April 15, 1986
Enclosure

EVALUATION OF PVNGS MASONRY WALLS

Background

A limited number of masonry walls have been used in the PVNGS Auxiliary and Control buildings as non-structural, non-bearing walls. These walls are subject to lateral seismic inertial loads due to their own weight and the weight of light attachments.

Several questions were raised regarding the adequacy of these walls as a result of a recent NRC audit. Meetings have been held with the NRC to discuss these issues on March 20-21 and April 11, 1986. There have been telephone calls to provide additional data on the evaluation performed and to bring in the perspective of the Chairman of the ACI 531 Code Committee.

Specific issues related to the design of the masonry walls have been discussed during the meetings and the telephone conversations. This document is intended to summarize the significant attributes of the issues involved and the results of the evaluation performed.

Attachment A provides the justification for the following three important items:

(A) Use of full (special inspection) allowable stresses.

(B) Effectiveness of staggered reinforcement splices.

(C) Strength of reinforcement with small lap length.

It also indicates concurrence by the Chairman of the ACI 531 Code Committee on the justifications.

Attachment B compares the required and available documentation on and inspection of the masonry walls and concludes, based on the extensive evidence available, that the as-built walls are sound and the use of special inspection allowables is valid.

Attachment C contains the results of recent structural evaluations to demonstrate the integrity of the walls in serving their intended functions.

Attachment D discusses the applicability of test data on bond strength and on effectiveness of staggered splices in the walls.

Structural Evaluations

The walls were evaluated to determine the stresses under the applicable loading conditions. The ACI 318 and 531 Code specified E and I values were used in conjunction with the working stress design method in this evaluation. Analytical models consisted of one-foot strips of the walls with hinged-hinged

boundary condition at elevation 74' and fixed-hinged conditions at elevation 100'. Reinforcement locations, used in the calculations are based on average "d" distance from an available Non Conformance Report (NCR's).

The out-of-plane bending of a wall under a horizontal seismic excitation was calculated by the response spectrum method of analysis. Damping ratios of 7% and 4% were used for SSE and OBE conditions respectively. Axial loads in a wall from dead load and vertical seismic excitation were included.

Attachment C shows the critical stresses in the walls of the Control Building. Results for the Auxiliary Building are not included since no Category I item is attached or is adjacent to these walls.

Under SSE conditions, the maximum calculated masonry, reinforcement, and bond stresses are all within the allowables based on the ACI 531-79 Code as modified by the SRP 3.8.4. The maximum flexural compression fibre stresses in the masonry exceed the reduced allowables (no special inspection case) at elevation 100' by about 5% and at elevation 74' by about 16%.

Under OBE conditions, all the walls satisfy the bond stress allowables. The maximum reinforcement stress exceeds the allowable at elevation 74' by about 8% (less than 50% of yield), and the maximum flexural compressive fibre stresses, although above code allowables in some cases, are approximately one-half of the specified minimum masonry strength of 1500 psi.

<u>Discussion of NRC Concerns</u>

During our meetings with the NRC staff several concerns were raised regarding the evaluation of these walls. These concerns are addressed as follows:

A. <u>Special Inspection - SRP/UBC</u>

Justification for the use of allowable stresses corresponding to special inspection is presented in Attachment B.

B. <u>Masonry Strengths in Tension, Shear and Bond</u>

The flexural tensile strength of the masonry units at PVNGS is not of concern since the tensile strength is not depended upon for carrying any loads. Likewise, the shear strength is not of concern since the calculated shear stresses are lower than the unfactored allowable values. In the case of the bond, the resistance is provided by the grout surrounding the reinforcement. Therefore, the significant parameters affecting the bond strength are the grout strength, rebar size, and available lap length, which are all known with a high degree of confidence level.

ACI 318-63 Code specifies allowable bond stresses for reinforced concrete structures. These are applicable to masonry walls since the rebar strength is mostly developed by bearing of rebar deformations against the concrete or grout. The tensile stresses

developed in the concrete/grout surrounding the rebar must be taken into account. A review of the minimum concrete covers for the ACI 318-63 Code substantiates that the available cover for the PVNGS masonry walls (a combination of grout and masonry unit) is adequate to develop the calculated bond strength.

It should be noted that the ACI 318-71 and 318-77 bond provisions are expressed in terms of development length rather than the allowable stresses. However, the bases for the development lengths are similar to those of ACI 318-63 allowable bond stresses.

C. UBC Provisions For Staggered Splices

It is recognized that the UBC Code contains provisions for staggered splice in the section for concrete and not in the section for masonry. However, the fact that the code is silent about this subject in the section on masonry does not imply that it is not permitted. The reinforced masonry industry code, ACI 531-79 does permit staggered splices up to 8 inches.

Results of the test performed by the National Concrete Masonry Association (NCMA) form the basis for the code provision for staggered splices. The test results and an evaluation indicating its applicability to the PVNGS design are included in Attachment D.

D. Strength of Lap Splices

The strength of the lap splices can be limited by the splitting strength of the concrete. Since the as-built average rebar cover in these walls is much greater than the minimum concrete covers established in the concrete code it may be concluded that the as-built splitting strength and, thus, the bond strength is at least as much as the values given in the concrete codes.

It is recognized that the pullout tests are not considered to be as effective as the beam tests in establishing the bond strength. There does not appear to be any beam test data available for the masonry bond strength. For this reason the pullout tests performed by the NCMA were quoted as a reference for establishing the adequacy of staggered splices. These tests are still considered to effectively demonstrate the adequacy of staggered splices. With regard to the bond strength, the results obtained in the NCMA tests are compatible with the tests which had formed the basis of the concrete code provisions.

Thus, it is concluded that the existing staggered splices in the PVNGS masonry walls are acceptable since the calculated bond stresses are within the code allowables.

E. Reduced Length of Lapped Splices

The minimum lap splice length in the PVNGS masonry walls, as determined by the rebar detectors, is 16 inches for the first lift and 24 inches for the subsequent lifts. In this case the minimum lap length requirements of the code, for full rebar strength development, are not met. The concrete codes permit reduced lap lengths provided the bond stresses are within allowable limits. As shown in Attachment C, the allowable bond stress determined from the ACI 318-63 Code has been modified to account for the splicing of all rebars at the same elevation. Use of this allowable stress to check the adequacy of the available lap length is within the current standard industry practice.

F. Other Concerns Regarding the Lap Splices

Providing splicing of all reinforcement at the same elevation is a standard industry practice. The codes recognize this fact and require an increase in the splice lengths to account for this condition. Accordingly, the allowable bond stresses shown on Attachment C have been modified by the factor 0.75.

It is also recognized that the lap splice is not sufficient to develop 125% of the yield strength to ensure a ductile failure mode. However, these walls are designed to remain within rebar elastic limits under the design loads. As shown in Attachment C, the maximum calculated rebar stress is in the order of one-half the yield point. Although it is desirable to provide ductility in the design of these structures it is neither required nor essential to do so since the provision to remain under elastic limits during an SSE inherently assures a high degree of confidence in the design.

An adequate assessment of safety margins requires consideration of variation in grout strength, variation in rebar location, lack of ductile failure, and impact of cyclic loading. In this evaluation the average grout strength was utilized, consistent with the conventional design. Variation in rebar location was considered through the use of the NCR reporting for rebar locations. As discussed above, lack of ductility will not adversely affect performance under the design loads. So far as the cyclic loads are concerned, the effects of repeated stresses are considered to be neglibible at these stress levels.

Conclusions

Based on the extensive evaluation of the Palo Verde masonry walls, it is concluded that the walls are adequately constructed and will perform their intended function following a seismic event.

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JUSTIFICATION OF ASSUMPTIONS IN MASONRY WALL EVALUATION

Assumption Item	Justification
Use of full allowable stresses	All significant structural attributes are based on as-built data (rebar size and location) or code specified strength (masonry). ACI 531 Code Committee Chairman concurs with the approach. See Attachment B for Comparison of Required and Available Documentation for Masonry Walls.
Staggered splices are adequate to transfer the tensile forces	ACI 531 code permits staggering up to 8 inches. ACI 531 Code Committee Chairman concurs with the validity of the assumption. Test data are available on staggered splices (see Attachment D). Staggered bars being in adjacent cells is permissible since masonry unit grout interface strength is not of concern due to mechanical locking provided by tapered masonry units. Test data show that failure is either by fracture of rebar or by pull-out, depending on the embedment length, thus indicating no adverse effect due to staggering of the spliced bars. (See Attachment D.)
Less than code specified lap length is acceptable	Lap splice requirements on various codes are based on average bond stress concept. Therefore, it is permissible to determine the actual bond stress, based on available lap length and rebar force to be developed, and compare it with allowable values. ACI 531 Code Committee Chairman concurs with the approach. Available test data show that most of the rebar force is transferred to the surrounding grout over the first 12 inches of embedment. (See Attachment D.)

COMPARISON OF REQUIRED AND AVAILABLE DOCUMENTATION FOR MASONRY WALLS

Introduction

Appendix A to SRP Section 3.8.4 "Interim Criteria for Safety-Related Masonry Wall Evaluation" provides minimum design considerations and criteria for the review of safety-related masonry walls. In addition, the SRP states that "Use of allowable stresses corresponding to special inspection category shall be substantiated by demonstration of compliance with the inspection requirements of the NRC criteria."

Design of the masonry walls, performed prior to issuance of the SRP, used allowable stresses corresponding to special inspection. Details of the specification requirement which permit the use of these allowable stresses are shown on pages 3 and 4 of this attachment.

This attachment demonstrates that available documentation coupled with the requirements contained in Uniform Building Code - 1979 provide adequate assurance that the walls were constructed in accordance with the provisions of the code and meet the intent of the code for special inspection requirements.

Uniform Building Code Requirements

Section 2413(d) 8 of UBC-1979 states "Special inspection during grouting shall be provided in accordance with Section 306; however, the work shall not qualify for the stresses entitled "Special Inspection" in Table No. 24-H unless fully inspected." This is a logical result of the Special Inspection provisions stated in Section 306. It merely states that implementing only one part of the special inspection requirements is not adequate for taking credit for full inspection; the other parts, namely inspection of masonry and mortar, and placement of the rebar must also be implemented.

From UBC-1979, Special Inspection requirements, Section 306(a)6 structural masonry requirement for special inspection are as follows:

1. Sampling and placing of all masonry units.
2. Placement of reinforcement.
3. Inspection of grout spaces.

From UBC Section 306(b) the Special Inspector shall:

1. Observe the work to be certain it conforms to the design drawings and specifications.

2. Furnish inspection reports to the engineer and bring all discrepancies to the immediate attention of the contractor for correction, then if uncorrected, to the proper design authority.

3. Submit a final signed report that the work was, to the best of his knowledge in conformance with approved plans and specifications and applicable workmanship of the code.

The value of f'_m shall be assumed to be 1500 psi when prism test are not made, as is the case with PVNGS, as defined in UBC Section 2404(c) 1 and 3, "Determination of Masonry Design Strength."

Discussion and Conclusions

There is sufficient evidence available to support proper sampling and placing of all masonry units. In addition the masonry units comply with Type N1.

Post construction inspection shows proper rebar size and spacing. Staggered splices were observed at some locations and some deviations of rebar location in the thickness direction were noted. Accordingly, these deviations have been accounted for in the design calculations (see Attachment C). In the spirit of the UBC, although after the fact, these deviations are taken in the context of being reported to the design authority. This meets the intent of the UBC since not corrected by the Contractor (Section 306b).

Adequate post construction inspection exists to demonstrate proper workmanship of filling grout spaces. As confirmatory evidence more than 3000 small diameter holes (one penetration for every 8 square feet of wall) have been drilled in the walls to provide for attachments. Only one observation of lack of grout in the masonry unit being drilled was noted (repaired by NCR).

Based on this extensive evidence it can be concluded that the use of special inspection allowables is valid and the as-built condition of the walls are sound and are constructed in accordance with the code. Those deviations noted during the post construction inspection have been factored into the calculation by the design authority in accordance with the code.

COMPARISON OF REQUIRED AND AVAILABLE DOCUMENTATION FOR MASONRY WALLS

Specification Requirement	Evidence of Compliance	Conclusions
A. Subcontract submittals.		
1. Samples of each type of masonry unit.	Telecon by M. Swift to B. Currlin dated 3-30-79.	Submittal required for color only. No structural applications.
2. Mortar and grout mix designs with certified test results.	Grout mix design submitted via memo dated 12-21-78.	Grout test data is available. Average strength is 2300 psi.
	Mortar mix design submitted via letter dated 4-13-78.	Mortar conforms to type S. Strength range is 2200-2600 psi.
3. Certification verifying masonry units conform to specification.	Letters dated 12-21-78, 4-13-78, 4-19-78 and 1-29-86.	Masonry units conform to type N1. Minimum strength is 2000 psi.
4. Certificate from testing laboratory verifying mortar and grout used complied with mix designs.	Bechtel surveillance testing performed on grout.	Average grout strength is 2300 psi. Mortar design strength is 2200 to 2600 psi.

COMPARISON OF REQUIRED AND AVAILABLE DOCUMENTATION FOR MASONRY WALLS
(Continued)

B. Inspection Documentation.		
1. Foundation surface preparation.	Subcontractor certificate of compliance included as part of billing invoice.	Shear is small. Therefore foundation surface preparation is not a significant structural parameter.
2. Masonry units installed comply with specification.	Subcontractor certificate of compliance included as part of billing invoice.	Masonry units comply with type N1.
3. Reinforcement installed in accordance with specification.	Subcontractor certificate of compliance included as part of billing invoice.	Post construction inspection shows proper rebar size and spacing. Staggered splices were observed at some locations. Deviations of the rebars from design location, in the thickness direction, is accounted for in calculations, based on NCR. Less than code required lap lengths are accounted for in the calculations.
4. Mortar and grout prepared and installed as specified.	Subcontractor certificate of compliance included as part of billing invoice.	Additional assurance of grout filling (i.e., no voids) provided by inspections of as-built wall and drilling of approximately 3000 holes in the completed walls. Visual observation shows proper mortar placement (uniform thickness; 5/8 inch or less). Visual inspection and scratch test indicate adequate mortar strength.

ANALYSIS RESULTS

The tables in this attachment provide the calculated stresses in the walls, under the SSE and OBE conditions. The calculated flexural stresses are compared with the allowable values based on the ACI 531-79 Code, as modified by the SRP 3.8.4. The calculated bond stresses are compared with the allowable values based on the ACI 318-63, since the SRP 3.8.4 does not provide a stress increase factor for bond. Review of these results indicate the following:

I. SSE Condition

A. Walls at Elevation 100'

All the masonry, reinforcement and bond stresses are within their allowable values. The maximum flexural compression fibre stress in the masonry exceeds the reduced allowable value by 5%. This is considered acceptable since the significant structural attributes of the wall are known, fulfilling the purpose of special inspection.

B. Walls at Elevation 74'

Similar to walls at elevation 100', all the masonry, reinforcement and bond stresses are within their allowable values. The maximum flexural compression fibre stress in the masonry exceeds the reduced allowable value by 16%. This is considered acceptable since the significant structural attributes of the wall are known, fulfilling the purpose of special inspection.

With regard to bond stresses, the calculated values are less than those allowed by the ACI 318-63 Code. In fact, the allowable value is a minimum of 30% greater than the maximum calculated bond stress. This margin is considered acceptable under the SSE loads.

II. OBE Condition

A. Walls at Elevation 100'

At elevation 100', all the calculated stresses are less than the full allowable values. The calculated maximum flexural compression stresses in the masonry exceed the reduced allowable by 5%. This is considered acceptable as noted for the SSE case.

B. Walls at Elevation 74'

The bond stresses are within the values allowed by the ACI 318-63 Code for working stress design. The rebar stress exceeds the allowable stress by approximately 8%, but is well below the minimum yield stress. The masonry stress exceeds the reduced and the full allowable values. However, it is only 53% of the ultimate strength of the masonry and therefore will remain well within the elastic range.

The out-of-plane wall deflection for the OBE is small while vertical deflections are negligible. Calculated differential displacements of small magnitudes are acceptable with no adverse impact on the conduits, instrumentation tubing, or small electrical panels, and their supports.

Therefore, even though the masonry stress for elevation 74' exceeds the allowable value, the wall is structurally capable of resisting the loads from the OBE and the light Category I attachments will perform all their intended functions.

III. Conservatisms in the Analyses

The results of the analysis are conservative for the following reasons:

1. The stresses were conservatively obtained from a response spectra analysis using a smoothed instructure response spectra as input. A coupled time history analysis with the masonry walls included in the building model and using the free-field time history as input would produce lower stresses.

2. The wall at elevation 74' was analyzed with a hinge-hinge boundary condition. In fact, there is some fixity at the base of the wall which would increase its frequency, lower the acceleration, and would lower the stresses.

3. The analysis used the Code specified values for E and I whereas EI dynamic could be used for the frequency calculation. Since EI dynamic is significantly higher, the use of the more realistic value for EI would result in a higher frequency of the wall and a corresponding lower acceleration and calculated stress.

4. The minimum material strengths were used in the analysis whereas the actual material strengths would be expected to be higher. Again, this would produce a higher frequency for the wall and a corresponding lower acceleration and calculated stress, and a higher allowable stress value.

5. The allowable bond stress from the ACI 318-63 Code is a lower bound value based on testing and already includes a capacity reduction factor.

The combination of these conservatisms are expected to provide 30 percent additional margin to those shown on the tables on pages 3-6.

MASONRY AND REINFORCEMENT STRESSES (psi) (1)

SSE = 0.20g

ELEV.	STRESS (2)	CALCULATED STRESS	ALLOWABLE STRESS (ACI-531-79) FULL	REDUCED	STRENGTH YIELD/ULTIMATE	COMMENTS
74'-0" (3)	f_m (max)	970	1,250	833	f'm = 1500	f'_m = 1500 is the code value. The minimum expected f'_m is 1800 psi. (5)
	f_s (max)	31,700	48,000	48,000	Fy = 60,000 Fu = 90,000	
100'-0" (4)	f_m (max)	880	1,250	833	f'm = 1500	f'_m = 1500 is the code value. The minimum expected f'_m is 1800 psi. (5)
	f_s (max)	32,700	48,000	48,000	Fy = 60,000 Fu = 90,000	

NOTES:
(1) EI is based on ACI 531-79 and ACI 318-83.
(2) f_m = Compressive fibre stress in masonry; f_s = tensile stress in reinforcement.
(3) Calculated stresses are based on average 'd' distance as determined from the NCR.
(4) Rebar location at the base of the wall at EL. 100'-0" is defined by the dowel locations shown on "Q" Drawing 13-C-ZJS-172. Because "Q" Dowel placement is inspected, the bars at the base are in the "as-designed" location. Therefore, stress calculation is based on as-designed condition.
(5) Based on ACI Journal, June 1979, Title No. 76-32, page 707.

(3) [illegible]

[illegible]

BOND STRESSES (psi)(1)
SSE = 0.20g

ELEV.	LOCATION	CALCULATED STRESS (2)(3)(4)	ALLOWABLE STRESS (5)	FACTOR OF SAFETY (6)	LAP LENGTH
74'-0"	First Course Lap Splice EL. 80'-0"	296	387	1.31	16"
	Second Course Lap Splice EL. 84'-8"	250	387	1.55	24"
100'-0"	First Course Lap Splice EL. 106'-0"	100	465	4.65	16"
	Second Course Lap Splice EL. 110'-8"	140	465	3.32	24"

NOTES:
(1) EI is based on ACI 531-79 and ACI 318-83.
(2) Calculated stresses are based on average 'd' distance as determined from the NCR.
(3) Stresses calculated based on f'_m = 1,500 psi.
(4) Allowable bond stress is based on: $9.5\sqrt{f'_c}/D$ per Section 1801(c) of ACI 318-63, a reduction factor of 0.75 per Section 805(b) of ACI 318-63, a capacity reduction factor of 0.85 per Sections 805(b) and 1504(b) of ACI 318-63, and a grout strength of f'_c = 2300 psi.
(5) Factor of safety = allowable/calculated bond stress. This factor of safety is in addition to the factors of safety inherent in the ACI 318-63 code.

MASONRY AND REINFORCEMENT STRESSES (psi)[1]
(OBE = 0.10g)

ELEV.	STRESS[2]	CALCULATED STRESS	ALLOWABLE STRESS (ACI-531-79) FULL	REDUCED	STRENGTH YIELD/ULTIMATE	COMMENTS
74'-0"[3]	f_m (max)	790	500	333	f'm = 1500	f'_m = 1500 is the code value. The minimum expected f'_m is 1800 psi. (5)
	f_s (max)	25,800	24,000	24,000	Fy = 60,000 Fu = 90,000	
100'-0"[4]	f_m (max)	350	500	333	f'm = 1500	f'_m = 1500 is the code value. The minimum expected f'_m is 1800 psi. (5)
	f_s (max)	12,800	24,000	24,000	Fy = 60,000 Fu = 90,000	

NOTES:
(1) EI is based on ACI 531-79 and ACI 318-83.
(2) f_m = Compressive fibre stress in masonry; f_s = tensile stress in reinforcment.
(3) Calculated stresses are based on average 'd' distance as determined from the NCR.
(4) Rebar location at the base of the wall at EL. 100'-0" is defined by the dowel locations shown on "Q" Drawing 13-C-ZJS-172. Because "Q" Dowel placement is inspected, the bars at the base are in the "as-designed" location. Therefore, stress calculation is based on as-designed condition.
(5) Based on ACI Journal, June 1979, Title No. 76-32, page 707.

BOND STRESSES (psi)[1]
OBE - 0.10g

ELEV.	LOCATION	CALCULATED STRESS (2)(3)(4)	ALLOWABLE STRESS (5)	FACTOR OF SAFETY (6)	LAP LENGTH
74'-0"	First Course Lap Splice EL. 80'-0"	224	230	1.03	16"
	Second Course Lap Splice EL. 84'-8"	201	230	1.14	24"
100'-0"	First Course Lap Splice EL. 106'-0"	30	276	9.20	16"
	Second Course Lap Splice EL. 110'-8"	50	276	5.52	24"

NOTES:
(1) EI is based on ACI 531-79 and ACI 318-83.
(2) Calculated stresses are based on average 'd' distance as determined from the NCR.
(3) Stresses calculated based on $f'_m = 1{,}500$ psi.
(4) Allowable bond stress is based on: $4.8\sqrt{f'_c}/D$ per Section 1301(c) of ACI 318-63, a reduction factor of 0.75 per Section 805(b) of ACI 318-63, and a grout strength of $f'_c = 2300$ psi.
(5) Factor of safety = allowable/calculated bond stress. This factor of safety is in addition to the factors of safety inherent in the ACI 318-63 code.

APPLICABILITY OF TEST DATA TO PVNGS DESIGN

Reference: Schoenfeld, G.D., "Effect of Bar Size and Embedment Depth on 'Pull-out' Resistance of Deformed Concrete Reinforcing Bars," Addendum to a report submitted to Department of Housing and Urban Development, Circa 1980, by the National Concrete Masonry Association.

A. Bond Strength: The bond strength for a No. 6 rebar was developed in specimen H6T-24(20) which failed in a pull-out mode. The bond strength was:

$$f_b = 42{,}080/(24)(2.356) = 744 \text{ psi}$$

Correcting for grout strength gives

$$fb = 744 \times \sqrt{\frac{2300}{4270}} = 546 \text{ psi}$$

where 2300 psi is the grout strength for PVNGS and 4270 is the grout strength used in the test program.

Conclusions: Based on the test results, the average expected bond strength in the PVNGS masonry walls is in the order of 550 psi. (Note: A capacity reduction factor must be applied to this value to account for testing variability).

B. Staggered Splices: Reference test data shows that no premature failure occurred due to spliced rebars being in the adjacent cells. The failure mode was either rebar fracture or pull-out, depending on the embedment length. The pull-out mode was preceded by the tension splitting of the masonry units. All these results are typical of reinforced concrete and indicates that staggered splices in adjacent cells have no adverse effects.

It is noted that the confinement provided by the rod-and-plate assembly in the referenced tests will be provided by the in-place continuity of the actual walls.

Conclusion: The existing staggered splices in PVNGS walls are acceptable.

C. Distribution of Bond Stress: Reference test data indicates that a great majority of the rebar load is transferred to the adjacent group during the first 12 inches of embedment, when the stresses are below yield level. In the PVNGS design, the maximum rebar stress was determined to be 32,700 psi or about 14,400 pound load on a No. 6 rebar. Review of the test data for PVNGS design conditions show the following:

Specimen	Embedment Length	Load lbs.	Max. Rebar Stress, psi	Distance to Gage, Inch	Rebar Stress at Gage Location, psi	Load Transferred to Concrete Within the Distance to Gage %
H6T-48 (12-24)	48	14000	31800	12	2960	91%
H6T-24 (20)	24	14000	31800	20	270	99%
H6T-48 (24-36)	48	14000	31800	24	530	98%

The above table shows that, for a No. 6 rebar, 98% of the rebar force is transferred to the surrounding grout in less than 20 inches of embedment length, at the maximum stress levels expected at PVNGS.

It should also be noted that, in test H6T-24 (20), with 24 inch embedment, a bar stress of 95,600 psi was developed before failure by pull-out occurred.

Conclusion: Since the minimum splice length at PVNGS is 16 inches at rebar force level of about 11,200 pounds and 24 inches at force level of 14,000 pounds, adequate embedment exists to transfer these forces to the surrounding grout.