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FACTOR OF SAFETY STUDY FOR VERMONT YANKEE NUCLEAR POWER STATION BASEPLATE FLEXIBILITY

Prepared For

Yankee Atomic Electric Company
25 Research Drive
Westborough, MA 01581

Prepared by

Earthquake Engineering Systems, Inc.
600 Atlantic Avenue
Boston, Massachusetts 02210

Prepared By: William J. Duffy
William J. Duffy, P.E.

Independent Reviewer (YAEC): Stanley L. Chi

Approved By: Paul D. Baughman
Paul D. Baughman, P.E.

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INTRODUCTION

This report, prepared by Earthquake Engineering Systems, Inc., for Yankee Atomic Electric Company, summarized a study of the pipe support baseplates at the Vermont Yankee Nuclear Power Station.

Based upon reviews of several nuclear facilities, criteria have been developed by the Nuclear Regulatory Commission (NRC) that define the limits of pipe support operability(1)*. Although anchor bolt safety factors of four or five are intended to be the final design and installation objectives, lesser margins may be acceptable on an interim bases.

The purpose of this study was to determine whether baseplates which had been designed for a safety factor of four using rigid plate assumptions will still have at least a safety factor of two when baseplate flexibility is considered. In addition, bolt load multipliers were determined for possible use in evaluating flexibility effects under calculated piping reaction loads.

*Numbers in parentheses indicate references at the end of the report.

BASEPLATE SELECTION PROCESS

Every as-built pipe support package was inspected, and a copy of each baseplate sketch obtained. Dimensions and other pertinent information were checked to assure that each sketch was complete. Once all the sketches were obtained, the baseplates were organized by sorting them into groups having common characteristics, with an additional group of nonstandard baseplates. Nonstandard baseplates are those which cannot be analyzed using the EPLATE computer program. All baseplates were accounted for by compliation of a master list which indicated the corresponding group for each baseplate.

In selecting a particular baseplate to be investigated, an attempt was made to find a plate which would behave in such a manner as to be an upper bound for a large number of plates. Assessment of plates was based upon visual inspection making note of the number of bolts, bolt spacing, attachment location and type, as well as the plate flexibility. In all, 34 plates were analyzed.

TENSION LOADING

An analysis was performed on each baseplate selected to determine the effect of a direct tension load. This load was calculated as the maximum tension in the attachment, using rigid plate assumptions, which does not exceed the allowable pullout force in any of the anchor bolts. With a non-eccentric attachment, all the anchor bolts are at the maximum; however, in other cases the bolt force varies.

Once the tension load was calculated, it was applied to a finite-element computer model of the baseplate(6). Because a baseplate is flexible rather than rigid, as was assumed when determining the applied tension load, there is a difference in the force distribution to the anchor bolts.

At first, the tension load applied to a baseplate was based upon the allowable values of pullout specified for HILTI "Kwik-Bolt" stud anchors with the minimum embedment(2). After some preliminary work, however, it was decided that the allowables would be based upon PHILLIPS "Red Head" concrete anchors(3), as this was the basis of the original design.

MOMENT LOADING

Several baseplates were investigated to assess the impact of flexibility upon the load distribution to the anchor bolts under a moment loading.

Baseplate flexibility can be a greater factor in the anchor bolt load distribution for moment-resistant baseplates, as opposed to baseplates which are loaded in tension only. This magnifies the difference between the assumed linear (rigid plate) and the actual non-linear (flexible plate) action.

In a manner similar to the method used in evaluation of the tension load cases, the external moment calculated was applied to a finite element computer model of the baseplate(6). The externally applied moment was found by determining the maximum moment by rigid plate theory causing all anchor bolt loads to be equal to, or less than the allowable pullout load. Normally the bolt forces varied throughout the plate according to the distance from the attachment.

Allowable anchor bolt pullout was based upon available information from the Phillips Drill Company for "Red Head" concrete anchors(3). Spacing reductions are from the same source.

CALCULATION OF GROUP ACTION

Anchor bolt pullout capacities for HILTI "Kwik-Bolt" stud anchors under various spacing arrangements were calculated in two different ways.

The first approach was to assume that the anchor bolts all had a minimum embedment depth per reference 7 and reduce the anchor pullout capacity on a straight-line basis for bolt to bolt spacing less than the minimum required for full anchor efficiency. When any anchor bolt is located so as to be within the minimum spacing distance of more than one contiguous anchor, each anchor had its allowable pullout reduced proportionately.

A group action procedure(4) was employed when the linear reduction method proved to be unsatisfactory. Capacity was calculated from the concrete tensile strength of the projected net area of the stress cones radiating anchor embedment to the concrete surface. Estimation of the projected areas was accomplished with either an approximate (equivalent squares) or an exact method as appropriate.

RESULTS

The factor of safety for a base plate was calculated by dividing the ultimate bolt capacity by the bolt force from the computer analysis. The ultimate capacity is based on the anchors now in place while the computer force derives from the allowables for the original installation and rigid plate assumptions. The results are tabulated in Tables 1 to 4.

On the first calculation, 22 plates were analyzed. The input attachment tension load was based on the Hilti allowable. The ultimate capacity is also the Hilti value. Thus, the safety factor computed reflects the decrease from 4.0 due to plate flexibility and preload. These results are tabulated in Table 1, Column A. As can be seen, 6 plates had a safety factor less than 2.0 indicating that the bolt forces were amplified more than twice over rigid plate assumptions.

The safety factors were then recalculated using the original Phillips bolts as the basis for the input attachment load. The allowables were also reduced for spacing according to Reference 3. Likewise, the Hilti ultimate capacities were reduced for spacing in accordance with Reference 2. These results are tabulated in Column B of Table 1. Because the original plate sorting was not done with bolt spacing in mind, another sort had to be made and 12 additional plates were selected for analysis. Of the 34 plates, 21 had a safety factor less than 2.0. For the 21 plates with safety factors less than 2.0, the ultimate Hilti capacity was recalculated based on group action in accordance with Reference 4. These results are shown in Column C of Table 1. Of the 21 plates, 8 had safety factors less than 2.0.

For the 8 plates in Column C still having safety factors less than 2.0, the group action calculation was repeated using the actual bolt embedments recorded during installation. The safety factors are tabulated in Column D of Table 1. Of the 8, two have safety factors less than 2.0.

Reference 4 gave an indication that in certain cases, the pull-out cone angle might be taken as greater than 90°, citing a TVA paper, Reference 5. For the final 2 plates, the cone angle required for a safety factor of 2.0 was calculated. The angles are shown in Column E of Table 1. They are based on minimum embedment. Unfortunately, a review of Reference 5 shows it to be of no value in opening up the cone angle for the embedments being considered.

Six of the 34 tension cases were selected for moment load consideration. The results are shown in Table 2. The last two digits of the file name designation correspond to the file designation for the tension only case. Thus, plate FSS 102 in Table 2 is the same as FSS 2 in Table 1. Safety factors tabulated in Columns F, G, H and J were calculated using the same assumptions as Table 1, Columns B, C, D and E, respectively. Of the six, two have safety factors under 2.0.

As part of this study, flexibility magnification factors were calculated for potential use during later reanalysis. For the 34 tension cases and six moment cases, the load output from the finite element analysis was divided by the rigid plate load. These loads are listed in Tables 3 and 4. Since these plate were picked to envelope groups of the standard plates, it is possible that these factors can be conservatively used to hand calculate bolt loads from known pipe support reactions. However, since the baseplate analysis is non-linear, the effect of the applied load magnitude on the magnification factors needs to be investigated to assure nservatism.

CONCLUSION

Of the 777 baseplates on safety related supports, 478 met the "standard" criteria and were covered by this study. Of the non-standard plates, many can probably be enveloped by the cases considered herein. However, a review of these plates showed that a detailed grouping effort is required to verify this opinion and some additional cases will probably need to be analyzed. These analyses will be more time consuming since the EPLATE computer program cannot be used.

In summary, for the 478 standard baseplates, and quite probably many of the 299 non-standard plates, the effects of flexibility and preload will not reduce the factor of safety against bolt pull-out to lower than 2.0, assuming that the plates were properly designed originally using rigid plate assumptions with a safety factor of at least 4.0. It is our understanding that the two exceptions, FSS 14 and FSS 19, have been upgraded during the last plant refueling outage.

LIST OF REFERENCES

1. U.S. Nuclear Regulatory Commission, I.E. Bulletin No. 79-02, Revision No. 1 (Supplement No. 1), "Pipe Support Base Plate Designs using Concrete Expansion Anchor Bolts", August 20, 1979.

2. HILTI FASTENING SYSTEMS, Bulletin TR 111 "Summary Report - Kwik-Bolt Testing Program", January 1974.

3. ITT-PHILLIPS DRILL DIVISION, Catalog F-500 "Concrete Anchoring Handbook and Specifiers Guide", 1973 Edition.

4. AMERICAN CONCRETE INSTITUTE, ACI Journal - August 1978, Pages 329 to 347, "Proposed Addition to: Code Requirements for Nuclear Safety Related Structures (ACI 349-76)".

5. TENNESSEE VALLEY AUTHORITY - CIVIL ENGINEERING BRANCH, "Anchorage to Concrete" Research and Development Report No. CEB 75-32, December 1976.

6. EARTHQUAKE ENGINEERING SYSTEMS, INC., "EPLATE - A Nonlinear Finite Element Program for Analysis of Baseplates" Version 3.0, January 1980, Revised June 1980.

7. Mercury Company's <u>Concrete Expansion Anchor Removal and Replacement Installation Procedure</u>, Revision 4, September 25, 1979.

APPENDIX

GUIDE TO TABLE 1 (TENSION LOAD)

Values tabulated in columns A, B, C and D are safety factors which have been calculated on the basis of the assumptions stated below.

Column A

The tension load input to the finite element program was based upon allowable values(2) of pullout for HILTI "Kwik-Bolt" stud anchors at minimum embedment, with no reduction for close spacing.

Ultimate anchor pullout, for the safety factor calculation, was not reduced for closely spaced anchors. The anchors were assumed to be at minimum embedment.

Column B

PHILLIPS "Red Head" concrete anchor allowable pullout loads were the governing values in the computation of the tension load. Reduction of the allowable pullout for close spacing was on a linear basis and followed the recommendations in the PHILLIPS catalog(3).

Ultimate anchor pullout(2) was reduced on a straight-line basis for close spacing. Minimum embedment depth was assumed.

Column C

Required only if Column B is less than 2.0. Input to the finite element model was the same as for column B (PHILLIPS allowable loads).

Ultimate anchor pullout was calculated by estimation of the concrete strength(4) for anchors with minimum embedment depth.

Column D

Required only if Column C is less than 2.0. The tension load is the same as in column C.

Calculation of ultimate anchor capacity considered the strength of concrete for actual depth anchor embedments.

Column E

Required only if Column D is less than 2.0. Internal failure cone angle (DEG) required to give safety factor of 2.0 based on concrete strength at minimum embedment.

TABLE 1 (TENSION ONLY)

Case No.	Support No.	A	B	C	D	E
FSS 1 & 1A	HPCI-H63	3.09	3.38	NA	NA	NA
FSS 2	HPCI-H82	2.70	2.10	NA	NA	NA
FSS 3	CST-H40A	3.83	2.51	NA	NA	NA
FSS 4	RHR-H173	2.65	3.32	NA	NA	NA
FSS 5 & 5A	CS-H46	2.52	2.32	NA	NA	NA
FSS 6	RHR-HD200B	2.99	3.88	NA	NA	NA
FSS 7	HPCI-H82	3.30	3.21	NA	NA	NA
FSS 8	HPCI-HD254	3.17	3.91	NA	NA	NA
FSS 9	RHR-H162	2.02	1.76	1.64	3.15	NA
FSS 10	RCW-HD127	1.84	2.06	NA	NA	NA
FSS 11	CS-HD67C	2.20	1.37	2.62	NA	NA
FSS 12	RHR-H195	2.26	1.32	2.30	NA	NA
FSS 13	ACSP-H27	1.58	1.96	2.05	NA	NA
FSS 14	RSW-H201	1.04	1.35	1.14	1.34	109°
FSS 15	RCW-H107	1.24	1.07	1.67	1.99	96°
FSS 16	RSW-HD225B	2.30	3.45	NA	NA	NA
FSS 17	RHR-H185	3.33	*	2.54	NA	NA
FSS 18	RSW-H807	1.97	1.20	1.97	3.39	NA
FSS 19	CST-H15	3.04	0.30	2.00	NA	NA
FSS 20	HPCI-HD28	2.20	2.72	NA	NA	NA
FSS 21	RHR-H129	2.51	3.22	NA	NA	NA
FSS 22	RCW-H100	1.95	0.51	2.22	NA	NA

TABLE 1 (T NSION ONLY) continued

Case No.	Support No.	A	B	C	D	E
FSS 23	CST-H21	NA	1.69	3.19	NA	NA
FSS 24	CS-H64	NA	1.90	3.33	NA	NA
FSS 25	CS-H53	NA	2.22	NA	NA	NA
FSS 27	RSW-H181	NA	0.81	2.20	NA	NA
FSS 28	CUW-H31	NA	*	1.79	2.62	NA
FSS 29	CST-HD25	NA	*	2.01	NA	NA
FSS 31	CST-H67	NA	*	2.16	NA	NA
FSS 32	CUW-H31	NA	*	1.73	3.41	NA
FSS 33	HPCI-HD81C	NA	0.76	1.95	3.11	NA
FSS 34	RCW-HP115C	NA	1.12	2.79	NA	NA
FSS 35	RCW-H153	NA	*	1.39	2.01	NA
FSS 37	RHR-H197	NA	*	2.07	NA	NA

* much less than 2.0 by inspection

Note:

Cases 26, 30 and 36 are omitted as they are non-standard plates.

GUIDE TO THE USE OF TABLE 2 (MOMENT)

Values tabulated in columns F, G & H are safety factors calculated from the output data of the finite element computer model. Input for this program was calculated under the assumption that PHILLIPS "Red Head" concrete anchor(3) allowable pullout values controlled the value of the moment loading.

Ultimate anchor pullout was calculated on the basis of the assumptions stated below.

Column F

Ultimate anchor pullout(2) was reduced on a straight-line basis for close spacing. Minimum embedment depth was assumed.

Column G

Concrete strength (group action) determined ultimate pullout. Minimum embedment depth was assumed.

Column H

Actual embedment depth is used. Otherwise the assumptions are the same as in Column G.

Column J

Internal failure cone angle (DEG) required to give safety factor of 2.0 based on concrete strength at minimum embedment.

TABLE 2 (MOMENT)

File Name	Support No.	F	G	H	J
FSS 102	HPCI-H82	1.89	2.08	NA	NA
FSS 106	RHR-HD200B	2.66	NA	NA	NA
FSS 109	RHR-H162	1.43	1.72	3.32	NA
FSS 114	RSW-H201	0.08	0.68	0.76	124^{o}
FSS 119	CST-H15	0.23	1.70	1.27*	97^{o}
FSS 127	RSW-H181	0.79	2.20	NA	NA

* This is less than Column G because actual embedment is less than the minimum per Reference 7.

TABLE 3

TENSION LOAD MAGNIFICATION FACTORS

CASE NO.	RIGID P_L LOAD	FLEXIBLE P_L LOAD	FLEX/RIGID
FSS 1 & 1A	4.05 KIPS	6.243 KIPS	1.54
FSS 2	1.955 KIPS	3.59 KIPS	1.84
FSS 3	1.352 KIPS	2.14 KIPS	1.58
FSS 4	1.015 KIPS	1.955 KIPS	1.93
FSS 5 & 5A	4.406 KIPS	7.99 KIPS	1.81
FSS 6	2.125 KIPS	3.63 KIPS	1.71
FSS 7	2.125 KIPS	3.515 KIPS	1.65
FSS 8	2.125 KIPS	3.603 KIPS	1.70
FSS 9	2.125 KIPS	4.822 KIPS	2.27
FSS 10	2.125 KIPS	5.03 KIPS	2.37
FSS 11	3.74 KIPS	8.25 KIPS	2.21
FSS 12	4.46 KIPS	8.826 KIPS	1.98
FSS 13	3.708 KIPS	10.49 KIPS	2.83
FSS 14	2.125 KIPS	8.36 KIPS	3.93
FSS 15	4.072 KIPS	13.955 KIPS	3.43
FSS 16	4.46 KIPS	8.643 KIPS	1.94
FSS 17	4.46 KIPS	7.03 KIPS	1.58
FSS 18	2.125 KIPS	5.08 KIPS	2.39
FSS 19	3.36 KIPS	5.77 KIPS	1.72
FSS 20	1.42 KIPS	3.215 KIPS	2.26
FSS 21	4.46 KIPS	7.97 KIPS	1.79
FSS 22	4.46 KIPS	10.54 KIPS	2.36
FSS 23	3.74 KIPS	6.715 KIPS	1.80
FSS 24	3.88 KIPS	6.77 KIPS	1.74

TABLE 3 (continued)

TENSION LOAD MAGNIFICATION FACTORS

CASE NO.	RIGID P_L LOAD	FLEXIBLE P_L LOAD	FLEX/RIGID
FSS 25	4.017 KIPS	6.464 KIPS	1.61
FSS 26	NA	NA	NA
FSS 27	1.806 KIPS	3.42 KIPS	1.89
FSS 28	1.881 KIPS	3.44 KIPS	1.83
FSS 29	3.59 KIPS	5.644 KIPS	1.57
FSS 30	NA	NA	NA
FSS 31	3.69 KIPS	6.80 KIPS	1.84
FSS 32	1.72 KIPS	3.391 KIPS	1.97
FSS 33	1.998 KIPS	3.388 KIPS	1.70
FSS 34	3.57 KIPS	6.671 KIPS	1.87
FSS 35	3.708 KIPS	8.851 KIPS	2.39
FSS 36	NA	NA	NA
FSS 37	3.626 KIPS	6.921 KIPS	1.91

TABLE 4

MOMENT LOAD MAGNIFICATION FACTORS

CASE NO.	RIGID P_L LOAD	FLEXIBLE P_L LOAD	FLEX/RIGID
FSS 102	1.955 KIPS	3.985 KIPS	2.04
FSS 106	2.125 KIPS	5.292 KIPS	2.49
FSS 109	2.125 KIPS	5.923 KIPS	2.79
FSS 114	2.125 KIPS	14.06 KIPS	6.62
FSS 119	3.36 KIPS	7.509 KIPS	2.23
FSS 127	1.806 KIPS	3.492 KIPS	1.93

NOTE: It should be noted that piping overstress will not be considered a problem because of the conservative ARS we are using for the analysis. Pipe supports though, will be modified if necessary, to account for higher support reactions.

The design criteria for pipe supports is presently not available. The basic code used in this evaluation will be AISC with no increase for SSE condition.