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ADDENDUM TO

REPORT ON CONTAINMENT BUILDING RING GIRDER

CONSTRUCTION AND REPAIR

THREE MILE ISLAND NUCLEAR STATION

UNIT No. 1

METROPOLITAN EDISON COMPANY

January 21, 1972

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INTRODUCTION

On December 3, 1971 Metropolitan Edison Company filed with the AEC a document entitled "Report on Containment Building Ring Girder Construction and Repair". The information included in this addendum to the aforementioned report was compiled to answer questions and comments raised by members of the DRL staff at a meeting held on December 28, 1971 in Bethesda, Maryland.

1.0 Visual Inspections

The possibility of using Sonotubes as an aid for visual inspection within the repaired area of the ring girder was reviewed. Sonotubes were rejected since they could only be placed in areas where their usefulness would be marginal, i.e. away from the anchorage zone where the rebar placement would not permit their use. In addition, they would only add to the congestion in the repair area and difficulty of concrete placement.

The Structural Integrity Test is established to verify the design of the containment building. Therefore, during the SIT the containment building is pressurized to 63.3 psig or 1.15 times the design pressure. At this time selected areas of the ring girder are inspected in accordance with the procedures of FSAR Appendix 5E and the section on crack inspection in this amendment.

In addition, these same areas will also be inspected for cracks prior to and during the initial prestressing, and during tendon surveillance.

Initial prestressing and structural integrity test pressure place the ring girder under the most severe load conditions normally expected and therefore will provide clear indication of the effects of repair on the ring girder behavior. A check for cracking patterns in the selected areas during the subsequent integrated leak rate tests where the pressure will be 30 psig is considered unwarranted.

However, surveillance of these same areas for any changes in cracking patterns and size of cracks will be carried out along with the tendon surveillance program established by the technical specifications.

Although the building has been conservatively designed for high pressure, it is considered undesirable to subject the building to the design pressure more than necessary. Therefore, following the initial integrated leak rate test at design pressure of 55 psig, subsequent integrated leak rate tests will be done at 30 psig. This lower pressure is sufficient to verify leak tightness without subjecting the building to unnecessary cycles of high pressure load.

2.0 Strain Gages

The behavior of the ring girder will be checked during the Structural Integrity Test through strategic location of strain gages.

The gages will indicate the strain in the reinforcing bars around the ring girder including those within the repaired areas. The gages are placed on the same group of reinforcing bars at different azimuths around the ring girder at the chosen elevation so that the strains will be comparable for each set of gages. The vertical and hoop strains are expected to be approximately the same for each azimuth at a given elevation. The corresponding stresses are expected to be less than the allowable tensile stress of 20 ksi.

The strain gage locations are listed in the FSAR Appendix 5E Table I. In addition to those three strain gages listed originally in Appendix 5E at azimuths 108°, 245° and 352° for elevation 446', two additional hoop strain gages will be added to azimuths 80° and 320° also at elevation 446'. These additional gages are located within the double dome tendon anchorage zone or within both the single and double tendon anchorage zones of the ring girder repair area to provide index of effect the repaired areas have on the function of the ring girder.

The use of strain gages is discussed in FSAR Appendix 5E Section 4.2. The gages are installed in pairs to preclude the loss of strain gage readings in a particular location should one of the pair of gages malfunction. The gages will be utilized during the Structural Integrity Test and, therefore, have been waterproofed and protected in such a manner to assure their usefulness at least to that time. Since it is not anticipated that they will be used again, they have not been designed for the plant service life. Inspection of cracking patterns in selected zones will be the method of future structural surveillance in the ring girder zone following the Structural Integrity Test.

All test results will be collected, recorded, and evaluated in accordance with the procedures given in Appendix 5E of the FSAR.

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3.0 Crack Inspection

Four strategic locations around the dome tendon anchorages will be inspected for cracking before and during prestressing, during the structural integrity test, and as part of the in-service tendon surveillance program. Each of the four areas will be a minimum of 6 ft. wide and 12' high encompassing more than the full height of the repair area. Three of the areas will be located in the north part of the ring girder in the repair area while the fourth area will be located in the south part. The exact locations of the white wash areas will be as stated in Table I of Appendix 5E in the FSAR. All cracks larger than five (5) mils width will be recorded. The initial inspection will be done prior to white washing. The area will then be white washed and gridmarked in accordance with FSAR Appendix E 5.1.3 prior to prestressing. Cracks will be recorded during and following the prestressing operation. During the Structural Integrity Test, these areas will be inspected again in accordance with the procedures for the SIT given in FSAR Appendix 5E.

And finally, these four designated areas will be inspected during each tendon surveillance for changes in the crack size or pattern. These results will be compared with earlier results and will be retained as records.

4.0 Electrical Conductivity Across Cadwelds

There is no design requirement for the reinforcing bars to serve as any part of the grounding system for the containment building and, therefore, electrical continuity through the cadwelds is not a requirement. The containment building has a separate lightening protection and grounding system.

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5.0 Local Stress Distribution Behind Bearing Plates

The anchorage zone design fulfills the allowable bearing stress requirements of ACI-318-63 section 2605C. The bearing stress on the concrete beneath the bearing plates does not exceed the allowable bearing stress in any of the anchorage zones including those in the repair area.

A 30 degree stress distribution is assumed in each direction beneath the bearing plate where the concrete configuration permits. The design beneath the plate is similar to spirally reinforced columns up to and conservatively beyond the point where the concrete compressive stress does not exceed the allowable compressive stress.

With the above distribution, the allowable compressive stress in the concrete is reached in less than 6 inches behind the bearing plate. Thus, considering the effect of adjacent bearing plates on the compressive stress at a point behind the plates, the design of the lead-in zone similar to a column conservatively ensures allowable compressive stresses beyond a very short distance behind the plates into the concrete.

The concrete has been excavated much more than 6 inches behind the bearing plates in the repair area. Therefore, no problems peculiar to the concrete in the repaired area are expected with regard to bearing stresses and distribution of prestress force into the ring girder.

6.0 Lap Splices of Reinforcing

Lap splicing of reinforcing steel is permitted in the ring girder in accordance with the requirements of ACI-318, Section 805.

7.0 Butt Weld Splices

Butt welding of reinforcing steel in the ring girder will be permitted when cadweld splices or lap splices cannot be executed provided the following conditions are observed:

a. Specific splice point and rebar location are identified.

b. Low stressed bars are involved

or

If higher stressed bars are involved, only a low percentage of these bars may be butt welded in a given area. Classification of bars (higher vs. lower-stressed) will be by a representative of Gilbert Associates, Inc. The basis for classification will be design stress requirements.

c. Specific approval of a representative of Gilbert Associates, Inc. is obtained on a case basis.

d. Welding is accomplished in accordance with a welding procedure prepared for welding rebar in the ring girder area. Welding will be done only by welders qualified to this procedure.

e. All required inspections and tests are carried out.

8.0 Reinforcing Steel Butt Welding Procedure

A welding procedure has been prepared for the butt welding of reinforcing steel in the ring girder and the weld repair of damaged rebar. The procedure covers base material, joint and welding details, weld rod, control of weld rod, welding parameters, inspections, tests, and qualifications.

The procedure has been reviewed by the Owner, the Engineer, the Owner's consultant, and formally approved by the Owner.

The procedure has been qualified with the rebar ends free and also with the rebar ends restrained.

Because much of the welding will be done in congested areas, welders who will be assigned to this work will have been qualified under space restricted conditions.

9.0 Nondestructive Testing of Butt Welded Rebar Joints

Reinforcing steel spliced by butt welding will be nondestructively examined as follows:

(a) Each completed joint - 100% visual examination

(b) Each completed joint - 100% magnetic particle inspection

(c) 10% of the completed joints will be radiographed. Joints to be radiographed will be selected on a random basis by UE&C QC personnel.

10.0 Repair of Reinforcing Steel Mechanical Defects by Welding

As stated in the report (Section 4.01.2) reinforcing steel containing notches in excess of 3/32" deep will be removed and replaced. However, in those instances where such removal would create significant additional damage which may be virtually impossible to repair, welded repairs to defects will be permitted, subject to the same limitations applied to butt weld splicing (Section 7.0). Separate qualifications of welders for such repair is required.

It is also anticipated that some larger defects may be present which obviously will not affect the strength or ductility of the repaired or replaced reinforcing. (For example, notches near the end of the rebar). Some such oversize defects may be allowed to remain unrepaired, but only with the specific prior approval of a representative of the Engineer.

11.0 Control of Welding Electrode

The welding electrode to be used in the butt welding of reinforcing in the ring girder will be of the low hydrogen type (E7018). The issuance and use of the electrode will be controlled in strict accordance with an approved construction procedure subject to Quality Control surveillance. Electrodes used in the ring girder repair will be from heats selected and maintained segregated for ring girder repairs.

12.0 Reinforcing Material Control

Reinforcing steel for the Three Mile Island project is purchased and received on a "heat" basis. After receipt, user samples (for mechanical tests) are selected from each heat and size received. Following user testing, the heats are "released for construction".

Review of receiving records and documentation show that no heat of Grade 40 rebar has been received with a carbon content in excess of 0.44% based on ladle analysis.

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13.0 Quality Control of Ring Girder Repair

Repair of the ring girder will be accomplished and controlled, using approved construction procedures covering all aspects such as excavation of unconsolidated concrete, tests of remaining concrete, repair of tendon conduit, removal and replacement of rebar, and concrete replacement. The construction procedures will also identify the construction records to be maintained.

The Quality Control checklist prepared to cover each applicable construction procedure will list inspection points, tests to be performed, acceptance criteria, and data to be recorded. Inspection points treated as "hold points" are identified.

Prior to replacement of concrete and after all excavation and repair is completed in each segment, the UE&C Quality Control Group will perform a final review of inspections and records to date prior to release of the segment for concreting.

Replacement of concrete will be inspected and tests performed under surveillance of a QC inspector assigned to the location of each repair placement.

14.0 Reinforcing Steel-Defect Size Test Program

The defects introduced in the reinforcing steel test program detailed in the report (Section 4.01.2) were accomplished manually using a wide blunt chisel. Photographs taken of the test samples are available at the project site for review.

15.0 Ring Girder - South 180°

The relocation of construction joints in the South half of the ring girder and the removal of lap spliced reinforcing steel above each lift provided complete visual and physical access to South lifts 3, 4, and 5. Therefore, consolidation of concrete and inspection of placing was accomplished without interference or unusual placing conditions.

A detailed visual inspection of the surface of the South 180° will be made prior to tendon installation. This inspection will be fully documented.

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16.0 Application of Bonding Compound

The surface of the concrete remaining after excavation will be somewhat rough and irregular. However, no minimum thickness of the bonding compound to be used is required to effect bond. Likewise excessive thicknesses do not affect the quality of the bond. Therefore, the following conditions governing application will be adhered to:

(a) Both spray and brush application may be used. Areas carefully checked for complete coverage according to manufacturers recommendation.

(b) Epoxy compound cures rapidly and must be "live" at the time of concrete placement. Application of compound shall proceed as closely ahead of the concrete placing as practical. However, if compound has set prior to placing concrete, it shall be reinstated "live" through application of an additional coat per the manufacturers recommendations.

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Several editorial comments about the report require clarification and these are as follow:

Table of Contents - omission

Add to second page of Table of Contents:
"4.025 - Quality Assurance & Control - Pg 4-7".

Sequence of Major Events - (Pg i) - erratum

The date shown for "Complete entire Ring Girder Repair" should be changed to 4/15/72.

Page 2-1 - Third paragraph - clarification

The report indicates a heavy concentration of reinforcing steel. It should be noted that rebar spacing is in accordance with the minimum requirements of ACI 318.

Page 2-2 - Last paragraph - clarfication

The weld of the galvanized tendon conduit to the ungalvanized bearing plate is made on the O.D. of the conduit to the inside face of the bearing plate. These welds were made in the fabricating shop. Galvanize coating was stripped from the O.D. of the conduit, and a galvanize paint applied over the surface of the finished weld.

Page 2-3 - First Paragraph - clarification

Mention is made of a "joint bonding agent". The material used was an epoxy based bonding agent commonly used and approved for use on the Three Mile Island project.

Page 2-4 - Third paragraph - clarification

Concrete placing operations for Lift 3, North 180°, were continuous. During placing operations at night, adequate lighting was employed. No precipitation occurred during the time of the pour.

Page 2-4 - Fourth paragraph - clarification

Mention is made of slow placement rate affecting slump and workability. It should be noted that all concrete placed, however, had a measured slump, just prior to discharge from the truck, of between 1" to 3½".

Page 2-5 - First Para. - Section 2.03.4 - clarification

Mention is made of rebar removed above each lift. The rebar temporarily removed was that which had been lap spliced. Splices were unwired, bars removed, and when bars were replaced they were again lap spliced using tie wire.

Page 2-5 - First Para. - Section 2.03.4 - clarification

Mention is made the South lifts 4 & 5 employed the use of a concrete pump in addition to cranes. It should be noted that a concrete pump was also employed in placing concrete in Lift 3 - South.

Four sets of test cylinders were made representing the 120 cubic yards placed in lift 3. The second and third of these sets were made from the same truckload of concrete, one set from the truck discharge and one set from the pump discharge. Slump at pump discharge was 1/2" lower (3" vs. 3½") and 28 day cylinder compressive strength was 5820 psi (pump discharge) vs. 5750 psi (truck discharge).

A rather extensive correlation has been made on the project between concrete properties at truck discharge vs. those at pump discharge. On the average it has been found that through the pump there is a 1/2" slump loss, a 1% air content decrease, and an 8% strength increase.

Page 2-6 - Section 2.04.1 - Second Paragraph - clarification

Mention is made of "Cadweld electrical connections". These are better defined as "Cadwelded grounding connections".

Page 2-7 - Section 2.04.3 - Second Paragraph - clarification

Mention is made of the "proper mix". The mix employed was identical to that described in report section 4.02.5 (Page 4-8).

Page 4-2 - Section 4.01.2 - Third Paragraph - clarification

Reinforcing bars remaining in the excavated areas will be thoroughly visually inspected for defects. It is not anticipated that the area will be so congested as to prevent direct inspection. However, should they be required, inspection mirrors will be used.

The inspector checking remaining rebar will be provided with a sketch of the area and as each bar is inspected, he will "color out" that bar on the sketch. (Or note it is to be removed and then verify removal).

Page 4-5 - Fifth Paragraph - clarification

Mention is made of "1" square hoops". These are better described as "1" square stock bent into a circle".

Page 4-8 - Second Paragraph - clarification

The air content of the mix described is shown as 4-6%. The air is entrained by use of a vinsol resin type chemical admixture.

Figure A-1 - Elevations

The elevation of the lower construction joint shown in the Figure is 443' - 0½". The elevation at the top of the vertical tendon bearing plate is 453' 6".

In figure A-6, A-7, and A-8, the exact elevations of the limits of the replacement concrete pours are unknown at this time and will vary somewhat from segment to segment depending on the amount of concrete actually excavated.