ENCLOSURE 2

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November 22, 1982

Director, Office of Nuclear Reactor Regulation
Attention: D. M. Crutchfield, Chief
Operating Reactors Branch No. 5
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D.C. 20555

Gentlemen:

Subject: Docket No. 50-206
Masonry Wall Test Program
SEP Topic III-6
Seismic Design Considerations
San Onofre Nuclear Generating Station
Unit 1

By letter dated July 19, 1982 we submitted to the NRC staff a description of our proposed masonry wall testing program. In a letter dated September 29, 1982, the NRC staff indicated that our proposed test program is acceptable "subject to the satisfactory resolution of the staff's concerns as delineated in the enclosure." Provided as Enclosure 1 to this letter is our response to each of the staff's concerns as identified in the NRC's September 29 letter.

Provided as Enclosure 2 to this letter is a schedule of the activities associated with the test program. As identified on this schedule we are currently proceeding with testing of the test apparatus. The Panel Type 1 walls have been constructed, however, the actual testing of these walls is currently on hold. As you can imagine proceeding with this test program will involve a considerable expenditure of resources. As such, it is our desire to get the unqualified acceptance by the NRC staff of our test program prior to proceeding with the actual testing. Therefore, it is our desire to meet with the NRC staff as soon as possible to resolve any remaining staff concerns. To avoid a significant delay in this program it is requested that you give this matter your prompt attention such that all issues can be resolved by about December 10, 1982.

If you have any questions on the enclosed information or if you wish to discuss this matter further, please let us know as soon as possible.

Very truly yours,

K P Baskin

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Enclosures

ENCLOSURE 1

RESPONSE TO NRC STAFF CONCERNS ON MASONRY WALL TEST PROGRAM

NOVEMBER 1982

SAN ONOFRE NUCLEAR GENERATING STATION

UNIT 1

QUESTION NO. 1

A discussion or assessment of the impact of not inputting vertical motion together with the horizontal one is needed to justify its omission in the test procedure. Also, expand to include the omission of the other horizontal motion (in-plane) in the test, in view of the fact that these components of earthquake motion should be included in the analysis.

RESPONSE

In the analyses of the masonry walls a very conservative method of evaluating the impact of the vertical component of motion was utilized. At each time step when deflections were greater than the wall thickness the overturning moments due to vertical seismic load and gravity were compared to the horizontal inertia restoring moments. For a limited number of time steps the inertial restoring forces were less than the overturning forces. However, these periods were of very short duration (maximum .045 seconds) and it is concluded that they do not affect the wall stability. For a more extensive description, refer to References 1 and 2. Therefore, as long as the actual deflections as determined from the tests are of the same order or less than those from the analyses, the vertical motion can be ignored.

The in-plane horizontal component of motion is not pertinent for the Turbine Building walls since the connection details at the top of these walls will prevent them from being subjected to this component of load. In addition, there is no vertical load on these walls other than their own weight. The Reactor Auxiliary Building walls would be subjected to low in-plane shear loads. This level of load has little or no effect on the out-of-plane response of the walls, and therefore, the consideration of this component is not necessary for this test sequence. The Fuel Storage Building walls would be subjected to both in-plane and out-of-plane loads during a DBE and the combined loads were analyzed in accordance with the criteria established in Reference 1. These analyses included two horizontal and one vertical component of motion as reported in Reference 3. Therefore, given this information, the inclusion of the horizontal (in-plane) and vertical motions would not affect the stated test objective. In addition, the physical limitations of the available test facilities do not permit the inclusion of two components of motion simultaneously. The only feasible method of testing walls of similar size to those in the buildings of interest at San Onofre is by the test method proposed.

References

1. Computech Engineering Services, Inc., "San Onofre Nuclear Generating Station Unit 1, Seismic Evaluation of Reinforced Concrete Masonry Walls, Volume 1: Criteria," forwarded by letter from K. P. Baskin to D. M. Crutchfield dated January 15, 1982.

2. Computech Engineering Services, Inc., "San Onofre Nuclear Generating Station Unit 1, Seismic Evaluation of Reinforced Concrete Masonry Walls, Volume 3: Masonry Wall Evaluation," forwarded by letter from K. P. Baskin to D. M. Crutchfield dated January 11, 1982.

3. Computech Engineering Services, Inc., "San Onofre Nuclear Generating Station Unit 1, Seismic Evaluation of Reinforced Concrete Masonry Walls, Volume 5: Fuel Storage Building Soil Backfill Condition Evaluation," forwarded by letter from K. P. Baskin to D. M. Crutchfield dated September 30, 1982.

QUESTION NO. 2

Discuss the basis of your selection of time history duration and demonstrate its adequacy considering the fact that the duration of motion may have important bearings on non-linear analysis results.

RESPONSE

The duration of the test input motions has been established with consideration of the nonlinear response of the walls. All the time histories that will be used in the testing of the wall panels are based on real recorded earthquakes. The duration of motion that will be used in the test program is 30 seconds for all the test walls.

The factors that influenced the selection of the history duration were the following:

1. The design strong motion duration which is appropriate for the San Onofre site was documented in Attachment 6 to SCE's letter to the NRC dated August 9, 1982 regarding free field ground motion. Based on the data included in that report, it is concluded that a mean design duration of about ten seconds is considered appropriate since this duration will be used with ground motion parameters which are at least 84th percentile instrumental values. Furthermore, it is concluded that the use of a twenty second duration would exceed the average maximum value of strong motion duration obtained from all sources in the literature and is very conservative. The duration of motion that will be used in the test program is thirty seconds for all the test walls. This duration is obviously conservative when compared to the values in the literature as summarized in Attachment 6 to SCE's August 9th letter.

2. Results from the nonlinear analysis of the masonry walls indicate that the maximum responses of key parameters, such as deflections and rebar strains, occur within the first 20 seconds of the response. The time histories used in these analyses had a duration

of 20 to 30 seconds. As stated above, these time histories were based on real recorded earthquake motions in which the strong motion part was contained within the first 20 seconds.

3. During the past 10 years of shake table testing at the Earthquake Engineering Research Center (EERC) of the University of California, Berkeley, several earthquake motions have been developed and are kept on file. Each record has the essential spectral characteristics of the recorded motion, but is modified with a high pass filter to ensure that the physical capabilities of the actuators used to drive the shake table are not exceeded. These records include El Centro, Taft, Olympia, and Pacoima dam events. These earthquake motions all have durations of 30 seconds. The EERC experience in using these records with all kinds of structures has been that they are of sufficient duration for the nonlinear response.

QUESTION NO. 3

Discuss the reason for not being able to input two distinct motions at the two actuator levels as was implied in the proposal. If indeed, the input motions can't be distinct, assess its impact on the test findings and test conclusions. Appropriate compensatory measures, if any planned for adoption in interpreting the data, should also be discussed.

RESPONSE

The test proposal states that "it is anticipated that the two actuators will be controlled by the same signal" and that "a study is underway to determine the feasibility of controlling the actuators with separate signals."

It is now clear that the two actuators can be controlled by one basic signal which may then be scaled differently by the controls to each actuator. This implies that for the Fuel Storage and Turbine Buildings the analysis motion at the bottom of the wall could be used as input to the bottom actuator whereas a scaled version of that same motion could be used for the top actuator. A typical scale factor for the Fuel Storage Building would be 1.45 whereas the scale factor for the Turbine Building still remains to be determined. The Reactor Auxiliary Building does not have any instructure amplification and the scale factor between top and bottom actuators would therefore be equal to 1.00.

Using the same displacement controlled signal with a different scale factor for the two actuators is realistic for the Fuel Storage Building. The instructure response spectra at the top and bottom of the walls are, in all practicality, identical over the frequency range of interest, with the exception of a scale factor. Therefore no compensatory measures for interpreting the data will be necessary for these walls.

If two different signals can not be used for the Turbine Building walls, the impact of using scaled signals at the top and bottom of the walls will not be significant relative to the objectives of the test program. This is true since the objective of the program is to validate the analysis methodology, and the same input used in the test program will be used in the correlation analysis.

The Reactor Auxiliary Building does not produce any amplified instructure motion and thus the same ground type motion will be used as input for both top and bottom actuators. Therefore no compensatory measures for interpreting the data will be necessary for these walls.

Although the method of using scale factors for the top actuator input signal is easy to implement, there do exist some physical limitations. For example, if too high a scale factor is used, the top actuator may be required to perform a task exceeding its capabilities (e.g. extension beyond the maximum 6-inch displacement). However, it is not expected that such conditions will occur. Refer to Question No. 6 with respect to limitations of the test equipment.

The possibility of using two different input motions is still being investigated, and the prospects are good for finding an acceptable solution. If such a solution is found, it is planned to input two different motions to the two actuators for the Fuel Storage and Turbine Building walls. Refer to Question No. 6 for the explanation on how this task would be accomplished. If not, the above method of using different scale factors will be utilized for both the Fuel Storage and Turbine Buildings.

QUESTION NO. 4

Describe how the closeness/similarity between the test walls and the insitu walls are to be assured with respect to the construction material and boundary conditions.

RESPONSE

While the objective of the program does not necessitate close similarity between the test walls and the insitu walls, steps are being taken to ensure that this similarity is as close as is reasonably possible. In order to achieve similarity of construction materials the original specification used in the construction of the insitu walls is used in the construction of the test panels. This specification (BSO-112, January 11, 1965) includes information on all the construction materials used in the walls. The implementation of the specification by the constructors will be monitored by engineers and samples of all construction materials will be tested as further verification of the quality and similarity of the construction materials.

All of the walls considered applicable for the nonlinear analysis are dowelled into concrete members (footings, slabs, etc.) at their base. The base detail for each of the test panels will also be dowelled into a concrete base, utilizing the same area of dowelled steel and the same embedment percentage as in the insitu walls. This assures the same boundary conditions at the base of the test panels as are present in the insitu walls. All of the walls considered applicable for the nonlinear analysis have a pin connection at their tops. This connection is typically a ledger angle or in the case of the Turbine Building (Panel Type 3) a plate. The test panels will all have connections at the top of the wall similar to that shown in Figures 4.1 through 4.3. The top actuator will be attached to the steel member at the top of the panel via a pin connection. Therefore, the top connection of the test panels will be a pin, just as in the in situ walls.

QUESTION NO. 5

Give detailed discussion about and justification for major piping/equipment attachment simulation in terms of weights, eccentricities and locations of such attachments. Also discuss the methods in attaching these piping/equipment.

RESPONSE

The items attached to the masonry walls considered applicable to the nonlinear analyis consist mostly of electrical conduit and minor items such as grounding cable, telephones, small diameter copper tubing and other similar minor items. Due to their light weights, all of the above items other than the electrical conduit are not considered relevant to the test program. In addition, piping is not considered relevant to the program since none of the masonry walls subject to the nonlinear analysis have significant piping attached to them.

The masonry walls in the superstructure of the Fuel Storage Building (represented by Panel Type 1) have very little electrical conduit attached to them. The small amount of electrical conduit attached to these walls is all surface mounted within one to two inches of the wall surface. The maximum weight of conduit tributary to any of these supports is approximately two hundred pounds. Of the approximately half dozen supports with conduit weights near or above one hundred pounds, all but a few occur within the top or bottom third of the wall. Therefore, due to considerations of weight, eccentricity, location and the low number of supports, attached electrical conduit is not considered to be relevant to the Fuel Storage Building walls. This being the case, Panel Type 1 will not have any attached equipment except those instruments necessary for testing.

The seventeen foot high walls of the Reactor Auxiliary Building (represented by Panel Type 2) have even fewer attached items than the Fuel Storage Building superstructure walls discussed above. Those few items which are attached to these walls are surface mounted with the heaviest tributary weight to any support being approximately fifty pounds. Therefore, Panel Type 2 will also have no equipment attached except those instruments necessary for testing.

The twenty-one foot high walls of the Turbine Building (represented by Panel Type 3) do have a significant amount of attached electrical conduit in many areas. Figure 5.1 shows a particular support which occurs on a Turbine Building wall. This support is typical of those supports with the greatest weight and highest eccentricity for these walls. The weight carried by this support is estimated to be 250 pounds and the eccentricity is ten inches. The support is located at the extreme top of the wall. The great majority of the electrical conduit supports on these walls support horizontal conduit runs and occur within the top or bottom third of the wall. For testing purposes it is intended to represent four or five of these supports or similar supports within the top and/or bottom third of the wall. The supports and weights associated with them will be configured and placed in such a manner as not to interfere with the testing instrumentation on the test panel. It should be noted that for simplicity, during the analysis it was assumed that all conduits were filled to capacity. This is not the case in reality and the information concerning weights contained in this response is based on information gathered during the current raceway support seismic reevaluation program.

QUESTION NO. 6

How are the in-structure amplified design spectra to be used in defining actuator's input motion determined? Procedures for verification of similarity of input actuator motions to those of in-structure design spectra should be provided. What are the criteria for judging acceptability of the similarity of the motions?

RESPONSE

The actuator input motion for the three wall types representing the Fuel Storage Building, the Reactor Auxiliary Building and the Turbine Building will be determined as follows:

a. The spectra for the Fuel Storage Building test walls (Panel Type 1) will be obtained from the three-dimensional nonlinear building analysis.

b. The Reactor Auxiliary Building was determined to respond in the rigid range and thus the spectra for these test walls (Panel Type 2) will be the unamplified ground motion at both top and bottom.

c. The spectra for the Turbine Building test walls (Panel Type 3) will be the ground spectra at the bottom of the walls and the spectra obtained from the linear analysis of the Turbine Building at the top of the walls if different input motions can be used as discussed in the response to Question 3.

Verification of similarity of input actuator motions to those of instructure design spectra will be done through spectral comparison. The criteria for acceptability of the similarity of the motions is that the actuator motion response spectra be as close as possible within the physical limitations of the actuators and their control system to the instructure design spectra over the frequency range of interest (i.e. the frequency range from the uncracked to the fully yielded masonry walls).

It will be necessary, because of the physical limitations of the test equipment, to filter out some of the low frequency components of the input motions. Inclusion of these low frequency components would require actuator displacements and velocities that exceed the capacity of $\pm$ 6 inches displacement and

26 in/sec velocity. This will require filtering out most frequencies lower than approximately 0.3 Hz. The impact of this will not be significant since this low frequency band (0-0.3 Hz) is outside of the frequency range of interest of the walls.

QUESTION NO. 7

How representative are the 3 types of panels selected compared to the range of walls considered applicable for the non-linear analysis computer code? A discussion to demonstrate that reasonable representativeness has been achieved should be provided.

RESPONSE

The three samples chosen for the test program represent those typical wall sections whose response would be the largest under the motion associated with the DBE. The rationale for the selection of the panels follows:

1. PANEL TYPE 1: This panel type is to be approximately 24 feet high with #7 reinforcing bars spaced at 32 inches for vertical reinforcement and #5 bars at 48 inches as horizontal reinforcing. All the walls in the Fuel Storage Building which are relevant to the test program have this same configuration. These Fuel Storage Building walls are all located in the second story of the building (twenty to thirty feet above grade) and will thus have amplified input motions. The Ventilation Equipment Building walls have the same reinforcing pattern, however, these walls are four feet shorter and are founded on grade. While Panel Type 1 is meant to represent the Fuel Storage Building walls, its response would also envelop that of the Ventilation Equipment Building walls. Therefore, Panel Type 1 represents the relevant Fuel Storage Building walls, and, in addition, it is a configuration which is considerably more conservative than that of all the Ventilation Equipment Building walls.

2. PANEL TYPE 2: This panel type is to be approximately 17 feet high with #4 reinforcing bars spaced at 32 inches as vertical reinforcement and #5 bars at 48 inches as horizontal reinforcing. This wall represents the tallest walls in the Reactor Auxiliary Building which have the same horizontal reinforcement and as a minimum #5 bars spaced at 48 inches as vertical reinforcing. The test panel

therefore provides equivalent reinforcing steel percentages to the actual walls. The remaining walls in the Reactor Auxiliary Building are approximately six feet shorter and are predicted to experience very little inelastic response due to their relatively short span. The minimum reinforcing for these walls is the same as for the 17 foot walls (worst case #5 @ 48 horizontal and vertical). Therefore, Panel Type 2 is representative of the worst case walls, considering span and reinforcing, in the Reactor Auxiliary Building.

3. PANEL TYPE 3: This panel type is to be approximately twenty feet high with #5 reinforcing bars spaced at 32 inches as vertical reinforcement and #5 bars spaced at 48 inches as horizontal reinforcement. This panel type represents the tallest walls in the Turbine Building. Since all the walls in this building considered to be applicable to the test program have the same reinforcing and are of the same height or shorter, Panel Type 3 is representative of the worst case walls in the Turbine Building.

Table 7.1 gives a summary of the above presented information. The six typical wall types applicable to the test program are all shown in this table.

TABLE 7.1

BUILDING	REINFORCING	SPAN	REMARKS
Fuel Storage (2nd Story)	#7 @ 32 V #5 @ 48 H	~24'	Panel Type 1 Configuration
Ventilation	#7 @ 32 V #5 @ 48 H	~20'	
Reactor Aux.	#5 @ 48 V #5 @ 48 H	~17'	Panel Type 2 Configuration
	#5 @ 48 V #5 @ 48 H	~11'	
Turbine	#5 @ 32 V #5 @ 48 H	~20'	Panel Type 3 Configuration
	#5 @ 32 V #5 @ 48 H	~16'	

QUESTION NO. 8

Provide a discussion of how pre-test prediction analyses by DRAIN-2D code will be done and what particular results will be submitted to NRC staff prior to the test?

RESPONSE

The pre-test prediction analyses will involve removing all known sources of conservatism from the previous analyses. These include:

1. Adjusting the extent of plastic hinging of the rebar by modeling every joint in each wall.
2. Including the tensile strength of masonry joints prior to the formation of cracking.
3. Adjusting material properties to reflect material test results for each panel type rather than using the specified minimum values.

The best estimate of the time histories that will be used in each test will be used in the pre-test analytical prediction. Reporting of the results will be presented in the same format as previously submitted to the NRC (Reference 1).

Reference

1. Computech Engineering Services, Inc., "San Onofre Nuclear Generating Station Unit 1, Seismic Evaluation of Reinforced Concrete Masonry Walls, Volume 3: Masonry Wall Evaluation," forwarded by letter from K. P. Baskin to D. M. Crutchfield dated January 11, 1982.

QUESTION NO. 9

Key rebar strains should be measured directly by strain gages so that the strain time history is known during testing.

RESPONSE

Since the submittal to the NRC staff of the written test program, a study of recent experiments involving strain gauges on embedded reinforcing bars has been performed. This study revealed that marking the bar and then attempting to measure strains after its removal from the wall is not practical. This study also revealed that a type of sealed strain gauge does exist and can be used in this test program. Therefore, in lieu of marking the bar and removing it after testing, these new sealed strain gauges will be placed on the bar over the central length of bar and near the base. These sealed gauges have a design limit of 2% strain. Due to this fact, some of the gauges may exceed their design limit during testing. However, curvature measurements will be available from which the strains can be deduced. The strain-curvature relationship can be developed up to the 2% limit of the gauges and extrapolated if necessary. The locations on the wall of some of the instrumentation may be modified to best accomodate this procedure.

QUESTION NO. 10

Direct measurement of longitudinal strains in the face shell (both on tension and compression side) should be made. If such a measurement is difficult to implement, then discuss what are the difficulties involved.

RESPONSE

Measurement of the longitudinal strain in the face shell of a masonry unit in a dynamic test of this nature is extremely difficult. A strain gage of sufficient length (4-6 inches) is required to measure the compression or tension strain over a reasonable length of the unit. It is very difficult to ensure that an adequate bond exists over the full length of the strain gage when bonding to a surface with the roughness of a masonry unit. If bond is lost along any part of the length of the strain gage due to local microcracking or other causes the strain gages tend to buckle and thus record tension rather than compression measurements. These problems have been observed in many previous test programs (e.g. References 1 and 2) and as a result strain gage measurements have not been utilized for masonry units in any recent masonry research programs.

Other parameters will be measured to deduce the strain in the masonry face shells. These are the rebar strains, wall displacements and wall curvature.

References

1. Becica, I. J., "Behavior of Hollow Core Masonry Prisms Under Axial and Bending Loads and its Implications on Ultimate Strength Design of Masonry Structures", Structural Models Laboratory, Report M-80-1, Department of Civil Engineering, Drexel University, Philidelphia, PA, June 1980.

2. Becica, I. J., and Harris, H. G., "Evaluation of Techniques in the Direct Modelling of Concrete Masonry Structures", Structural Models Laboratory, Report M-77-1, Department of Civil Engineering, Drexel University, Philidelphia, PA, June 1980.

QUESTION NO. 11

Describe the means of precracking the panels and at the same time preventing the yielding of the rebar. How is the panel to be loaded? Also, testing of pre-cracked panels should proceed only after an acceptable mapping of the cracks is taken.

RESPONSE

There are several means by which the panels may be precracked without yielding the rebar. In the test program two alternatives are proposed. The second alternative will only be used if there are unforeseen difficulties with the first method. It should be noted that when a wall is precracked, an attempt will be made to have the cracks form at the same location as they did in the full intensity test of the wall tested without precracking. This would correspond to the worst location with respect to the analysis.

The two alternatives are:

1. Displacement controlled point load applied at the appropriate height of the wall until cracking occurs.
2. A low amplitude sinusoidal displacement time history will be applied by top and bottom actuators. The excitation displacement will have a frequency equal to the fundamental frequency of the wall that is being precracked, thereby creating resonant response of the wall until it cracks.

In the case of the point load a strong I-beam will be placed against the wall and a hydraulic jack located between the beam and a reaction frame. This system will apply a displacement controlled force normal to the face of the wall. During this load application the actuators at the top and bottom of the wall will be locked in place. Because of the nature of a displacement controlled force, it can only be maintained at a constant level for a certain displacement as long as the stiffness of the structure (in this case the wall) remains unchanged. Therefore, as soon as the wall cracks, the load will drop to a much lower level, thereby avoiding yielding of the rebar.

In the case of a sinusoidally applied displacement time history the same principle applies. As soon as the wall cracks, its frequency drops and excitation of the wall at its resonant frequency stops. This significantly decreases the force to which the wall is subjected, thereby preventing yielding of the rebar. To obtain the cracking load, the amplitude of the sinusoidal input will be slowly increased until the wall cracks. The disadvantage with this method is that the height at which cracking occurs cannot be controlled. Cracking will occur at the weakest joint in the region of the highest applied moment.

Marking the cracks may be difficult because although the cracks open up slightly in the deformed shape, they close completely when the wall is in the undeformed position. This makes the cracks almost, if not completely, invisible. However, every attempt will be made to locate the cracks and mark them.

QUESTION NO. 12

How will the test results be used to interpret the analysis results? The criteria to judge the acceptability of the test results as a means of verifying the analysis methodology and their bases should be provided.

RESPONSE

The main objective of the test program is to demonstrate the overall conservatism of the analysis methodology and not to proof test the walls. In order to achieve this objective, several parmeters of importance from the results, both test and analytical, will be compared. These include maximum deflections, rebar strains, masonry strains and the extent of plastic hinging. If the test results yield values lower than or similar to those of the analysis for these key parameters, then verification of the analysis methodology will be complete. If the test results yield higher or non-conservative values for the key parameters, then the significance and sensitivity of the parameter(s) on the analysis results will be reassessed. The most important comparisons between the analysis and test results will be the maximum deflection of the wall, the amount of yielding of the rebar and the ability of the wall to withstand the compressive strains induced by the maximum deflection. If the compressive masonry strains are excessive, face shell spalling along the entire length of a bed joint will be visually apparent from the tests.

QUESTION NO. 13

The means and criteria to dispose significant differences between the test results and the computer analysis results and basis thereof should be provided.

RESPONSE

The means and criteria to dispose of significant differences between test results and computer analysis results will depend upon what, if any, differences occur between the key parameters of the analytical model and those obtained from the test results. Three scenarios are possible:

1. The key parameters from the test results are more conservative than those used in the analytical model. In this case the analytical results will be conservative and therefore acceptable.
2. The key parameters from the test results are less conservative than those used in the analytical model. In this case the significance and sensitivity of these less conservative results on the response of the walls will have to be assessed. This may require revision of the model and reanalysis of some or all of the walls.
3. Some of the key parameters from the test results are less conservative and some are more conservative than those used in the analytical model. In this case some of the walls may need to be reanalyzed to determine the overall impact of the changes in the key parameters.

As noted in Question No. 12, the major criteria in evaluating the impact of a change in one or more of the key parameters will be:

a. Deflection predicted by the analytical model;
b. The ability of the wall to resist the compressive strains induced in the face shell of the masonry units;
c. The amount of yielding in the rebar.

A specific quantitative disposition criteria is not practical because of the relationship between the parameters affecting response.

QUESTION NO. 14

On page A-38 of Section 5.1.2, Appendix A of your Response to NRC Review of Methodology, you referred to ACI-318 regarding a method of computing the effective moment of inertia. This method was used in the deflection calculations of Turbine Bldg. Group I walls. Also, on page 36 of Volume 4 "Fuel Storage Building," Fuel Storage Building wall stiffness was based on 1.5 times the cracked moment of inertia. The equations (9-7), (9-8) and (9-9) of ACI-318 code provide expressions for Ie, Mcr and fr respectively, where Ie is the effective moment of inertia for computation of deflection, Mcr is the cracking moment and fr is the modlus of rupture of concrete.

In view of the above, provide a discussion as to whether the pertinent "fr" values for masonry wall have been used in your analysis calculations and whether a number of flexural tensile strength prism tests are necessary in order to validate the "fr" values used in the analysis. If such tests are considered unnecessary, provide the basis thereof.

RESPONSE

The basic model used in the analysis of masonry walls assumed that the walls were precracked at critical locations. This was done to ensure that conservative analytical results were obtained and that the worst condition of the wall was accounted for. Therefore the modulus of rupture was not used in the analytical model.

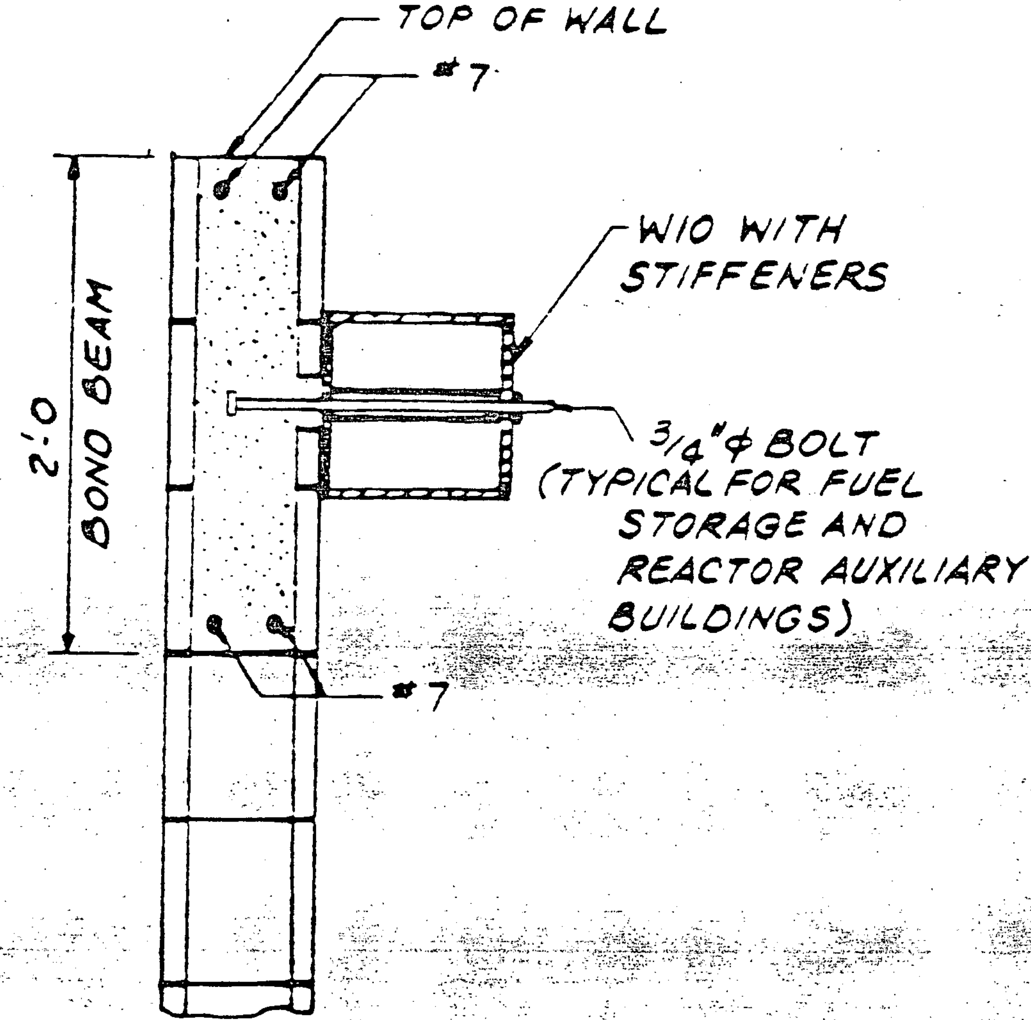
The value of 1.5 times the cracked moment of inertia was both developed and verified by matching the load-deflection curves taken from available test data on masonry walls. The determination of this value was not based on equations (9-7), (9-8) and (9-9) of the ACI-318 code. The details of the development and verification of this value may be found in Reference 1. Therefore, the deflections reported in Reference 2 were not based on the ACI-318 equations. The ACI-318 equations were used only in Appendix A of our Response to NRC Review of Methodology to show the correlation between the load-deflection curves resulting from (1) the model used in the non-linear analysis and (2) those curves resulting from the ACI-318 equations.

The only reason to obtain fr from flexural tensile tests would be to use this value in a best fit model of the wall. It must be noted that inclusion of this parameter in the analytical model will decrease the response of the wall since it will delay yielding of the rebar. The flexural tests are therefore not considered necessary.

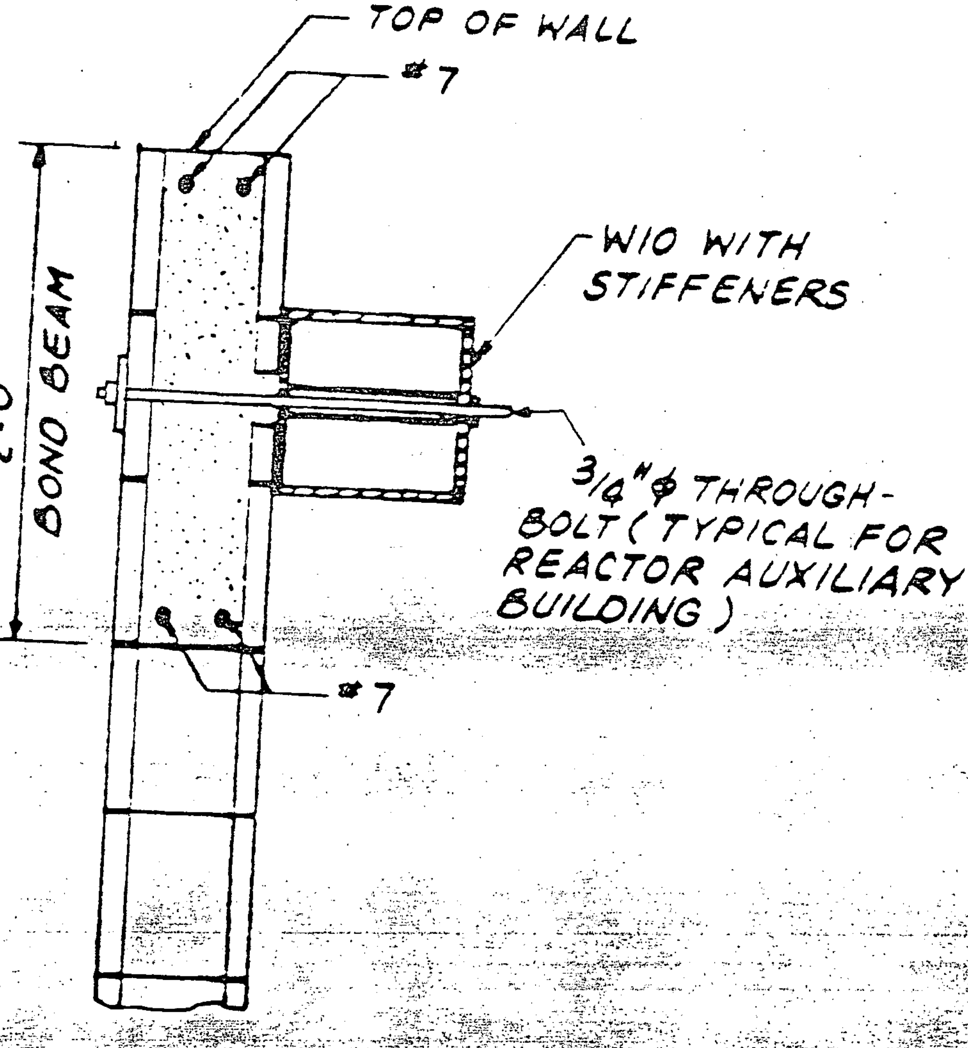
References

1. Computech Engineering Services, Inc., "San Onofre Nuclear Generating Station Unit 1, Seismic Evaluation of Reinforced Concrete Masonry Walls, Volume 2: Analysis Methodology," forwarded by letter from K. P. Baskin to D. M. Crutchfield dated January 11, 1982

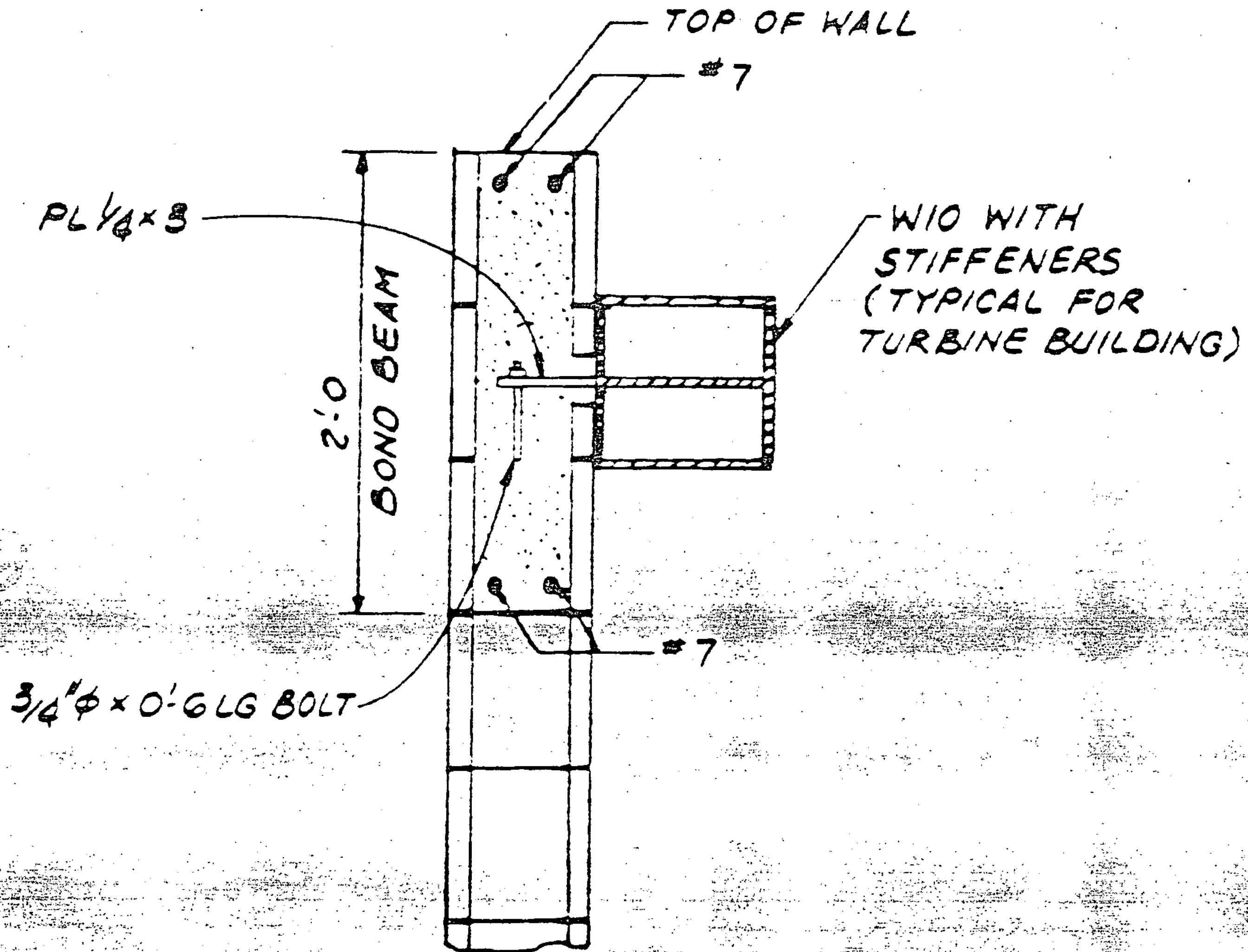
2. Computech Engineering Services, Inc., "San Onofre Nuclear Generating Station Unit 1, Seismic Evaluation of Reinforced Concrete Masonry Walls, Volume 3: Masonry Wall Evaluation," forwarded by letter from K. P. Baskin to D. M. Crutchfield dated January 11, 1982.



TYPICAL CONNECTION
AT TOP OF TEST PANEL
FIGURE 4.1



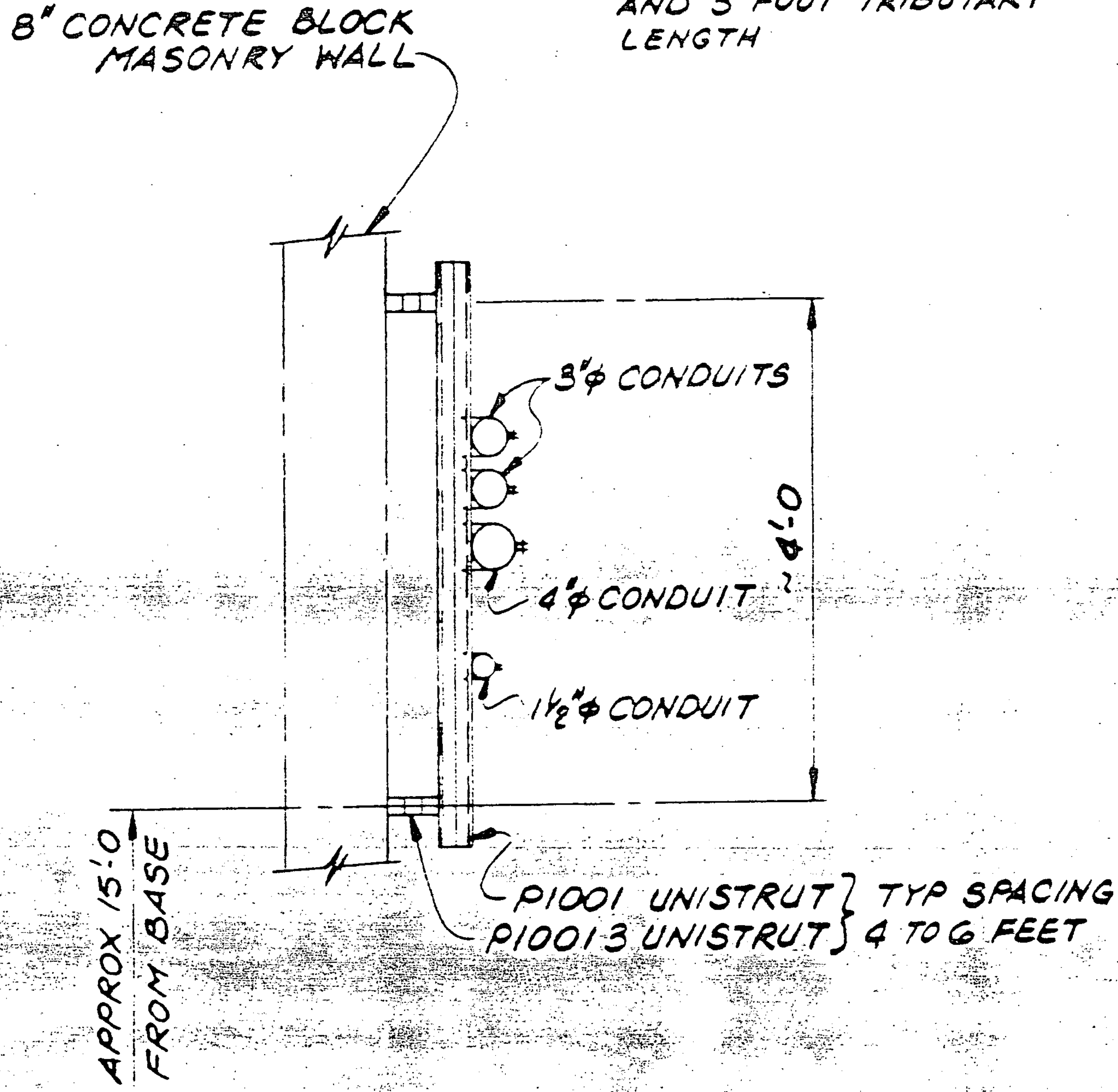
TYPICAL CONNECTION
AT TOP OF TEST PANEL
FIGURE 4.2



TYPICAL CONNECTION AT TOP OF TEST PANEL

FIGURE 4.3

TOTAL WEIGHT TO SUPPORT 250 LBS BASED ON 70% FILL AND 5 FOOT TRIBUTARY LENGTH



CONTROL CONDUIT SUPPORT
FOR PANEL TYPE 3
ELEVATION VIEW
FIGURE 5.1

M[illegible]NRY WALL TEST SCHEDU[illegible]