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ADDENDUM 3

REPORT ON CONTAINMENT BUILDING RING GIRDER

CONSTRUCTION AND REPAIR

THREE MILE ISLAND NUCLEAR STATION

UNIT NO. 1

METROPOLITAN EDISON COMPANY

August 23, 1972

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Metropolitan Edison Company
Three Mile Island Nuclear Station, Unit #1
Structural Engineering Branch

Request for Additional Information

5.25 The Addendum #2 should present, in addition to the four loading cases listed (normal operation, test condition, accident condition, initial prestress), the load combinations which include the OBE, DBE, airplane impact and tornado loads. A listing of minimum safety factors in the repaired girder joints should be included. Where the shear bars are used to carry also the bursting forces due to prestresses and the tensile stress concentrations due to tendon curvature and the "hole" effect of flexible sheathing, the corresponding loads should be included in the analysis. The effects of shrinkage, creep of concrete, and relaxation of prestressing should be considered in all cases where they may modify the normal forces acting on the horizontal joints and therefore change the friction resistance of the joint.

Response

The horizontal ring girder construction joint condition at elevations 443 ft-0 in. and 446 ft-9 in. has changed substantially from that investigated in Addendum 2 to the Ring Girder Report. Concrete excavation into the ring girder has extended up to and behind the vertical tendon conduits for more than fifty percent of the South 180°. Figure 1 indicates the extent of concrete removal for the South 180°. Construction joints for the replacement concrete will be located at different elevations than the original joints. In addition all joints will be carefully prepared ensuring complete aggregate interlock at the joints. The shear study on the construction joints presented in Addendum 2 was based on assumptions that are no longer valid; namely, the construction joints at elevation 443 ft-0 in. and 446 ft-9 in., extending horizontally across the full thickness of the ring girder, utilize only friction for transferring shear. The repair will provide complete aggregate interlock at the new construction joints, located at different elevations than the original construction joints.

It should be noted, however, to make Addendum 2 complete that the two joints were investigated for all load combinations with the results of four representative cases presented in that Addendum. As stated in Addendum 1, the stresses introduced into the ring girder by aircraft impact are relatively small and do not control the design of the ring girder (Reference Addendum 1, Section 5:04). Section 5.2.3.2 of the FSAR states that the wind load basedon a 390 MPH tornado does not exceed the design earthquake load for any aspect of the design. The required coefficients of friction for the OBE and DBE load combinations are less than those required for Normal Operation listed in Table 2 of Addendum 2.

Shear resistance across the thickness of the ring girder at elevations 443 ft-0 in. and 446 ft-9 in. should be as good as that intended by the original design following the repair. The original design was not based on a factor of safety analysis; however, a reasonable estimate of the minimum factor of safety at these two elevations against shear loading on the repaired ring girder would be two.

Shear bars are not used to carry bursting forces due to prestress. There are no tensile stress concentrations due to tendon curvature in the ring girder because all tendon trajectories are straight within the ring girder. The "hole" effect of the flexible sheathing will result in a small increase in stress around the hole. However, since the concrete stress in the area around the tendon sheathings is low, the net result of the redistribution of stresses into the concrete around the holes is still within the allowable concrete stresses.

All analytical work used for the evaluation of the two construction joints of Addendum 2 considered the effects of shrinkage, creep of concrete, and relaxation of prestressing (Reference FSAR Section 5.2.2.3.6). The above effects have also been addressed in Addendum 1 with regard to the North 180° repair. The conclusions presented on these effects are valid for the South 180° repair.

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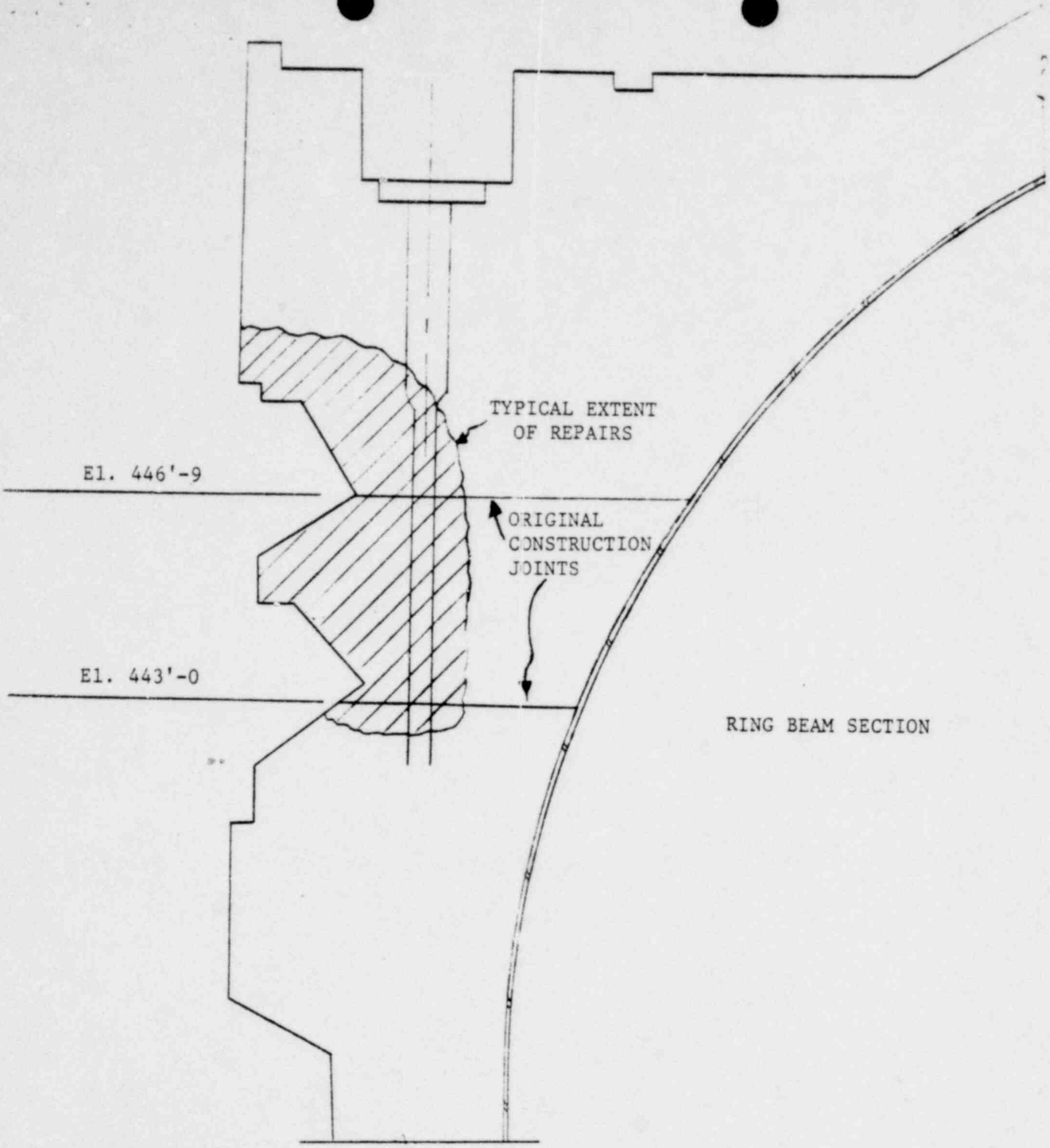

FIGURE 1
TYPICAL EXTENT OF REPAIRS

5.26 It had been previously agreed that welding of rebars will be acceptable only when the conditions listed in Sections 7.0 (Butt Weld Splices), 8.0 (Reinforcing Steel Butt Weld Procedure), 9.0 (Nondestructive Testing of Butt Welded Rebar Joints), and 10.0 (Repair of Reinforcing Steel Mechanical Defects by Welding) of Addendum #1 are satisfied. Indicate compliance with these conditions and the percentages of bars butt welded and/or repaired by welding and for the north and south parts of the ring girder separately.

Response

Welding of rebars for the South 180° of the ring girder shall be subject to the same acceptance criteria which was used for the North 180° of the ring girder. The North 180° in which the repair has been completed did not have any "butt weld splices" or "repair of reinforcing steel mechanical defects by welding." All reinforcing steel repairs to the North 180° were made by Cadweld splicing.

Conditions of excavation for the South 180° are much the same as that of the North 180° and cadwelding is expected to be used almost exclusively for the repairs.