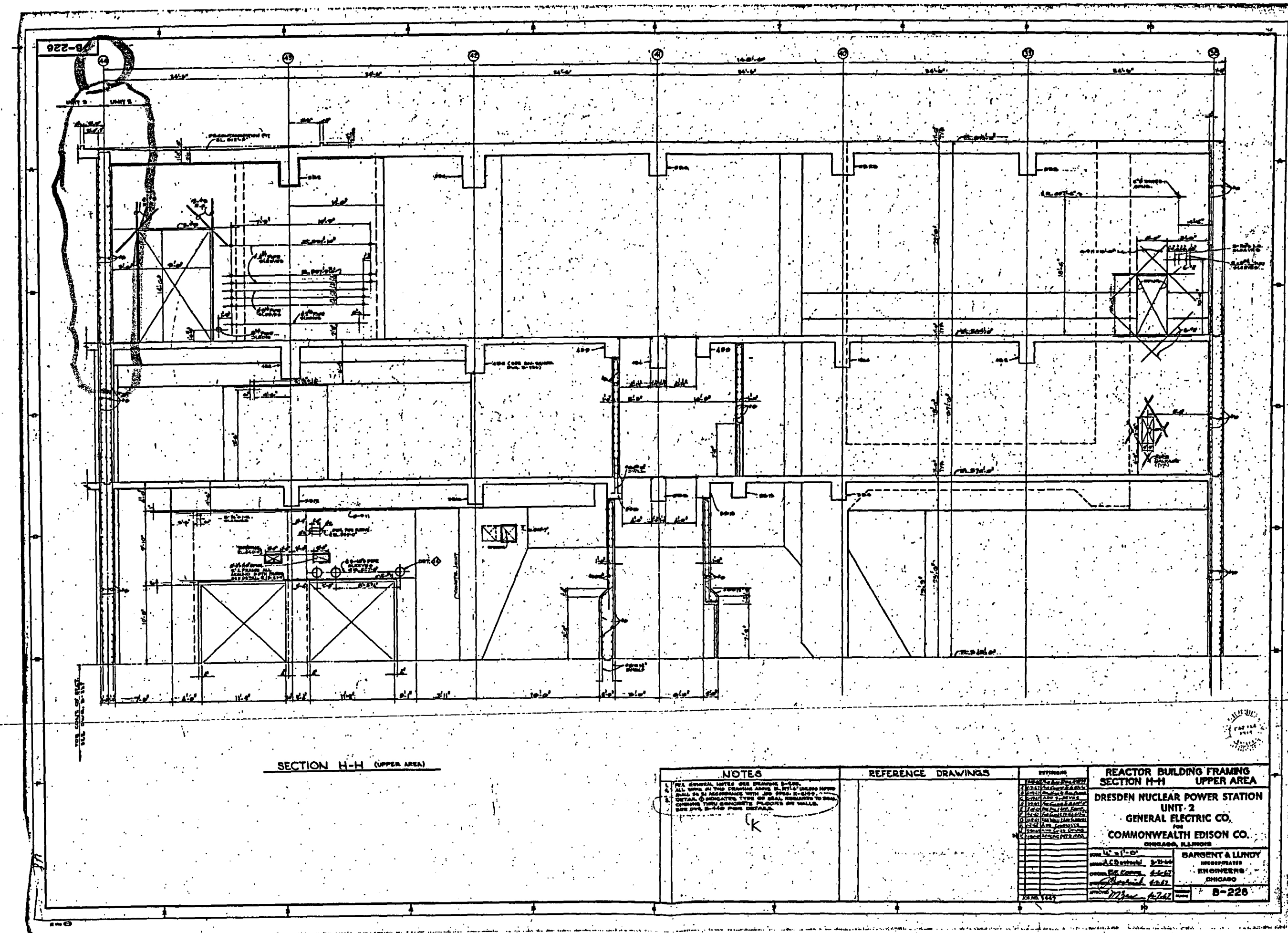


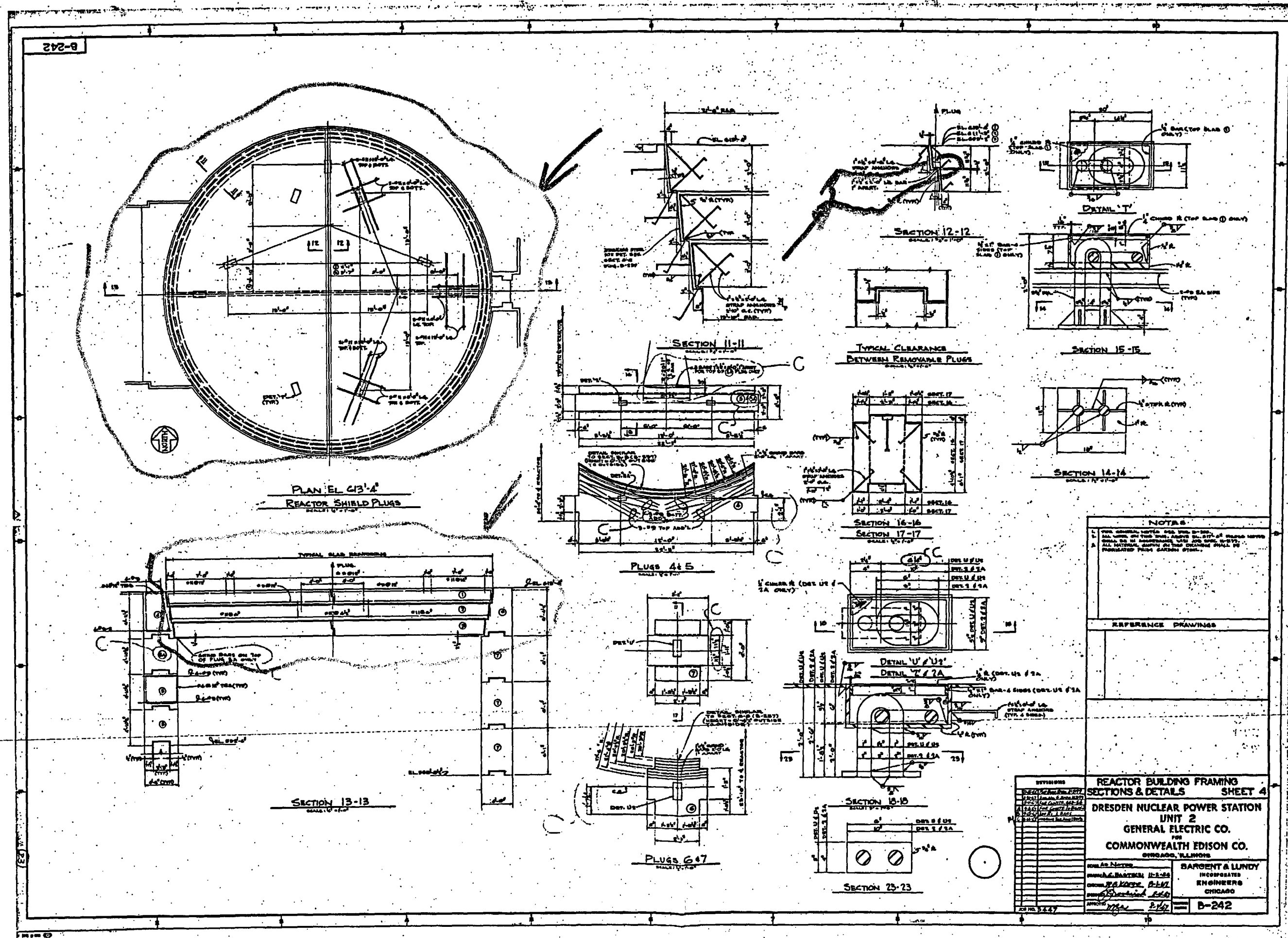
REACTOR BLDG. FRAMING PLAN, EL. 613'-0", SOUTH AREA



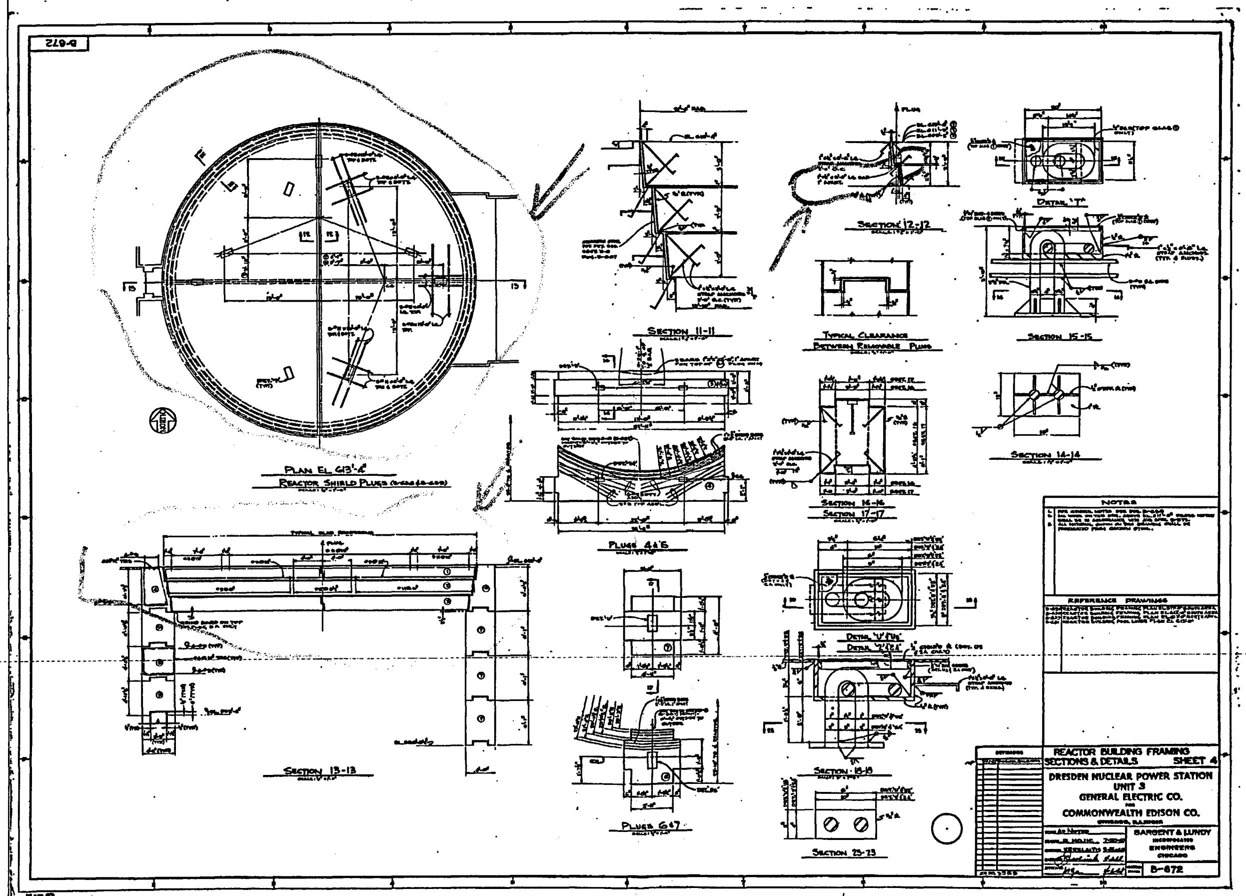
REACTOR BLDG. FRAMING SECTION C-C, UPPER AREA UNIT



REACTOR BLDG. FRAMING SECTION H-H, UPPER AREA UNIT



REACTOR BLDG. FRAMING SECTIONS & DETAILS, SHEET 4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