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Bechtel

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October 21, 1991

Mr. Brian E. Holian
Project Manager
Project Directorate III-1
Division of Reactor Projects III/IV/V
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Dear Mr. Holian:

Subject: Palisades Anchor Bolt Design

Reference: Brian E. Holian letter to D. C. Kansal dated
June 13, 1991, "Palisades Anchor Bolt Design",
Docket No. 50-255

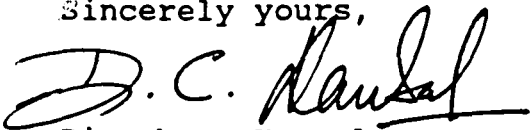
Enclosed is Bechtel's response to the open issues and questions
addressed in the reference, concerning our anchor bolt design work at
Palisades plant.

To provide additional assurance that the approach used by Bechtel for
the design of the Palisades anchor bolts was technically acceptable,
the enclosed response was reviewed by Dr. Edwin Burdette, Professor
of Civil Engineering, University of Tennessee, and Dr. Richard
Klingner, Professor of Civil Engineering, University of Texas.
Drs. Burdette and Klingner have extensive experience with, and are
recognized nationally as experts in the testing and evaluation of
concrete anchors, and both concurred with the enclosed response.

Also, the attached response has been reviewed and concurred with by
Consumers Power Company.

Should you have any questions or need additional information, please
feel free to contact me at (301) 417-3777.

Sincerely yours,



Dinesh C. Kansal
Manager of Quality Assurance

DCK/ajj

Enclosures

cc: D. Joos, CPCo, w/a
Dr. E. G. Burdette, w/a
Dr. R. E. Klinger, w/a



Bechtel Power Corporation A unit of Bechtel Corporation

At Encl
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October 21, 1991

**RESPONSE TO
JUNE 13 1991 NRC LETTER TO BECHTEL
REGARDING
PALISADES ANCHOR BOLT DESIGN**

1.0 INTRODUCTION

The NRC letter of June 13, 1991 (Reference 1) discusses Bechtel's evaluation of anchor bolts at the Palisades Nuclear Power Plant. Specifically, the letter is concerned with Drillco Maxi-bolts in close proximity to Hilti Kwik Bolts such that the concrete stress cones of the two types of bolts overlap. The letter identifies two issues, which, in the opinion of the NRC staff, need additional validation. These two issues are:

1. The concrete holding capacity of Hilti Kwik Bolts is not reduced even though they are closely spaced to the Maxi-bolts and their stress cones overlap with those of the Maxi-bolts.
2. The Drillco Maxi-bolts behave as ductile anchors.

These issues arise out of methodology used in calculations prepared primarily to evaluate the preservation of the ductile behavior of Maxi-bolts when located in proximity to non-ductile (ND) anchors such as Hilti Kwik bolts. The capacity of each Maxi-bolt group and ND anchor group to independently carry their design loads had been previously established.

The anchor bolts in question were evaluated initially by a simplified approach (previously reviewed by the NRC during the Region III inspection of the Palisades Steam Generator Replacement Project, Reference 2).

In preparation for this response, a more accurate detailed approach was developed and used to reevaluate all previous applications. Both methods produced the same conclusions (i.e. the Maxi-bolt installations remain ductile and ND anchors sustain no additional penalty).

The methodology and our response was reviewed by Dr. Edwin Burdette, Professor of Civil Engineering at the University of

Tennessee, and Dr. Richard Klingner, Professor of Civil Engineering at the University of Texas. Both professors are nationally recognized experts in the testing and evaluation of concrete. They both concur with our approach and our logic.

The rationale behind the methodology used and pertinent background information are discussed in Sections 2.0 and 3.0. The two NRC issues identified above are addressed specifically in Section 4.0. The extent of application of these methods is given in Section 5.0

2.0 BACKGROUND

For purposes of this discussion the ultimate tensile capacity of an anchor is controlled by one of two major modes of failure; steel related failure (bolt yielding) or concrete failure (cone pullout). A third failure mode associated with expansion anchors is bolt slip (which can affect ultimate capacities for various embedment depths) is also possible but does not directly affect the two specific issues of concern to the NRC staff. According to the nomenclature given in Appendix B of ACI 349 (Reference 3), anchors controlled by steel capacity are termed "ductile" and those governed by concrete capacity are considered "non-ductile". To ensure a ductile failure, a sufficient volume of concrete must be engaged by the anchor to provide a concrete capacity in excess of the force required to yield the anchor bolt. The Maxi-bolts satisfying embedment criteria were classified as ductile anchors. Other adjacent anchors were considered to be non-ductile.

The primary purpose for performing the calculations referenced in the June 13, 1991 NRC letter was to ascertain that sufficient concrete anchorage for the Maxi-bolts was provided to assure ductile behavior, acknowledging the close proximity of non-ductile anchors. To evaluate the interaction of ductile and non-ductile anchors, the criteria included in ACI 349, Appendix B was employed. ACI 349 provides direction in the design of anchor bolts which represents methodology developed on an industry wide basis, including input from NRC staff members.

The ACI 349 criteria indicate that the concrete capacity for a single anchor bolt can be represented by the development of tension stresses over the projected area of a pullout cone in the concrete as shown in Figure 1. This results in a structural model that simulates the concrete rupture mode at ultimate capacity. Using this model, the concrete capacity can be determined by multiplying the projected area of the concrete cone by an appropriate conservative estimate of the ultimate tensile (pullout) strength of the concrete, as specified in ACI 349, Appendix B. The adequacy of this model has been confirmed by test in many applications. Two sets of test data are summarized in Attachment 1. In both sets, data are compared as the ratio of

the anchorage capacity computed by ACI 349 methods with the average test results for those cases where concrete was the mode of failure. Similar comparisons utilizing other test data are contained in Reference 4.

ACI 349 also acknowledges that when two or more anchors have their cones intersect, the total concrete capacity of the group can be conservatively represented by the combined projected area of the cones at the concrete surface as shown in Figure 2, multiplied by the specified concrete pullout stress. The data included in Attachment 2 (which was also included as an attachment to the June 13, 1991 NRC letter) support this concept. All of the data presented in this attachment are for four-bolt patterns with center-to-center spacings that cause significant intersection of the individual cones. Computed concrete anchorage capacity was determined by the ACI 349 method for multiple anchors. Note that the ratio of the actual failure load to the computed concrete capacity for those cases where failure was ultimately in the concrete (failure type 2 in column 9) ranged from 1.0 to 1.16 times the actual failure load. This indicates excellent correlation (which is properly on the conservative side) between calculated and actual concrete capacity.

In a group of anchors with the same embedment depth, the resulting total projected area of intersecting cones is less than the sum of the individual projected areas of the cones. This results in a reduction in total concrete capacity. To maintain the same concrete capacity, it is necessary to embed the anchors with intersecting anchorage cones to a greater depth. The major purpose of the calculations referenced in the NRC letter was to establish that the required embedment was provided.

A common practice in the industry for determining the individual capacity of an anchor bolt affected by an intersecting cone from another anchor is to proportion the intersected area in the manner shown in Figure 3. This practice was employed in the Palisades anchor bolt calculations where anchorage cones from non-ductile anchors intersect other anchorage cones from non-

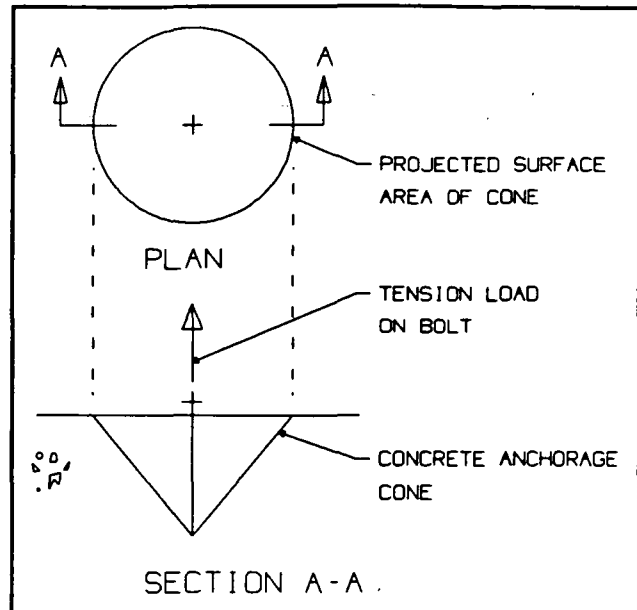


Figure 1
CONCRETE ANCHORAGE CONE FOR A
SINGLE ANCHOR

ductile anchors. However, this approach is appropriate, and was used, only for applications where the adjacent anchor bolts have similar failure modes.

3.0 EVALUATION APPROACH

Evaluation of anchorages involving overlapping ductile and non-ductile anchor stress cones using conventional procedures such as the area proportioning method described in Section 2.0 for determining individual anchor capacities would be inappropriate and could be unconservative. Therefore, a methodology for evaluation of this type of anchorage was developed. It specifically addresses the behavior differences associated with these two failure modes.

The ultimate capacities of both the ductile (Maxi-bolts) and non-ductile anchor(s) with respect to the concrete anchorage capacity is properly accounted for in assessing the retention of Maxi-bolt ductile behavior.

Confirmation of necessary load capacities of both the ductile and non-ductile anchor(s) is provided.

3.1 Detailed Approach

The detailed approach for evaluation of anchorage configurations with overlapping ductile and non-ductile anchor stress cones is summarized as follows:

1. Ascertain that the ductile anchor group satisfies load requirements and ductile anchorage criteria without interference from the adjacent non-ductile anchors.
2. Ascertain that the non-ductile anchor group likewise satisfies load requirements (with due consideration of

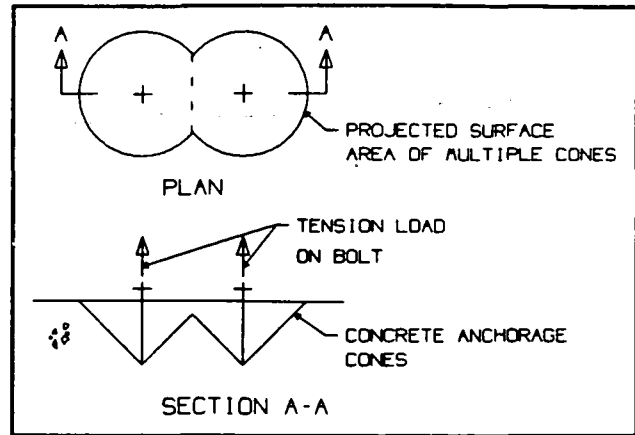


Figure 2
CONCRETE CONE FOR MULTIPLE ANCHORS

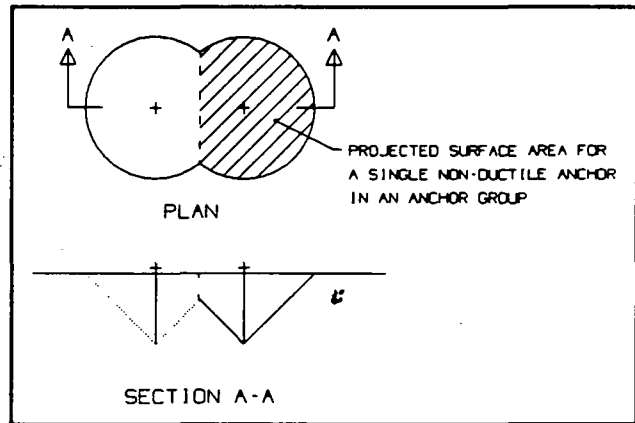


Figure 3
CONCRETE CONE AREA FOR A SINGLE ANCHOR IN A NON-DUCTILE GROUP

any overlap of non-ductile anchor stress cones) without interference from the adjacent ductile anchors.

3. Compute the combined load capacity that can be applied to the concrete by both the ductile and the non-ductile anchors:

$$P_{dn} = A_{ts} f_{ut} + P_{na} \quad \text{Eq. 1}$$

where:

- P_{dn} = the load capacity of the ductile and non-ductile anchors
- A_{ts} = the effective tensile stress area of the ductile anchors
- f_{ut} = the minimum specified steel tensile strength of the ductile anchors
- P_{na} = the ultimate tensile capacity of the non-ductile anchors (as determined by tests)

4. Compute the concrete capacity (reduced by an understrength factor) of the combined projected cone area:

$$\phi P_{nc} = 4 \phi \sqrt{f'_c} A_{nc} \quad \text{Eq. 2}$$

where:

- ϕ = the understrength factor for concrete cone capacity (a conservative value of ϕ of 0.65 from ACI 349 was used)
- P_{nc} = the anchor capacity as governed by cone failure
- f'_c = concrete compressive strength
- A_{nc} = the enveloped projected area of the overlapping cones (combined projected area of both cones, see Figure 4).

5. Check for ductile behavior of the combined anchorage. To ensure ductile behavior of the combined anchorage, the following relationship must be satisfied:

$$4 \phi \sqrt{f'_c} A_{nc} > A_{ts} f_{ut} + P_{na} \quad \text{Eq. 3}$$

Equation 3 requires that the concrete capacity for the enveloped projected area of both Maxi-bolt(s) and non-ductile anchor(s) exceeds the total combined load capacity of the Maxi-bolts and

non-ductile anchors in question. This ensures that failure by pull-out of the combined volume of concrete as a unit is precluded until after inelastic ductile behavior has been achieved in the ductile portion of the anchorage. This is precisely the design requirement of ACI 349, Appendix B for the design of ductile multi-anchor attachments subjected to cone overlap.

The conservative aspects of this approach are summarized as follows:

1. Loads from all anchors are assumed to be maximized at the same time (regardless of actual imposed loading).
2. Reduction in concrete anchorage capacity due to cone overlap is shared by both the Maxi-bolt and the non-ductile anchor.
3. The sum of the individual test-determined ultimate load capacities of the non-ductile anchors (unreduced by any overlapping non-ductile anchor cones) is added directly to the required pull-out capacity associated with the ductile anchors.
4. If earlier failure were to occur locally in the non-ductile anchor group (such as from non-ductile anchor cone overlap) the load on the combined area would be less than that assumed in the analysis.
5. Steps 1 and 2 provide assurance that each anchor group can satisfy load requirements and other design criteria without help (or hindrance) from the adjacent anchor group.

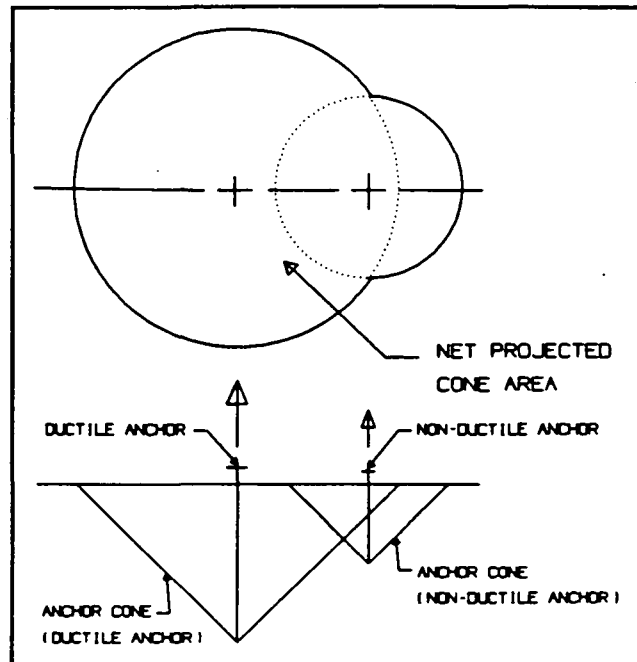


Figure 4
EXAMPLE OF NET CONE AREA FOR
DETAILED METHOD

3.2 Simplified Approach

The 'simplified approach' utilizes the same basic approach as the 'detailed approach' except that the effective projected concrete cone area available to the ductile anchors (Maxi-bolts) for concrete anchorage is reduced by deducting the entire overlapped projected cone area associated with the non-ductile anchors.

Ductile behavior is assured by satisfying the following relationship:

$$4 \phi \sqrt{f_c'} A_{net} > A_{ts} f_{ut} \quad \text{Eq. 4}$$

A_{net} = the projected area of the Maxi-bolt not shared with the non-ductile anchor (see Figure 5).

Equation 4 can be rewritten to enable direct comparison with Equation 3. Substituting $A_{net} = A_{nc} - A_{non}$ and rearranging terms:

$$4 \phi \sqrt{f_c'} (A_{nc} - A_{non}) > A_{ts} f_{ut} \quad \text{Eq. 5}$$

Rearranging Equation 5,

$$4 \phi \sqrt{f_c'} A_{nc} > A_{ts} f_{ut} + 4 \phi \sqrt{f_c'} A_{non} \quad \text{Eq. 6}$$

A_{non} = the projected surface area of the non-ductile anchors.

Equation 3 (used in the more precise detailed approach) and Equation 6 (simplified approach) differ only in the last term of each equation. The degree of conservatism between Equations 3

and 6 depends on the relative values of P_{ns} and $4 \phi \sqrt{f_c'} A_{non}$.

For the anchorages evaluated, both methods yielded the same results. The numerical differences between the two methods were small with no consistent pattern regarding relative conservatism. The simplified approach is, however, expected to be more conservative for the more deeply embedded non-ductile anchors (because of anchor slip associated with the ultimate strength of the anchor).

3.3 Analysis Summary

The anchor bolts in question (referred to in Reference 1) were initially evaluated using the simplified approach as described in

Section 3.2. They were again analyzed using the more detailed approach described in Section 3.1 (except for 3 cases where very small applied loads permitted obvious qualification based on non-ductile anchor criteria).

Reevaluation by the more detailed approach required identifying the types of non-ductile anchors, their embedment depths and their associated ultimate strengths. The ultimate capacities of the non-ductile anchor bolts were defined using the average test determined values summarized in the manufacturers' catalogs (References 5 through 7). In one case the concrete stress cones from an embedded Unistrut anchorage overlapped Maxi-bolt stress cones. In

this case, the Unistrut anchorage was treated as non-ductile with an ultimate capacity anchorage based on the minimum specified tensile strength of the individual steel anchors as defined in Reference 8. The anchor type and embedment were determined from design documents and field walk-down data. Where the actual embedment depths were not well documented, all embedment lengths for which published test data were available were evaluated.

The results of the evaluations using the detailed approach were identical to those using the simplified approach. The Maxi-bolt anchors in all cases were classified as 'ductile anchors' except for the three cases with very low loads that were evaluated as non-ductile anchors. In all cases the anchors were determined to have sufficient capacity to sustain the imposed loading without exceeding design limits.

4.0 NRC ISSUES

The following sections specifically address the two primary NRC issues raised in their June 13 letter (Reference 1).

4.1 Capacity of Hilti Kwik-bolts

It is stated in Reference 1 that "Bechtel needs to demonstrate that the concrete holding capacity of the Hilti Kwik-bolts would not be reduced when they are closely located to the Drillco Maxi-bolts...". For the case cited (with appended calculation

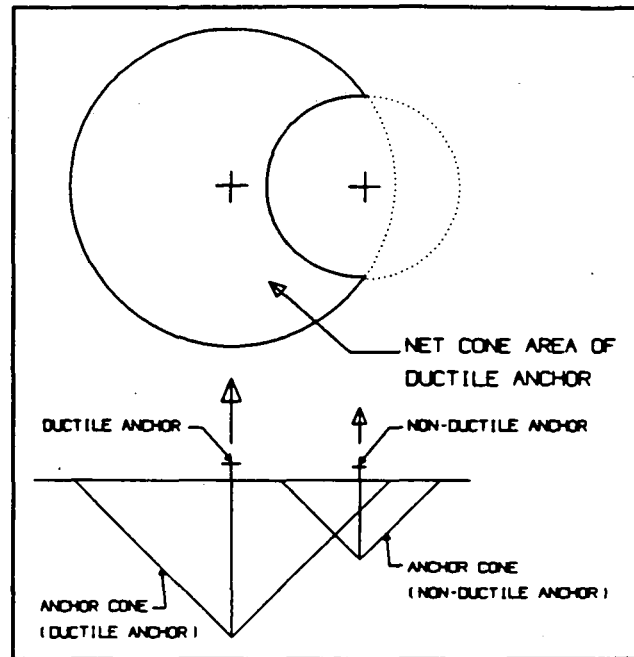


Figure 5
EXAMPLE OF DUCTILE ANCHOR NET
CONE AREA FOR SIMPLIFIED METHOD

sheets), the determination of the Kwik-bolt capacity penalties is summarized as follows:

1. The projected stress cones of the Kwik-bolts do not overlap each other. Therefore, no capacity reduction is required using the conventional methods as described in Section 2.0.
2. The Kwik-bolts satisfy design load requirements without interference from adjacent ductile anchors (Section 3.1, step 2). This precludes premature local concrete failure of the non-ductile anchors.
3. The combined Maxi-bolt/Kwik-bolt anchorage is classified as 'ductile' in accordance with the conservative methodology described in Section 3.1. This precludes premature concrete failure of the combined anchorage as a unit.

The effect of the overlapping ductile and non-ductile stress cones is compensated for by the anchor bolt embedment provided. Therefore, no reduction in the Kwik-bolt capacity is required to ensure acceptable anchorage capacities.

If, however, the anchorage classification were determined to be 'non-ductile', appropriate capacity reductions would be required using conventional methods as described in Section 2.0.

4.2. Ductility of Maxi-bolts

The methodology and supporting rationale for classifying an anchor as 'ductile' or 'non-ductile' are presented and discussed in detail in Section 3.0. However, criteria defining ductility merits further discussion.

In the June 13 NRC letter it was stated that, for an anchor to be ductile implies that "the steel anchor will fail and the concrete that holds the anchor will not". We do not fully concur with that description but believe there may be some confusion associated with terminology (e.g., the term "ductile failure" is commonly used in lieu of more precise terminology such as "ductile behavior prior to failure").

The purpose of using "ductile" anchors is primarily to provide for conditions that may induce strains on the anchor which may not be completely captured in the design. Examples include strains induced in the anchor due to unexpected thermal displacements, and anchor strain associated with connection deformation/rotation resulting from boundary condition variations between analysis and the actual behavior of the connection. However, the important issue is that the bolts have the

capability to strain in excess of their design loads without losing their ability to transfer their design load.

Provisions for deformation beyond that identified specifically in the analysis are not limited to ductile anchor bolts. Common design practice, as delineated in building codes, is to provide ductility in the overall structure for similar reasons. This is accomplished in steel structures by defining capacities of steel structural elements with respect to the steel yield stress. This principle is also used for concrete structures. For example, concrete beams (per the ACI 318 and ACI 349 codes) are proportioned such that the reinforcing steel ratio is limited to 75 percent of the ratio producing "balanced conditions" (where simultaneous yielding of steel and crushing of concrete would occur). This insures that the reinforcing steel will yield producing ductile deformation of the beam prior to the final sudden compression failure of the concrete.

Most types of steel exhibit considerable inelastic strain deformation beyond their yield stress limit. Displacements many times the elastic yield displacement typically occur with little or no corresponding increase in stress. This provides the basic mechanism for ductile behavior of steel elements. The steel elements can sustain further inelastic displacements without rupture (but with nominal stress increase - strain hardening) until the ultimate strength of the steel is reached.

The actual useful load limit of an anchor bolt is below the yield limit. The key to designing a ductile anchor is to assure that the bolt can reach its yield stress limit before failure of the anchorage can occur by any other mechanism. Similar to the provisions for design of ductile concrete beams, the mechanism by which the ductile anchor bolt system ultimately ruptures, whether it be due to concrete or steel, is of less significance, providing the mechanism for ductile behavior by yielding the anchor bolt is preserved.

Standard practice in the procurement of steel is to identify strength properties in terms of minimum specified yield stress, and minimum specified ultimate tensile strength. Experience has shown that the actual yield stress is typically less than the minimum specified ultimate tensile strength. The selection of the minimum ultimate tensile strength combined with conservative stress levels to establish concrete capacity provides assurance that the strains associated with the actual yielding of the bolt material can be obtained. Bechtel used this approach in the evaluation of the Palisades anchors referenced in the NRC letter dated June 13, 1991. Further discussion regarding the selection of the minimum ultimate tensile strength is found in the ACI 349 commentary, included herein as Attachment 3.

The minimum specified yield stress of a Maxi-bolt is 105 ksi and the minimum ultimate tensile strength is 125 ksi. This suggests that the actual yield stress is expected to typically lie somewhere between these two limits. The test data provided as an attachment to the NRC letter indicate that the ruptures of the ductile anchor bolt systems all occurred at stresses in excess of 146 ksi, confirming that the concrete strength was sufficient in all cases to develop the ductile behavior of the bolt.

5.0 EXTENT OF APPLICATION

The specific approach employed in the evaluation of the cases referenced in the June 13 NRC letter for Palisades was uniquely developed by Palisades project personnel solely for that application and to the best of our knowledge is an isolated case. More specifically, the practice of severely penalizing certain anchors within a group by assuming other non-ductile anchor bolts maintain their full capacity is believed to be unique to the Palisades application. There is no Bechtel standard that specifies this approach or that either suggests or imposes that the concrete holding capacity of non-ductile anchors not be reduced when they are closely spaced to the ductile anchors.

Regarding the second NRC concern, Bechtel Design Guide C-2.40, Rev. 1, Section 3.2 indicates that those anchors which meet the appropriate requirements of ACI 349 may be qualified as ductile expansion anchors.

The definition of ductile anchors, as clarified above, and the basic criteria to develop concrete capacity of single and multiple anchors as provided in ACI 349, Appendix B, have been used both by Bechtel on many plants, and according to our understanding, by most other architects/engineers, engineering consultants and utilities. ACI 349 Appendix B was developed by a group of anchorage experts from various architect/engineering firms, utilities, academia, and the NRC staff. ACI 349 Appendix B represents the only nationally recognized code on this subject and it is our understanding that it is receiving wide usage by sources both within and outside Bechtel design control.

6.0 SUMMARY

The two issues identified in the June 13 NRC letter relating to the anchor bolt evaluation for Palisades have been addressed. In summary the following has been resolved:

1. Two structural models (one simplified and the other more detailed) based on principles of mechanics and the guidance of Appendix B to ACI 349, were employed to determine the interaction between ductile and

non-ductile anchors. For those cases evaluated with these models both the ductile and non-ductile anchors were shown to maintain their basic full tensile capacity. Although the simplified method was used in the calculations, the more precise detailed method produced the same final conclusion in all cases.

2. Ductile behavior for the Maxi-bolts is based on a structural yielding of the anchor bolt (associated with the actual yield stress of the bolt), and not on the actual ultimate failure strength of the bolt. This provision for ductile behavior is consistent with ACI 349, Appendix B for concrete anchorages (as well as for both ACI 318 and ACI 349 provisions for beams) and provides the necessary allowance for additional strain required to exhibit ductility in the anchorage system.
3. Bechtel's design approach in this case was consistent with good structural engineering practice. Bechtel performed their design and evaluation by developing and utilizing analytical models which were based on accepted principles of engineering mechanics and supported by a body of test data. Bechtel further employed criteria established in nationally recognized codes to carry out their design and evaluation. Simplification was employed in the application of the methodology as discussed above, but further evaluation indicates that the simplification produced conclusions consistent with the more precise methodology. The results of the analysis indicate that the anchorages are acceptable and adequate conservatism exists in the design.

REFERENCES:

1. B. E. Holian (NRC), "Palisades Anchor Bolt Design", Letter to D. C. Kansel (Bechtel) Dated June 13, 1991.
2. H. S. Miller (NRC), Letter to Gerald B. Slade (Consumers Power) Dated May 24, 1991.
3. Code Requirements for Nuclear Safety Related Concrete Structures, ACI 349-85, American Concrete Institute, September 1, 1985.
4. Klingner, R. E. and Mendonca, J. A. "Tensile Capacity of Short Anchor Bolts and Welded Studs: A Literature Review," Journal of the American Concrete Institute, Proceedings Vol. 79, No. 4, July-August 1982, pp. 270-279.

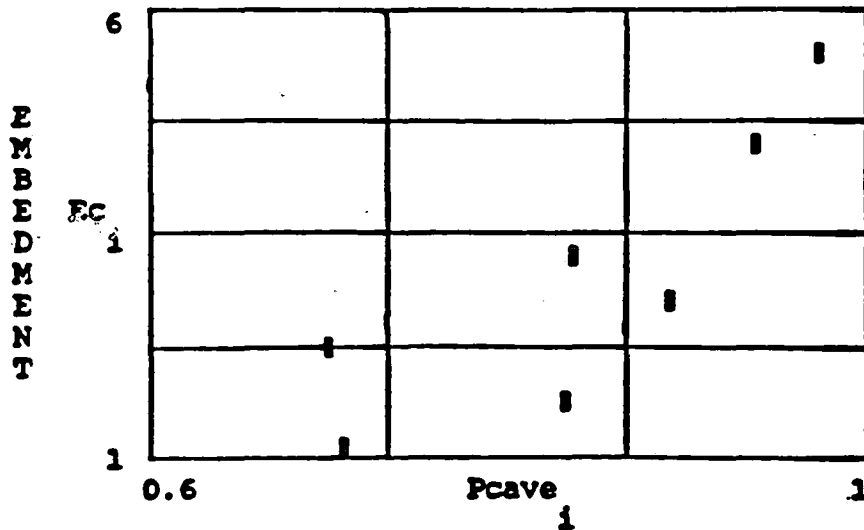
5. Maxi-bolt Anchoring System, Drillco Devices, Ltd.
6. Hilti Kwik-bolt Average Ultimate Tensile and Shear Loads, File No. H2189-S1, Report No. 8783 R, Abbot A. Hanks Testing Lab., March 24, 1977.
7. Hilti Fastening Technical Guide, H437B--9/90, Hilti Inc., 1990.
8. Unistrut General Engineering Catalog No. 10R, Unistrut Corp., 1986.

SUMMARY: HILTI KWIK-BOLTS

ANCHOR SIZE Sc	EMBED Ec	MEAN ULTIMATE STRENGTH			STRENGTH RATIO Pcave
		fc=2000 Tc	fc=4000 Tc	fc=6000 Tc	
0.25	1.125	975	1455	1755	0.708
0.375	1.625	2245	2355	2810	0.833
0.5	2.25	4545	5510	6845	0.699
0.625	2.75	5410	6600	7700	0.892
0.75	3.25	8155	10150	10860	0.838
1	4.5	14000	16000	20500	0.939
1.25	5.5	19000	23000	31200	0.973

Bolt data are from Reference 1. The Strength Ratio, Pcave, is the average ratio of the Calculated Cone-out Strength (per ACI Code) vs Test Cone-out Strength for each bolt size for concrete strengths from 2000 to 6000 psi. These ratios would be somewhat smaller if the bolt slip before cone-out were accounted for. The slip data were not available and, therefore, this adjustment could not be made.

1. Summary Reoprt - KWIK-BOLT Testing Program, Abbot A. Harks, Inc. Testing Laboratories, San Francisco Ca., March 24, 1977.



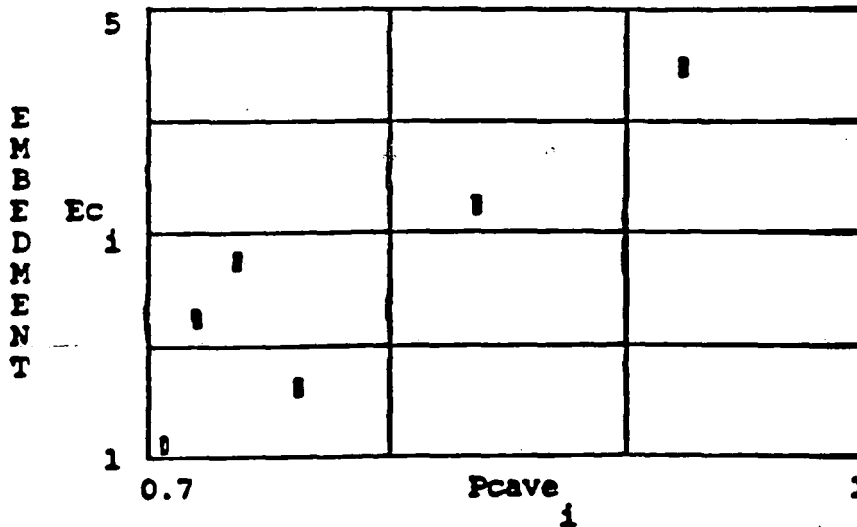
RATIO - ACI CONE-OUT CAPACITY to TEST CAPACITY

SUMMARY: HILTI-KWIK-II (HKB-II) BOLTS

ANCHOR SIZE Sc	EMBED Ec	MEAN ULTIMATE STRENGTH			STRENGTH RATIO Pcave
		fc=2000 Tc	fc=4000 Tc	fc=6000 Tc	
0.25	1.125	1000	1430	1750	0.707
0.375	1.625	2000	2825	3200	0.762
0.5	2.25	4400	5450	6500	0.721
0.625	2.75	6000	8000	10000	0.737
0.75	3.25	7400	10000	12000	0.837
1	4.5	12500	17500	21000	0.924

Bolt data are from Reference 1. The Strength Ratio, Pcave, is the average ratio of the Calculated Cone-out Strength (per ACI Code) vs Test Cone-out Strength for each bolt size for concrete strengths from 2000 to 6000 psi. These ratios would be somewhat smaller if the bolt slip before cone-out were accounted for. The slip data were not available and, therefore, this adjustment could not be made.

1. HILTI Fastening Technical Guide, HILTI Corp., Tulsa, Oklahoma, 1990



RATIO - ACI CONE-OUT CAPACITY to TEST CAPACITY

1 Test #	2 Anchor Diameter (Inches)	3 Number of Bolts in Pattern	4 Center to Center Spacing (Inches)	5 Intended Embed (Inches)	6 Actual Embed (Inches)	7 Area of Concrete Based on Intended Embed (Sq. In.)	8 Area of Concrete Based on Actual Embed (Sq. In.)	9 Failure Type	10 Failure Load (Kips)	11 Failure Stress (KSI)	12 Phi at Maximum Load	13 Phi at 125 KSI Based on Intended Embed	14 Phi at 125 K Based on Actual Embed
105	3/8	4	4	4 1/2	4 5/8	162.26	168.15	2	47.815	153.25	1.09	0.92	0.89
106	3/8	4	4	4 1/2	4 5/8	162.26	168.15	2	45.736	146.59	1.04	0.92	0.89
110	5/8	4	4	9	8 15/16	444.23	439.52	2	116.760	129.16	1.02	0.97	0.98
111	5/8	4	4	9	8 15/16	444.23	439.52	2	120.930	133.77	1.05	0.97	0.98
113	3/4	4	4	11 1/2	11 5/16	660.32	643.22	2	187.650	140.46	1.12	0.97	0.99
114	3/4	4	4	11 1/2	11 1/4	660.32	637.56	2	181.395	135.78	1.09	0.97	1.00
119	1/2	4	4	6	6 1/16	241.25	244.77	2	70.682	124.44	1.10	1.13	1.11
122	1/2	4	4	6	6 1/16	241.25	244.77	2	66.720	117.47	1.04	1.13	1.11
120	5/8	4	4	7 1/2	7 7/16	337.99	333.87	2	89.655	99.18	1.03	1.28	1.29
123	5/8	4	4	7 1/2	7 7/16	337.99	333.87	2	87.570	96.87	1.00	1.28	1.29
121	3/4	4	4	9 1/4	8 7/8	469.65	440.96	2	129.270	96.76	1.12	1.36	1.45
124	3/4	4	4	9 1/4	8 15/16	469.65	445.68	2	135.525	101.44	1.16	1.36	1.43
101	1/4	4	3	2 3/4	2 7/8	70.93	74.90	1	19.750	154.30	1.01	0.86	0.82
102	1/4	4	3	2 3/4	2 7/8	70.93	74.90	1	19.750	154.30	1.01	0.86	0.82
107	1/2	4	3	8	7 15/16	326.41	322.39	1	94.868	167.02	1.13	0.83	0.84
108	1/2	4	3	8	7 15/16	326.41	322.39	1	93.829	165.19	1.11	0.83	0.84
109	1/2	4	3	10	9 15/16	468.07	463.26	1	91.740	161.51	0.76	0.58	0.59
112	5/8	4	4	12	12	698.96	698.96	1	143.865	159.14	0.79	0.62	0.62
116	1	2	5 1/2	12 1/2	12 3/8	698.00	686.21	1	177.225	146.23	0.99	0.83	0.84
104	3/8	4	4	4 1/2	4 5/8	162.26	168.15	3	47.815	153.25	1.09	0.92	0.89
115	3/4	4	4	8 3/4	14 1/2	991.77	963.92	3	204.330	152.94	0.81	0.64	0.66
117	1	2	5 1/2	12 1/2	12 1/2	698.00	698.00	3	175.140	144.51	0.96	0.83	0.83
103	1/4	Not	Tested										
118	1	Not	Tested										

- Notes: 1. Column 9 Failure Type
1 = Steel Failure
2 = Concrete Cone Failure
3 = Maximum Load Applied with No Failure
2. Intended Embed = Theoretical Embedment Based on Anchor Sleeve Length
3. Actual Embed = Anchor Overall Length - Measured Stud Bolt Projection
4. Compressive Strength of Concrete at Time of Tests = 4270 psi
5. Column 12 = Failure Load / (4 * $\sqrt{4270}$ * Actual Area of Concrete)
6. Column 13 = (Area of Steel * 125,000 psi) / (4 * $\sqrt{4270}$ * Intended Area of Concrete)
7. Column 14 = (Area of Steel * 125,000 psi) / (4 * $\sqrt{4270}$ * Actual Area of Concrete)

Taken from Perry Nuclear Power Plant
Close Spacing Tests - Drillco Maxi-Bolts
(Ductile Anchors) by Gilbert Commonwealth.

Attachment 3

The Commentary to ACI 349, Appendix B discusses the use of the minimum specified tensile strength to assure ductile behavior as follows:

"The requirement ... that the design pullout strength of concrete exceed the minimum specified tensile strength of the steel is to assure ductile behavior of the embedment in the event of overload. Typical embedment steels exhibit significant variations in actual yield strength as compared to the minimum specified. In addition, the ratio of tensile strength to yield strength may vary from approximately 2 to 1.15. Considering these variations, it is necessary to specify design pullout strength of concrete as a function of f_u rather than a factor times f_y to insure ductility.