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REPORT

ON

EVALUATION OF POSSIBLE DAMAGE

TO

EMBEDDED REINFORCING STEEL DURING INSTALLATION OF DRILLED-IN ANCHORS

REACTOR CONTAINMENT

NORTH ANNA UNITS 1 AND 2

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INDEX

PURPOSE 1.9

The purpose of this report is to provide an evaluation 1.11
of possible damage to embedded reinforcing steel in Reactor 1.12
Containments Unit 1 and Unit 2 during the installation of anchors
drilled in concrete and to determine if there were any effects 1.13
significant enough to compromise the structural design of the
reinforced concrete.

FINDINGS 1.16

Investigations were conducted at the jobsite to 1.18
determine the extent of cutting of embedded reinforcing steel. 1.19
Findings are summarized below: 1.21

1. On August 19, 1975, the special Drillco., Inc., diamond 1.23
tipped drill bit, called the "Rebar Eater" (trade name), 1.24
specifically designed to bore into concrete and to cut
any interfering reinforcing steel, came into use by pipe 1.25
hanger installation crews at the site. These drills 1.26
were used to install mechanically expanded drilled-in
anchors which generally ranged in size up to a maximum 1.27
of 1 in. in diameter.

2. Prior to August of 1975, the primary tool used on the 1.29
project for drilling concrete for mechanically expanded,
drilled-in anchor bolts was the Hilti Fastening Systems, 1.30
Inc., electric drill and carbide tipped bit. Hilti bits 1.31
have been demonstrated to be ineffective as a means of
cutting embedded reinforcing steel. Extreme pressure is 1.32
required on the drill in order to make any penetration
into steel and many bits are often damaged in the 1.33
process. A site test using a 1/2 in. diameter Hilti bit 1.34
showed that it took two hours to penetrate 7/8 in. into
a 2 1/2 in. diameter rebar.

3. On March 29, 1976, Engineering issued documents which 1.36
restricted cutting of reinforcing steel without 1.37
Engineering approval, and on April 9, 1976, instructions
were issued and implemented by our field forces to 1.38
ensure compliance.

4. The only methods available for cutting reinforcing steel 1.40
in holes for mechanically expanded drilled-in anchors, 1.41
other than with the special Drillco bit, were with air-
arc equipment, utilizing a copper-coated carbon rod and 1.42
a welder's electrode clamp which had provision for an
air supply, or with carbon steel drill bits. Since the 1.44
air-arc equipment is not designed for this application
and may in fact be hazardous to the worker when so 1.45
employed, a thorough investigation has identified only
six pipe hangers on which it was used. Similarly, since 1.46
the effort required to cut a reinforcing bar with a

carbon steel bit is extensive and time consuming, 1.47
investigations indicate this was an extremely rare
practice.

5. Electrical, heating, ventilation, and air conditioning 1.49
(HVAC) and instrumentation disciplines used only Hilti 1.50
concrete bits when drilling holes for anchors and no
evidence can be found that prior to April 9, 1976, they 1.51
used air-arc equipment, carbon steel drill bits, or the
special Drillco bit, to cut embedded reinforcing steel.
Further, the majority of the types of anchors used by 1.52
these disciplines were designed to be installed to 1.53
embedded concrete depths which were less than the
specified cover over reinforcing steel.

6. When interferences existed between a proposed location 1.55
for a drilled-in anchor and an existing embedded 1.56
reinforcing bar, the alternatives available to craftsmen
prior to April 9, 1976, were:

   a. Reposition the drill hole 1.58

   b. If the Hilti drill bit struck a rebar off center, 2.1
   it could sometimes be deflected off the bar,
   resulting in a hole at a slight angle which could 2.2
   be utilized with a bevelled washer.

   c. Install an anchor bolt in the hole as is, with an 2.4
   embedded depth equivalent to the concrete cover 2.5
   over the reinforcing bar.

   d. Cut through the reinforcing bar and continue the 2.7
   drill hole to the required depth. 2.8

Cutting of reinforcing steel was only one of the four 2.10
alternatives available when boring holes for drilled-in
anchors and being the most time-consuming, was the least 2.11
likely to have been carried out.

SCOPE 2.14

These findings indicate that our primary consideration 2.16
should be with the extent of possible damage to embedded
reinforcing steel when holes were drilled in concrete for pipe 2.17
hanger and support installations during the period between 2.19
August 19, 1975, and April 9, 1976.

APPROACH 2.22

General 2.24

Reinforced concrete structures are comprised primarily 2.26
of four kinds of structural elements: slabs, walls, columns, and 2.27
beams. Slabs and walls are generally reinforced with uniformly 2.29
spaced, orthogonal patterns of reinforcing bars, in the top and 2.30
bottom of slabs, or in the near face and far face of walls.
Columns and beams are generally reinforced with groups of closely 2.31
spaced, parallel reinforcing bars, in all four faces of columns, 2.32
and in the top and bottom of beams. Orthogonal reinforcement is 2.33
always provided in the form of ties, uniformly spaced in columns,
and is sometimes provided in the form of stirrups, uniformly 2.34
spaced in beams, near the supports.

The frequency of possible interferences between drill 2.36
holes and embedded reinforcing steel will decrease as the center 2.37
to center spacing of rebars becomes large, such as in slabs and
walls as compared to columns and beams. Therefore, given a known 2.39
quantity of drill holes in a slab or wall, bored with the special
Drillco bit, the likelihood of interference and damage is far 2.40
less than if the same number of holes were bored into a beam or 2.41
column. For this reason an evaluation of structural integrity of 2.42
columns or beams requires a more precise determination of damage 2.43
to reinforcing steel in all elements, while it may be possible to
base an evaluation of walls and slabs on a statistical analysis 2.44
of a sampling of such elements to determine the maximum possible
damage to reinforcing steel.

Columns and Beams 2.47

The Project Structural Engineer made inspection trips to 2.49
the jobsite on October 21, and November 10, 1976, and identified 2.50
columns and beams having any drilled-in anchors. Elements were 2.52
selected for analysis if they had any anchors at points of
critical stress, or where several anchors were installed 2.53
regardless of location. Date of installation of the anchors had 2.54
no bearing on this selection. Subsequent to these inspections, 2.55
locations of reinforcing bars in these columns and beams were
determined either by using the "R Meter" manufactured by James 2.56
Electronics, Inc., or by chipping away portions of the concrete 2.57
cover. Where an apparent interference existed between a 2.58
reinforcing bar and a drilled-in anchor for a pipe support,
ultrasonic test methods were used to check the depth of embedment 2.59
of the anchor. The amount of concrete cover over the bar in 3.1
question was measured and compared to the depth of embedment of 3.2
the anchor. Based on these measurements, possible damage to the 3.3
bar was evaluated. This check was made regardless of how or when 3.4
the pipe support anchor was installed.

Based on a conservative estimate of the extent of 3.6
possible damage to reinforcing steel in a given column or beam,

and knowing the design forces and moments, an analysis was then 3.7
performed to determine if the revised capability of the element
equaled or exceeded the capability required by the original 3.8
design.

This approach constituted a 100 percent visual 3.10
inspection of all beam and column elements, and an analytical 3.11
review of those having any possible significant damage to
embedded reinforcing from holes drilled for pipe supports. As a 3.12
result of our inspections, it was determined that there were no
drop beams or concealed (local) beams which required review. 3.13

Slabs and Walls 3.16

Inspections by our field forces indicated that the 3.18
highest concentrations of pipe support embedments installed with 3.19
drilled-in anchors occurred in the walls and in the underside of
floors of the three steam generator cubicles in the Unit 1 3.21
containment. This is consistent with the fact that the pipe 3.21/1
support embedment installation effort had barely begun in the
Unit 2 containment at that point in time. 3.21/2

Since structural designs for each of the cubicles are 3.23
similar, only one reactor coolant pump (RCP) side radial wall, 3.24
steam generator (SG) side radial wall, and one section of the
crane wall, with the largest quantity of drilled-in anchors for 3.25
pipe supports, was initially selected for anchor installation
review. Thus, in Reactor Containment - Unit 1, the RCP side 3.26
radial wall studied was from Cubicle A, the SG side radial wall
from Cubicle C, and the crane wall from Cubicle B. In Reactor 3.28
Containment - Unit 2, the RCP side radial walls had too few
anchors to warrant study, the SG side radial wall studied was 3.29
from Cubicle A, and the crane wall was from Cubicle C.

Due to the existence of highly localized design loadings 3.31
on the cubicle floor slabs and the fact that the locations of 3.32
critical sections for stress may vary, it was decided to
investigate each cubicle floor slab in Units 1 and 2
individually. Anchor installations at points of critical stress 3.32/1
were studied and a conservative estimate of possible rebar damage
was made.

In general, the procedure for anchor installation review 3.35/1
consisted of identifying all pipe support embedments having 3.36
drilled-in anchors and plotting their locations on each face of
wall elements and on the underside of the floor slab (the floor 3.37
slab top surface did not warrant an analytical review). Next the 3.38
date of installation of all drilled-in anchors for pipe supports
was researched to determine which of those were installed within 3.39
the "period of concern" from August 19, 1975, to April 9, 1976.
If a date of installation was uncertain or unknown, it was 3.40
assumed to have fallen within the period of concern (see Findings 3.41
Nos. 1 and 3).

At this point a statistical analysis was performed to 3.43
determine the maximum possible interferences between known anchor 3.44
locations for pipe supports installed during the period of
concern, and a known orthogonal reinforcing steel pattern. All 3.45
holes for anchors installed in this period were assumed to have
been drilled with the special Drillco bit and were, therefore, 3.46
potentially damaging to embedded reinforcing steel.

In sections having two or more layers of reinforcing 3.48
steel in the same direction, if the type of drilled-in anchor 3.49
used was commercially available in a length sufficient to reach
the deeper bar, then the statistical analysis included the deeper 3.50
bar as potentially damaged. This was conservative. However, in 3.52
highly stressed areas having multiple layers of rebar, ultrasonic
test methods were sometimes utilized to determine the embedded 3.53
length of the anchor, and thus, the number of layers of rebar
potentially damaged.

Assuming the maximum possible damage to reinforcing 3.55
steel in a given slab or wall, and knowing the actual yield 3.56
strength of the reinforcing steel from mill test reports, a
comparison was made to determine if the loss of reinforcing bar 3.57
area was sufficiently compensated for by bar strength properties
in excess of those assumed in the original design. In those 3.59
cases where the excess yield strength was not sufficient, the
design forces and moments were reviewed for comparison with the 4.1
section capacity as reduced by bar damage.

This approach constituted a 100 percent visual 4.3
inspection of all slab and wall elements and an analytical review 4.4
of all those elements which could have been critical.

ANALYSIS 4.7

Columns 4.9

Structural analysis of columns, to determine their 4.11
revised capability after bar damage, was performed using the 4.12
methods of ACI 318-71 and the governing load equations of NRC 4.14
Standard Review Plan, Section 3.8.3, Paragraph II.3. Where the 4.15
ultrasonic test showed penetration of a reinforcing bar by a
drilled-in anchor, the anchor was assumed to have hit dead center 4.16
and to have perforated the bar. In most cases the anchor 4.17
diameter was smaller than the reinforcing bar and, therefore,
even a dead center hit could not completely sever the bar. In 4.19
those instances, the damaged bar was treated analytically as one
having an area equivalent to the reduced cross sectional area of 4.20
the perforated bar.

In columns, where the location of damage to one bar was 4.22
separated from the location of damage to another bar by a 4.23
distance equal to or greater than the development length (per
ACI 318-71) of that bar, the damage was treated analytically as 4.24

only one reduction in cross sectional area of a bar. This is 4.25
justified by the fact that the two damaged bars in effect lap
each other, and can still transfer loads carried at their reduced 4.26
sections by bond development. The section of each column then 4.27
having the greatest reduction in bar area was identified.
Verification analysis was then performed on this critical section 4.28
by computer using a S&W program entitled, "Ultimate Strength 4.29
Analysis of Concrete Columns." It is based on accepted ultimate 4.30
strength theories for reinforced concrete design, and where
applicable, assumptions and limitations conforming to ACI 318-71. 4.31

Slabs and Walls 4.34

Statistical analysis of slab and wall elements to 4.36
determine an assumed maximum possible bar damage was performed 4.37
graphically. Locations of all drilled-in anchors installed 4.39
during the period of concern were plotted to a large scale. Then 4.40
a transparency was prepared showing, to the same scale, the
orthogonal rebar pattern indicated on the design drawing in the 4.41
element. The transparency was overlaid on the anchor location 4.42
plot to determine the position of the rebar pattern, relative to 4.43
the anchors, resulting in the greatest number of bar/bolt
interferences. This was done, for each of the two directions, by 4.44
shifting the transparency, approximately one inch at a time,
checking interferences and computing bar area reductions, until 4.45
all possible positions of the rebar pattern had been checked. 4.46
The reduced cross sectional area at a bar/bolt interference was 4.47
computed assuming complete perforation of the bar and based on a 4.48
dead center hit or quarter point hit, whichever appeared closer
on the graphical presentation. 4.49

In slabs and walls which were designed as singly 4.51
reinforced, drilled-in anchors in zones of compression were not 4.52
considered a possible problem provided they were beyond the
cutoff point required for anchorage of flexural tension steel.
Also, where the location of damage to one bar was separated from 4.53
the location of damage to another bar by a distance equal to or 4.54
greater than the development length (per ACI 318-71) of that bar,
the damage was treated analytically as only one reduction in 4.55
cross sectional area of a bar, similar to the column analysis 4.56
method.

In portions of the cubicle floors, it was necessary to 4.58
compare the results of the statistical analysis with design bar 4.59
stresses. Bar stresses were computed using the load equations of 5.1
FSAR Section 2.8.2.2.

RESULTS

Columns

Table I summarizes the results of columns investigated and analyzed. Octagonal columns under the steam generator cubicle floors are given the preface "SG" followed by the letter designating the cubicle they support. Rectangular crane wall columns are designated by the numbers corresponding to the annulus area steel column lines which pass radially through, or on either side of, the concrete column.

Where drilled-in anchors were shown to interfere with column tie bars, the elevation was noted and repair of the tie scheduled if required. Column capacities were therefore based on an evaluation of damage to axial rebars only and no further reductions are required for tie damage.

Many of the columns designated as having drilled-in anchors were shown by further field investigation to be free of any anchor bolt/rebar interferences. These columns have been tabulated anyway, since they were investigated, but are identified as having no bar damage. These columns did not require analysis.

Walls

Table II summarizes the results of walls investigated and analyzed. For each element listed, the following information is tabulated:

1. The percentage by which the actual minimum yield strength for the reinforcing steel for that type of element exceeds the design yield strength. Thus, the mill test reports for the RCP side radial wall of all three cubicles were reviewed to determine which cubicle had the minimum yield value for that type of element, etc.

2. The theoretical percent of reduction in reinforcing bar area under the following conditions:

    a. Only drilled-in anchors for pipe supports installed during the period of concern could have cut reinforcing steel regardless of the method of installation.

    b. All drilled-in anchors for pipe supports could have cut reinforcing steel regardless of the date or method of installation.

The percent reduction in reinforcing bar area was computed by dividing the total area of reinforcing steel cut, as determined by the statistical analysis, by the total area of steel in that face, in that direction.

We believe the individual cubicle elements selected for 5.50
anlaysis are representative of their respective elements in the 5.51
other two cubicles in terms of the quantity and location of
"critical period" pipe support anchors. This belief is based on 5.53
the fact that pipe support installation work proceeded
concurrently in each cubicle.

In the remote event that any cubicle wall element not 5.55
analyzed could have slightly more "critical period" pipe support 5.56
anchors than the respective element analyzed, we have included an
analysis assuming all pipe support anchors could have cut 5.57
reinforcing steel, regardless of the date or method of 5.58
installation, to provide an upper bound on the problem. We 5.59
believe this assumption is unreasonable and entirely unrealistic
and, therefore, we have shown only the average reduction in 6.1
reinforcing bar area for this condition rather than the maximum.
Our assessment of the reduced capacity of the wall element uses 6.2
the larger reduction in rebar area of either the "Pipe Support 6.3
Anchors August 1975-April 1976 Max" or "All Pipe Support Anchors 6.4
Avg."

In order to determine whether the percentage by which 6.6
the actual yield strength of the reinforcing exceeded the design 6.7
yield was sufficient, the following relationship was developed:

F = required yield strength   Ar = reduced rebar area 6.11
Fd = design yield strength   Ad = design rebar area 6.12

$F = Ad \times Fd/Ar$ 6.14

Review of the results contained in Table II show that 6.18
for Reactor Containment - Unit 1 the reductions in rebar area are
not fully compensated for by the actual yield strength of rebar 6.19
in the reactor coolant pump side radial wall of Cubicle A and in 6.20
the crane wall of Cubicle B. In those instances, the design 6.21
stresses due to combined thermal, differential pressure, and pipe 6.22
break loadings were recomputed using these reductions. This 6.23
analysis assumed the maximum reduction in rebar area for a given
layer of rebar in one face of the wall occurred in both faces. 6.24
This is conservative. This analysis has shown that these wall 6.26
elements still meet the design criteria contained in FSAR
Section 3.8.2.2.

Slabs 6.29

Reactor Containment - Unit 1 6.31

The highest design stresses in the floor slabs of the 6.33
steam generator cubicles result from concentrated loads 6.34
associated with primary coolant loop pipe break bumpers. These 6.36

stresses are highly localized relative to the entire floor slab
and, therefore, an assessment of potential bar damage, in terms 6.37
of a percent reduction in rebar cross sectional area, was
directed at a minimum 10 ft x 18 ft area of each floor slab in 6.39
each of the three steam generator cubicles. This area included 6.40
those portions of the floor slab having the highest design
stresses and was also visually representative of the most densely 6.41
drilled portion of the entire floor slab. The actual yield 6.42
strength of reinforcing steel used for analysis was 40 ksi for
slabs in Cubicles A and B, and 42.3 ksi for Cubicle C. 6.43

Estimates of the maximum reduction in rebar area, 6.45
assuming all pipe support anchors installed during the "critical 6.46
period" could have cut rebar, were made for local portions of the 6.47
sample area in each of the three cubicles. The design stresses 6.48
due to combined seismic, thermal, differential pressure, and pipe 6.49
break bumper loadings were recomputed using these reductions.
This analysis has shown that the floor slab still meets the 6.50
design criteria contained in FSAR Section 3.8.2.2.

Reactor Containment - Unit 2 6.53

In Unit 2, a more simplified approach was used to 6.54/1
minimize the need for time consuming field studies of anchor
installations on the underside of the cubicle floor slabs. The 6.54/4
actual yield strength of slab bottom rebar used for analysis was
40 ksi for Cubicle A, 63.5 for Cubicle B, and 56.4 for Cubicle C. 6.54/5
Therefore, only the underside of the Cubicle A floor slab was 6.54/6
studied for "critical period" anchors. Based on experience 6.54/7
gained in Unit 1, a 15 percent reduction in radial and
circumferential rebar area was conservatively assumed for slab 6.54/8
bottom rebar, in Cubicles B and C, to account for possible anchor
installation damage. It is noteworthy that the investigation for 6.54/9
Cubicle A showed that no pipe supports were installed on the
underside of the floor slab prior to April 9, 1976. This fact 6.54/1
was consistent with general observations about the limited extent
of Unit 2 pipe support installations up to that time, and 6.54/1
highlights the conservatism of the 15 percent reduction assumed
in Cubicles B and C.

The design stresses due to combined seismic, thermal, 6.54/1
differential pressure, and pipe break were recomputed using the 6.54/1
actual yield strengths of rebar, and conservative bar area
reductions. This analysis showed that the floor slab still meets 6.54/1
the design criteria contained in FSAR Section 3.8.2.2.

TABLE I

COLUMNS

Column No.	Base Elev.	Axial Rebars Damaged	Factored Loads Moment, M (K-Ft) Axial Loads, P (Kips) Design Loading	Capacity With Rebar Damage
Reactor Containment - Unit 1				
SG-A	214'-5"	None	-	-
SG-B	214'-5"	None	-	-
SG-C	214'-5"	None	-	-
3	214'-5"	None	-	-
7-8	214'-5"	1 bar	P=6077 Mx=5273 My=2486	P=6423* Mx=5545* My=2616*
12-13	214'-5"	None	-	-
18-1	214'-5"	2 bars	P=5341 Mx=6107 My=1400	P=5341 Mx=8371 My=1916

NOTE: *Capacity figures are based conservatively on f 4,000 psi while actual core samples show the i ace compressive strength is 5,307 psi minimum.

TABLE I (CONT'D)

COLUMNS

Factored Loads
Moment, M (K-Ft)
Axial Loads, P (Kips)

Column No.	Base Elev.	Axial Rebars Damaged	Design Loading	Capacity With Rebar Damage	
Reactor Containment - Unit 2					7.41
SG-B	214'-5"	None			7.43
3-4	214'-5"	2 bars	P=5667 Mx=1000 My=8495	P=5722** Mx=1121** My=9540**	7.44 7.44/1 7.44/2
4	214'-5"	1 bar	P=6289 Mx=1547 My=7357	P=6370** Mx=1586** My=7548**	7.45 7.45/1 7.45/2
10	214'-5"	None			7.46
11	214'-5"	None			7.47
14-15	214'-5"	1 bar	P=5370 Mx=1886 My=6310	P=5727** Mx=2043** My=6833**	7.49 7.49/1 7.49/2

Column 13-14 also had damage to one bar. It has not been shown in the table, however, because it was modeled and analyzed compositely with column 12 and the results do not lend themselves to this tabular format. Results demonstrated that the bar damage is acceptable. 7.50/4 7.50/5 7.50/6

NOTE: **Capacity figures are based conservatively on f'c= 3,650 psi while actual core samples show the in-place compressive strength is 4,572 psi minimum.