SAN ONOFRE NUCLEAR GENERATING STATION
UNIT 1

SEISMIC EVALUATION OF REINFORCED CONCRETE MASONRY WALLS

MASONRY WALL TEST PROGRAM

PRETEST ANALYSIS - WALL PANEL TYPE 1

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1 INTRODUCTION

A test program is being conducted as part of the masonry wall evaluation for the San Onofre Nuclear Generating Station, Unit 1 (SONGS-1). The aim of this test program is to demonstrate the conservatism of the analysis methods used for the wall evaluation. In conjunction with the aim of the test program a commitment was made to the NRC staff to perform an analysis of one test specimen to predict a "best-estimate" response prior to the test being performed.

This report presents the results of a "best-estimate" analysis for Wall Panel 1. In the model used for the response computed herein an attempt was made to accurately represent all aspects of the test specimen and thus compute a response as close as possible to the actual measured response. The detail in the model was such as to enable the test responses to be predicted with a high level of confidence. To allow for the inherent variability in a number of parameters upper and lower bound response quantities with a much lower probability of occurrence have also been included. The actual tests will be the basis for the validation of the conservatism in the analytical methods, not the results of this analysis.

2 WALL TEST SPECIMEN

The test specimen for Test 1 is as shown in Figure 2.1. The specimen is an 8'-0" wide section of the walls typical of the Fuel Storage Building at SONGS-1. It is constructed of 36 courses of hollow concrete masonry blocks, giving a total height of 24'-0". The section is vertically reinforced with 3-#7 rebars, and is grouted in the reinforced cells only except for the lower three courses and the bond beam at the top support which are fully grouted. Horizontal reinforcing consists of #5 bars at 4'-0" centers.

The specimen is constructed on a reinforced concrete footing and is connected to this footing through #5 dowels at 16" centers which are lapped with the vertical reinforcing.

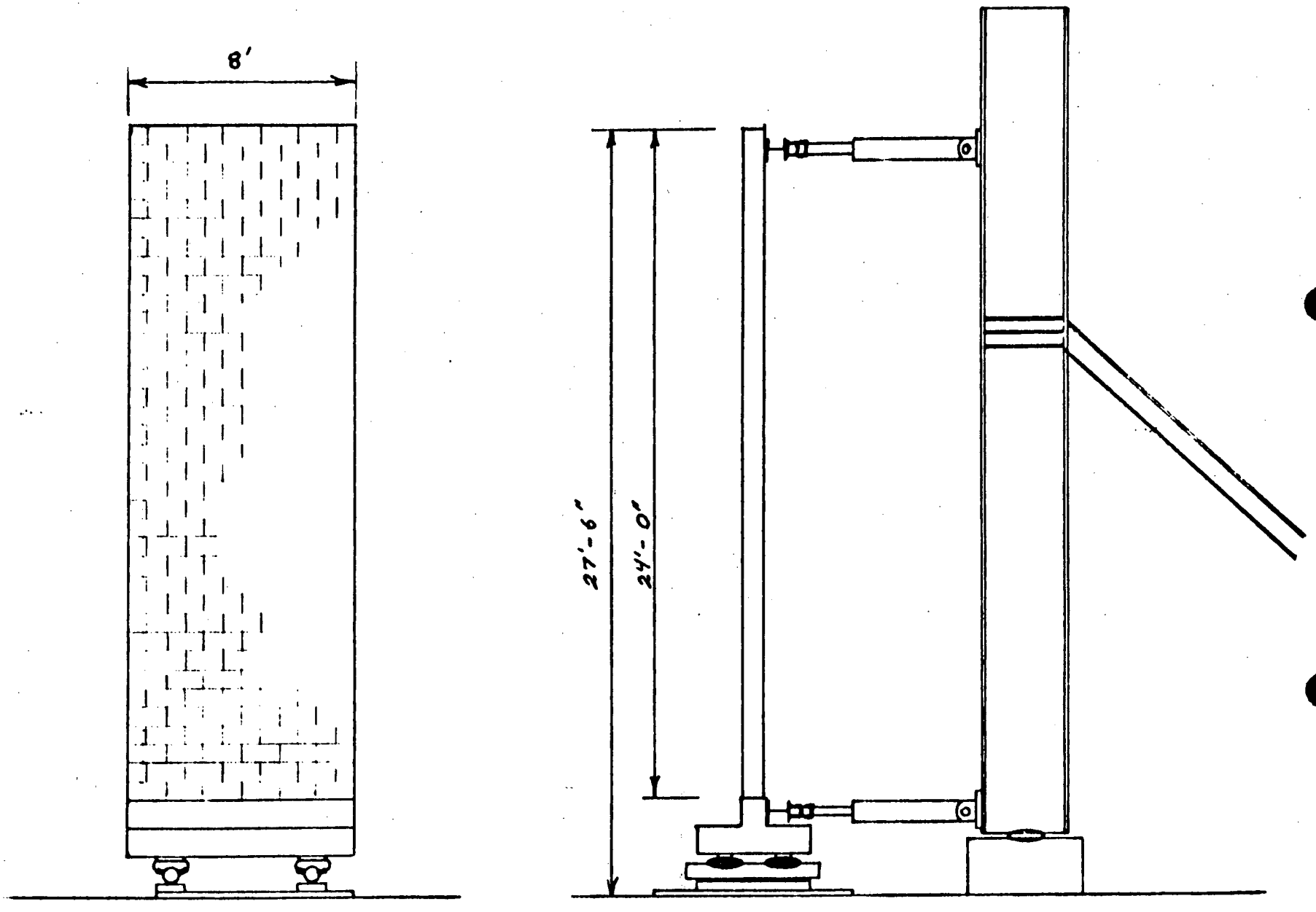

FIGURE 2.1 : PROTOTYPE WALL SPECIMEN

3 WALL MODEL

The pretest analyses were performed using the DRAIN-2D2 computer program, a general purpose program for the dynamic non-linear analyses of plane structures. This program is the same as that used for the evaluation of the SONGS-1 walls but a number of modifications were made to the models to remove conservatism and obtain the "best-estimate" response. These alterations are discussed in detail in the following sub-sections.

3.1 Global Model

The overall model was assembled as a sequence of "block" and "joint" substructures. The block substructures comprised four 4-node plane stress finite elements. These elements remained linear throughout the analysis and represent the masonry blocks between joints. Each joint substructure consisted of two face shell elements plus a central rebar element. The face shell elements remained elastic in compression but had a specified tension limit. Once this limit was reached the tensile strength and stiffness reduced to zero. The central rebar element had a bilinear stress strain curve with a yield level corresponding to the yield stress of mild steel.

Figure 3.1 shows the general form of the global model. In Figure 3.2 a more detailed representation of the block and joint substructures is shown.

The main difference between this overall model and that used in the wall evaluation is that this model explicitly includes every joint whereas the evaluation model included only the central 5 joints explicitly and accounted for the effects of more widespread cracking by an effective moment of inertia over the remainder of the model. This was recognized at the time to be an approximation, although a conservative one, and so the "best-estimate" model has removed this approximation. Other differences between this model and the evaluation model include the effects of vertical gravity loads and the inclusion of the initial gross stiffness of the masonry wall prior to joint cracking.

3.2 Model Variables

The general form and overall dimensions of the global model explicitly represent the geometry of the test specimen. Other properties used as the

final parameters of this model are discussed in the following sub-sections.

3.2.1 Joint Width

The joint is the weakest section of a masonry wall and non-linear effects due to mortar cracking and the steel rebar yielding are concentrated at these joints. The properties of these joints vary depending on the three possible conditions of each joint:

a. Joint face shells uncracked. The stiffness of the joint is based on the gross moment of inertia of the section. This is provided by the stiffness of the elastic slope of the face shell elements.

b. Joint face shells cracked – steel elastic. The stiffness is based on the transformed moment of inertia, provided by the compressive stiffness of one face shell plus the elastic rebar element.

c. Joint face shells cracked – rebar yielded. The stiffness is approximately 5% of the transformed stiffness, formed by the strain hardening stiffness of the rebar element plus the compressive face shell.

Extensive parameter studies and the examination of available literature performed during the wall evaluation [References 1 and 2] had led to the conclusion that approximately 18 inches total of rebar over the yielding zone contributed to the stiffness for b. and c. above. The evaluation model provided this length in a single element. As all joints were modelled for the "best-estimate" model this length was distributed over the number of joints expected to comprise the yielded zone, 12 joints. Based on the total length of the rebar element expected to yield and the number of joints in the yield zone the joint width was assigned a length of 18/12 = 1.5 inches.

It is recognized that the effective length of rebar as represented by the joint width is not constant but varies in a non-linear way both with the position of the joint and the intensity of loading. The variation is caused by the progressive loss of bond stress between the rebar and the grout core. As the stress in the rebar increases bond is lost and the length

over which yielding occurs increases. However pending the results of this test series sufficient information is not available to warrant a detailed representation of this non-linear variation. Therefore the 1.5" value is viewed as a representative mean value. The more highly strained joints will have larger effective lengths and this has been recognized in the conversion of output bar extensions to equivalent steel strain ratios, where a value of 2" has been used in the upper hinge and 3" at the lower hinge.

The length of each course must be accurately modelled in order that the correct displacements due to joint rotation are obtained. In the model no lateral displacements are permitted across the 1.5" joint width but the joint substructures do model the joint rotation. Thus to obtain the correct displacement an 8" length of finite element, the actual course height is required. It is for this reason that the model dimension is 26'-11" between supports compared with the prototype height of 22'-8", the extra length coming from the the 34 1.5" joint substructures.

3.2.2 Damping

Damping constants for the "best-estimate" model were selected to provide 3% of critical damping at the frequency of the wall in its uncracked state increasing to 7% for the cracked and yielded wall. In the evaluation only the cracked and yielded states were modelled and a damping ratio of 7% of critical was used.

3.2.3 Block Properties

The masonry blocks between the joints for the "best-estimate" evaluation will remain elastic and therefore the properties for these elements were selected to provide the gross moment of inertia of the specimen throughout the analysis. The modulus of elasticity was based on 1000 times the expected masonry prism strength of 2025 psi for the test panel. (Note that this value is 1.5 times the specifed minimum strength of 1350 psi). The wall evaluation used a modulus of elasticity of 800 or 1200 times the specified minimum strength of 1350 psi, the actual factor being selected on a wall-by-wall basis to give the maximum response.

3.2.4 Joint Properties

As stated above, the joint substructure is modelled by two face shell elements plus a central yielding rebar element. The properties of the face shell elements and rebar elements were derived from the actual material properties and the required section modulus. A masonry tensile strength of 65 psi was used for the face shell elements prior to cracking. The residual stiffness under reversing loads before the face shell elements again come into compression is theoretically zero but cyclic test results show a finite value. A stiffness of 0.6% of the uncracked stiffness was used for this value. Some variations in these properties could be expected because of the non-homogeneous nature of masonry construction and these are discussed further in Section 4. For the Fuel Storage Building wall evaluation a reversing stiffness of 2.15% was used with zero tensile strength.

3.3 Solution Procedures

The analyses were performed using a constant time-step solution scheme with an event-to-event strategy to reduce the unbalanced loads. Such a solution procedure had been determined to be necessary in the wall evaluation due to the abrupt changes in stiffness in the cracking and yielding wall. This corresponds to the same technique as used for the wall evaluation.

3.4 Time Histories

The test time histories were obtained from analyses of the Fuel Storage Building [Reference 3]. These have been modified to accomodate constraints imposed by the test equipment. Because of scaling performed to ensure that the test spectra enveloped the spectra from the analysis time histories they are generally 10% higher than the Fuel Storage Building time histories in the range of 0.4hz to 50 hz. For the actual test two independent inputs will be used and thus different time histories will be applied at the top and bottom of the wall to reflect the values obtained from the 3-dimensional model of the Fuel Storage Building. The computer program used for the "best-estimate" analyses does not have the capability of using different input time histories at the top and bottom of the wall. To approximate this effect the test time history to be used at the top of the wall was scaled by 0.875 for the analytical model. This scale factor was computed by using a weighted average of the response spectra produced at the top and bottom of the

wall. In general the lower record was 65% to 75% the spectral amplitude of the upper record and so the scale factor used is such as to produce somewhat greater than the mean response at most frequencies. As the two signals are largely in phase due to the dominant rocking mode of the Fuel Storage Building it is not expected that this analysis approximation will have a significant effect.

It should also be noted that the interaction between the test frame and the specimen will modify the input signal to an extent unknown until the actual test. Although this coupling is not expected to be significant it may effect the correlation between the response predicted herein and the actual response. The extent of interaction will be determined from measurments taken on the actuators and on the opposite side of the wall during the tests.

The time history used in this analysis and the 7% damped spectra derived from it are plotted in Figures 3.3 and 3.4.

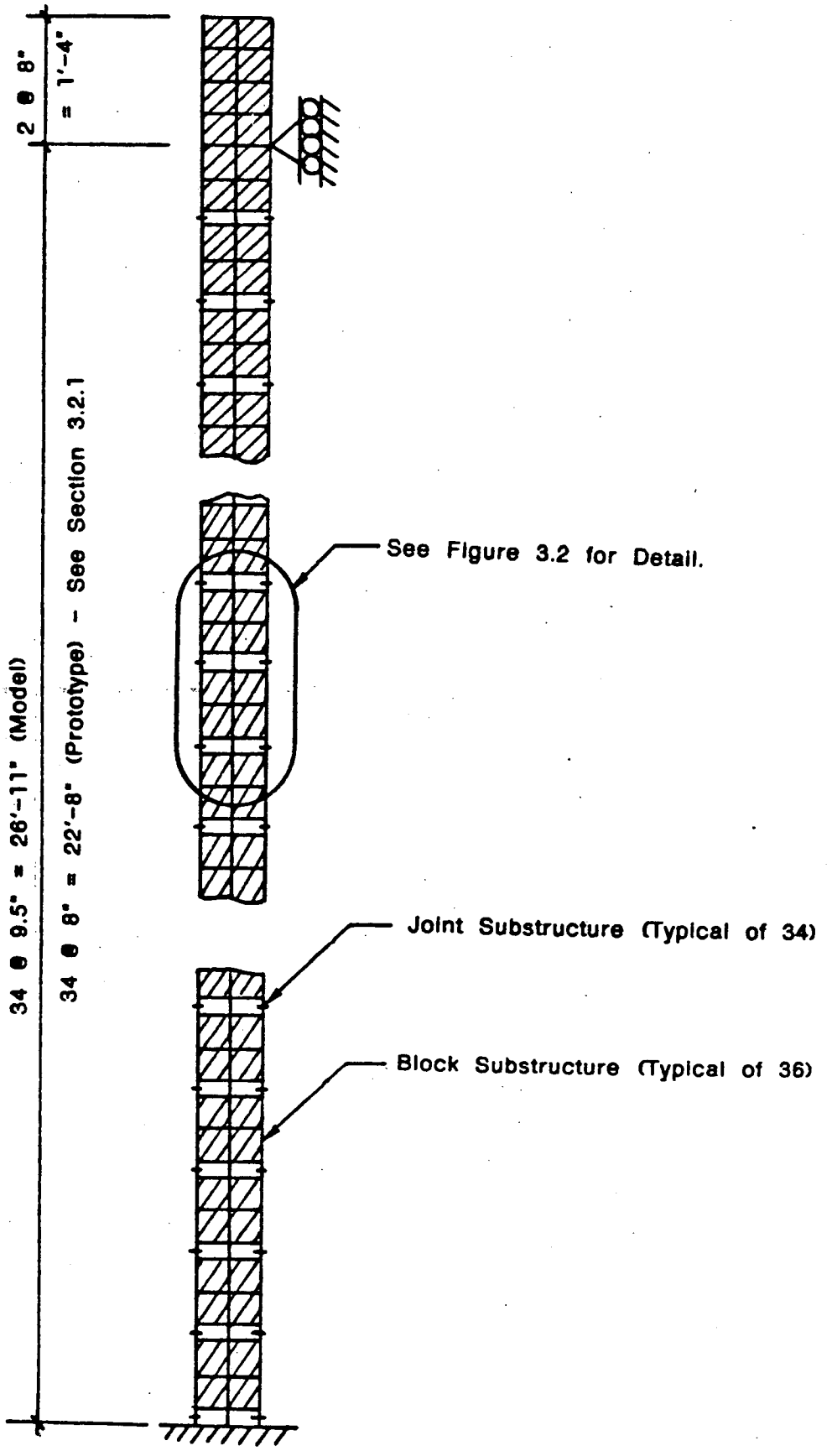

FIGURE 3.1 : GLOBAL MODEL

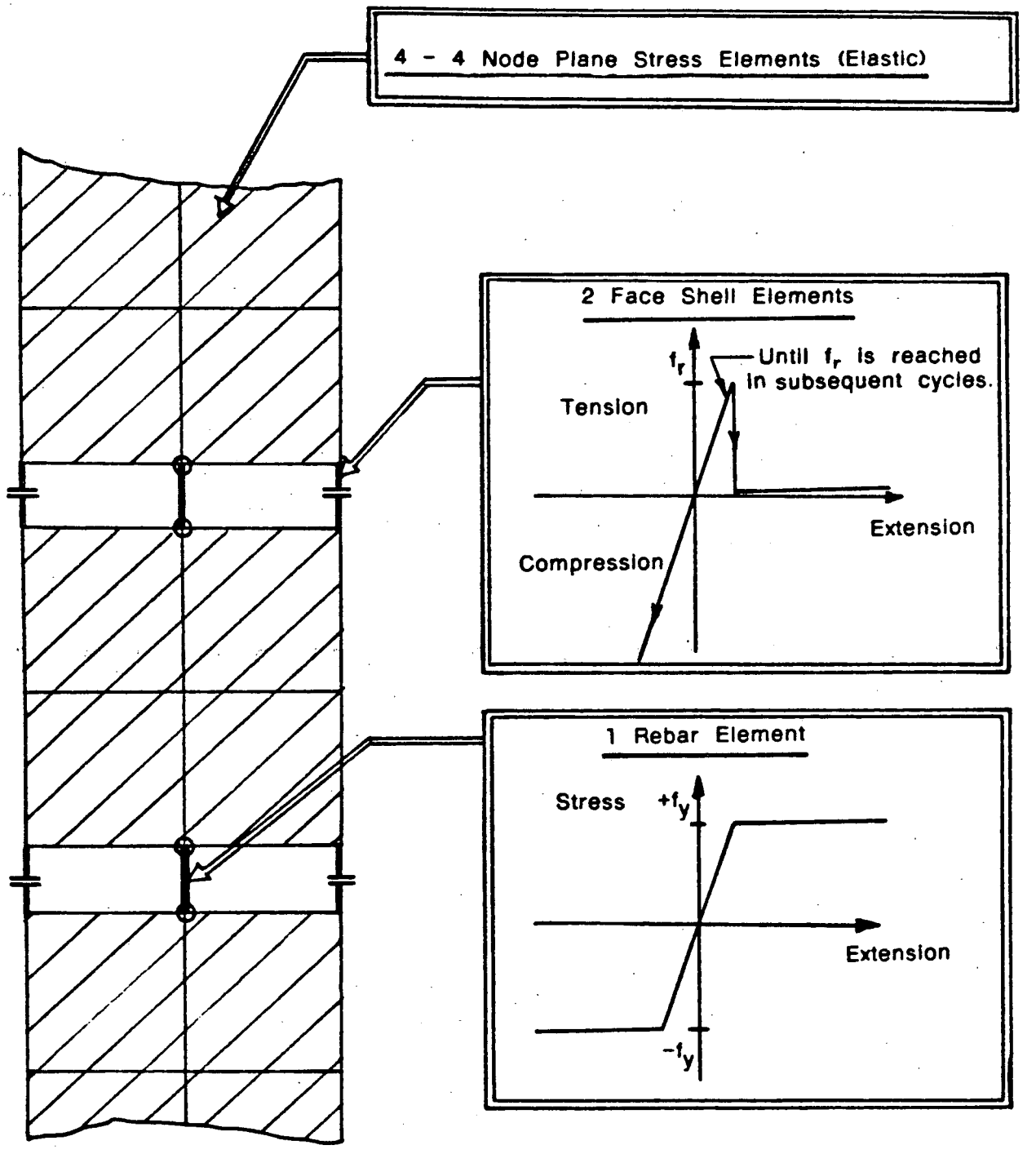

FIGURE 3.2 : DETAIL OF MODEL

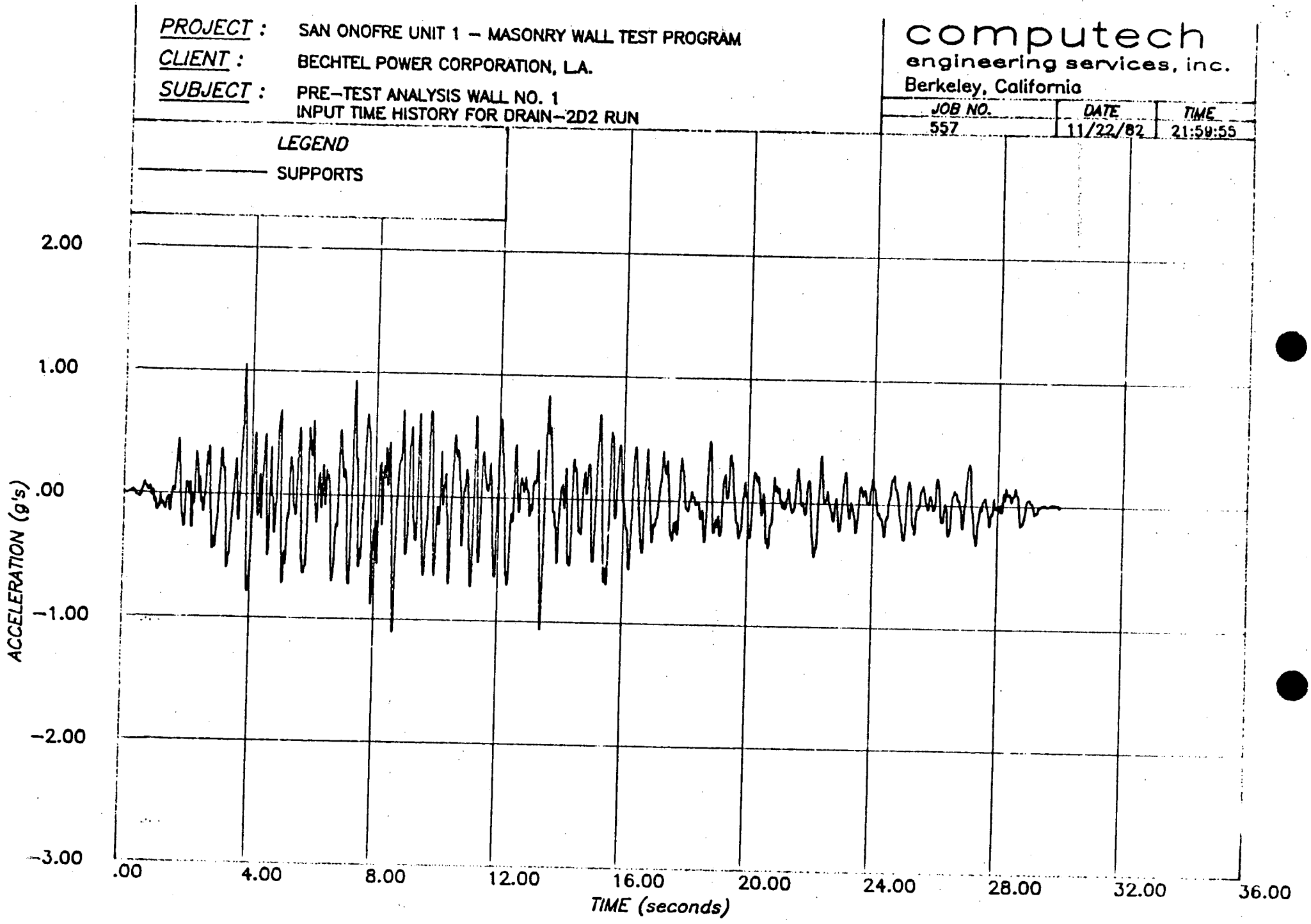

FIGURE 3.3 : INPUT TIME HISTORY USED FOR ANALYSIS

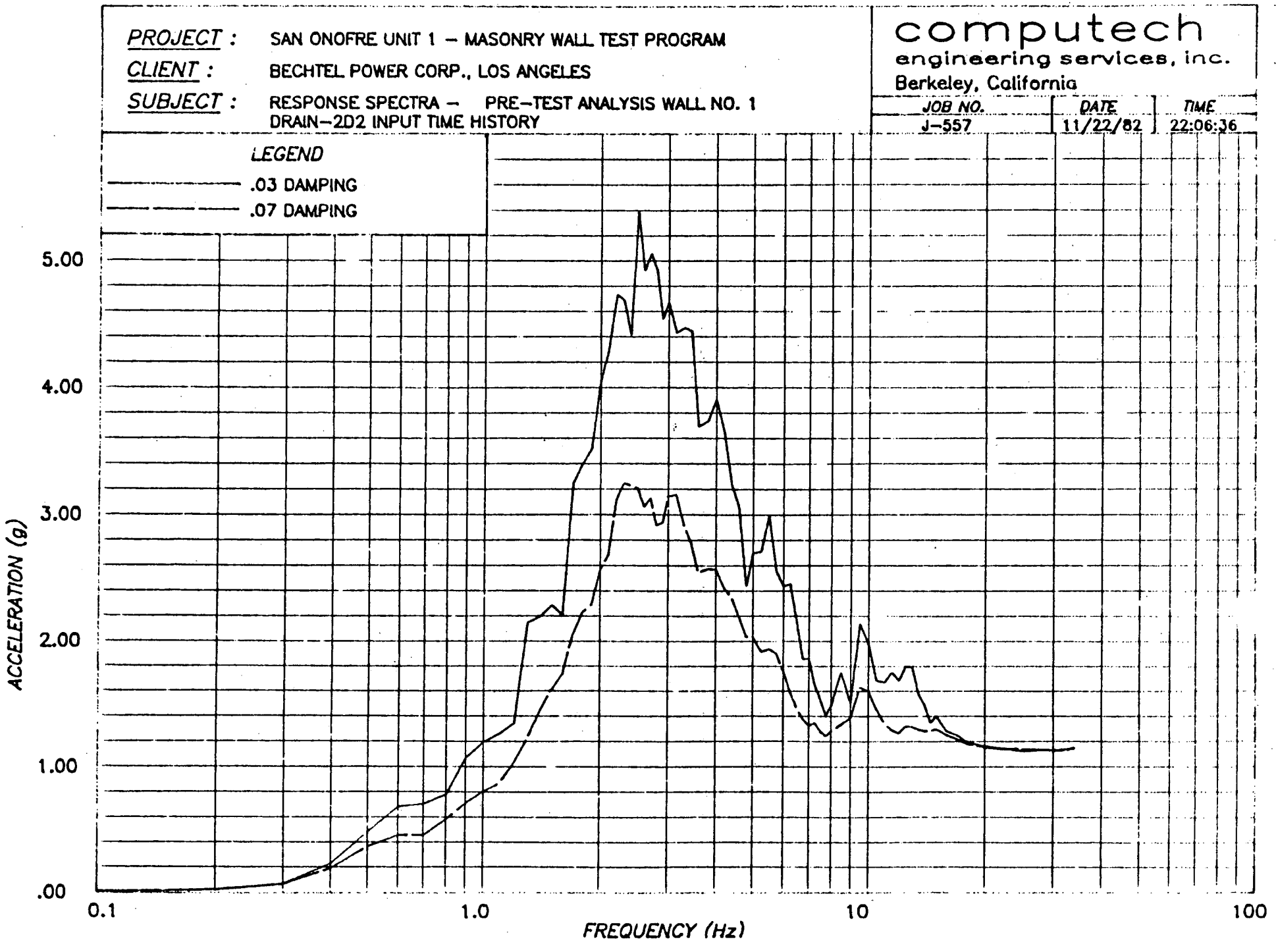

FIGURE 3.4 : RESPONSE SPECTRUM FROM INPUT TIME HISTORY

4 RESULTS OF ANALYSES

Table 4.1 presents the predicted best-estimate maximum displacements and steel strain ratios. The displacement time history of the node with maximum displacements is shown in Figure 4.1 for the complete 30 second record to be used in the test. In Figures 4.2 and 4.3 the first 4.5 seconds and 9.0 seconds respectively are plotted to a larger scale to more clearly illustrate the effect of cracking and yielding of the rebar on the frequency content of the response.

In Table 4.2 critical parameters are identified and best estimates for each of these parameters are listed. These best estimates were used in the pretest analysis reported here. Also listed in Table 4.2 are the lower and upper bound values for each parameter. As stated in the introduction these bounds allow for the inherent variability of reinforced masonry, and are associated with response quantities with a much lower probability of occurrence. A number of preliminary analysis runs of limited duration were performed to study the effect of parameter variations between the lower and upper bound values and the effects on maximum displacements were as given below:

1. A damping value varying from that used in the analysis e.g. 3% to 10% rather than to 7% at low frequencies, plus variations in procedures for accounting for damping in the mathematical model, could influence response by ± 0.5".

2. Modulus of elasticity, Em. A variation of ±20% in this value would effect the displacement by approximately ± 0.5".

3. Masonry tensile strength, fr, varying by ± 50% would influence the maximum displacement by ± 0.5".

4. Residual section stiffness on load reversal, i.e. deviation from a zero stiffness, could also have ± 0.5" effect on displacement.

The overall effect of all these parameters produces a maximum range of uncertainty of +2" and −2": that is, maximum displacements in the range of 5" to 9". The extremes of this range are very unlikely, however, since the 7" value is based on the best estimate for each of the critical parameters and the four parameters would have to simultaneously deviate the maximum amount from the best estimate value.

The effect of the variations in the above parameters on maximum steel strain

ratios would be similar to that for displacements but for these ratios there is an additional uncertainty due to the non-linear nature of the bond and thus effective rebar length. As the stress in the rebar increases bond is lost and the length over which yielding occurs increases. Therefore the steel strain ratio upper and lower bounds have been set further from the mean value than the displacement bounds. The effective rebar length used to calculate the ductility ratios was 3 inches at the base of the wall and 2 inches at the mid-height of the wall.

RESPONSE QUANTITY	BEST ESTIMATE	LOWER BOUND	UPPER BOUND
MAXIMUM DISPLACEMENT (inches)			
POSITIVE	6.8	5"	9"
NEGATIVE	-6.8	-5"	-9"
MAXIMUM STEEL STRAIN RATIO			
BASE OF WALL	12	6	20
UPPER YIELD POINT	7	4	12

TABLE 4.1 : PREDICTED MAXIMUM RESPONSE QUANTITIES

PARAMETER	BEST ESTIMATE	LOWER BOUND	UPPER BOUND
DAMPING			
Uncracked	3%	2%	5%
Cracked	7%	5%	10%
MODULUS OF ELASTICITY , E'm	1000f'm	800f'm	1200f'm
MASONRY STRENGTH			
f'm	2025 psi	1350 psi	2700 psi
ft	65 psi	30 psi	100 psi
REVERSING STIFFNESS	0.6%	0	1.0%

TABLE 4.2 : BEST ESTIMATES OF CRITICAL PARAMETERS

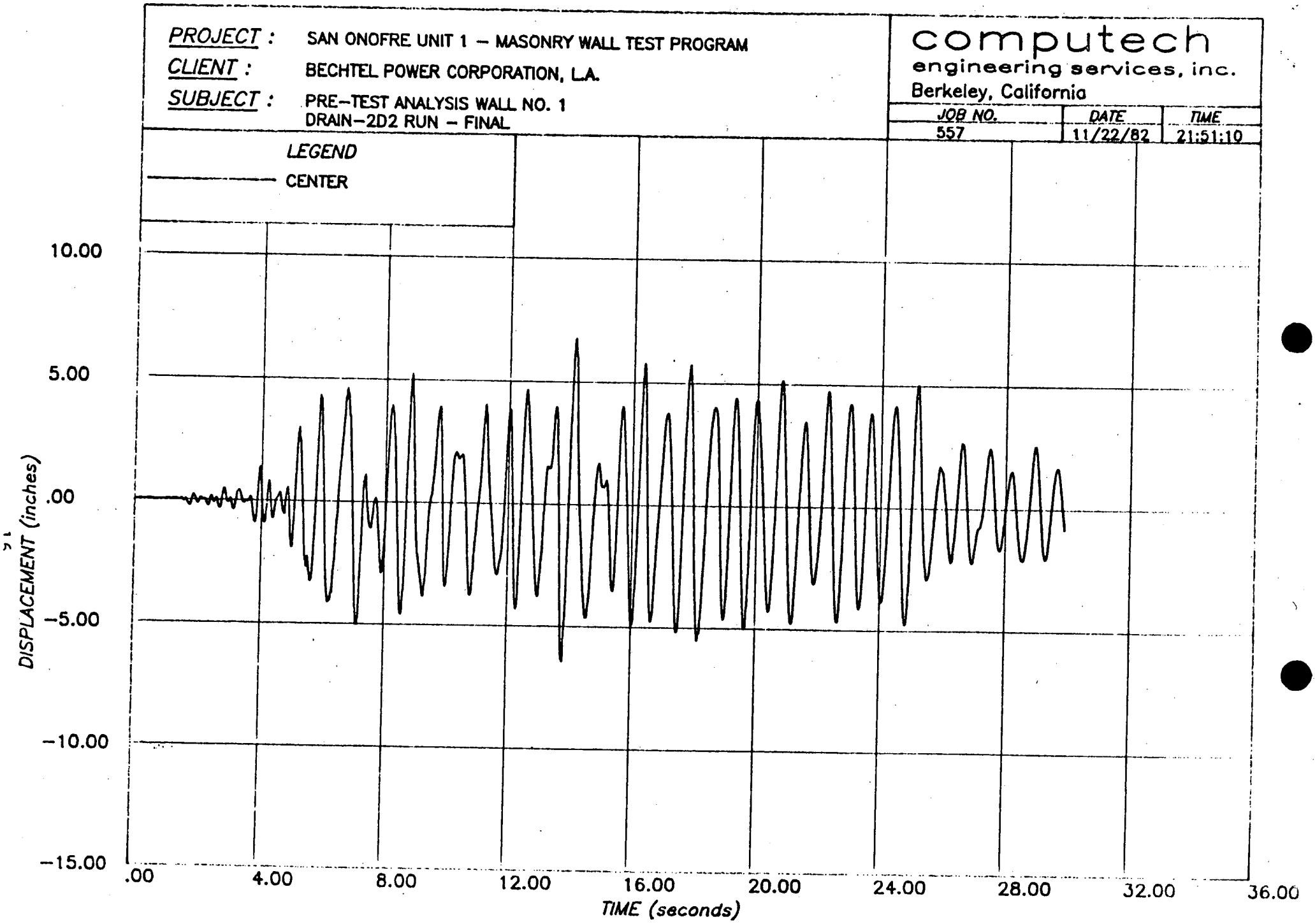

FIGURE 4.1 : TIME HISTORY OF DISPLACEMENT (0 to 30 Seconds)

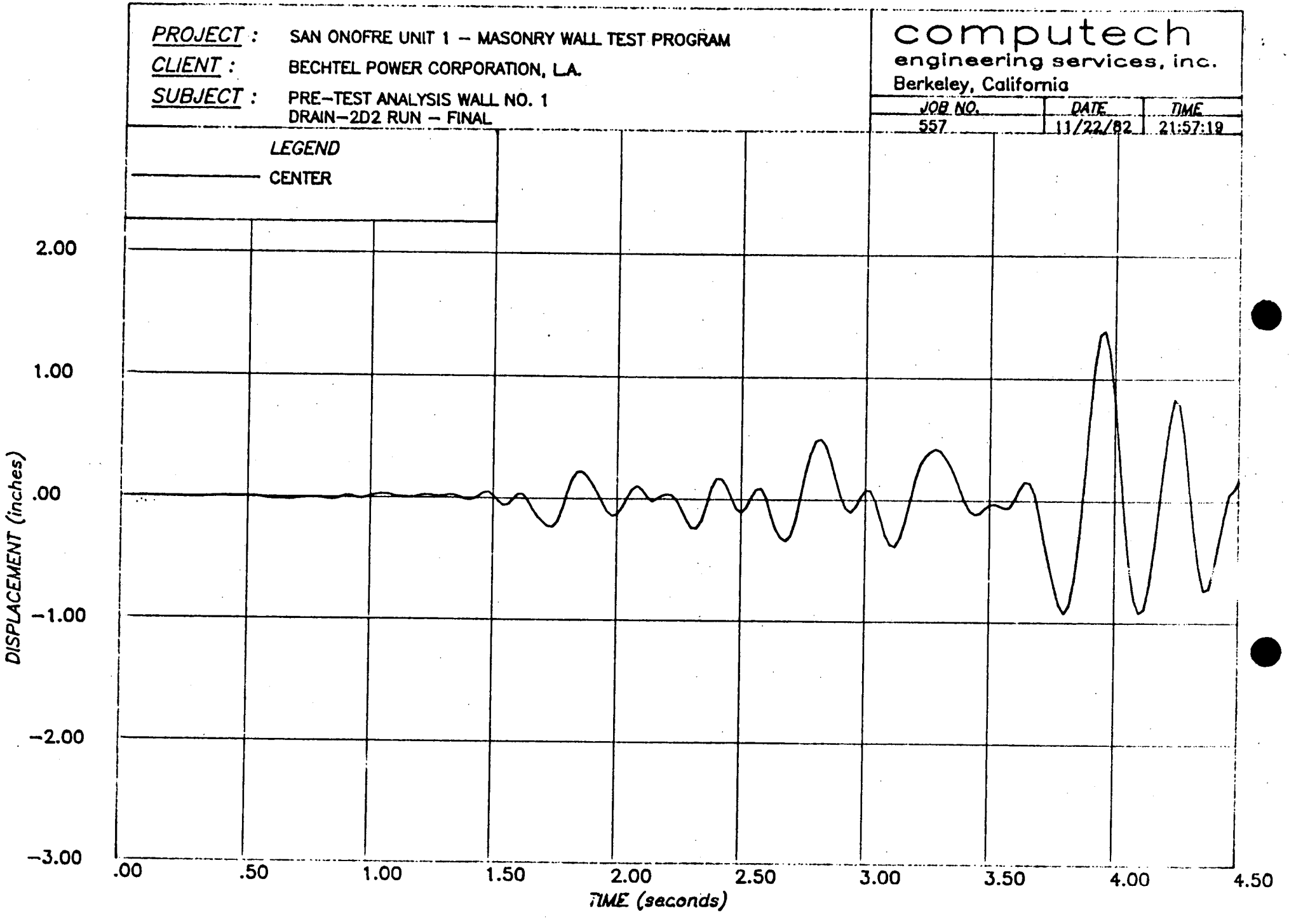

FIGURE 4.2 : TIME HISTORY OF DISPLACEMENT (0 to 4.5 Seconds)

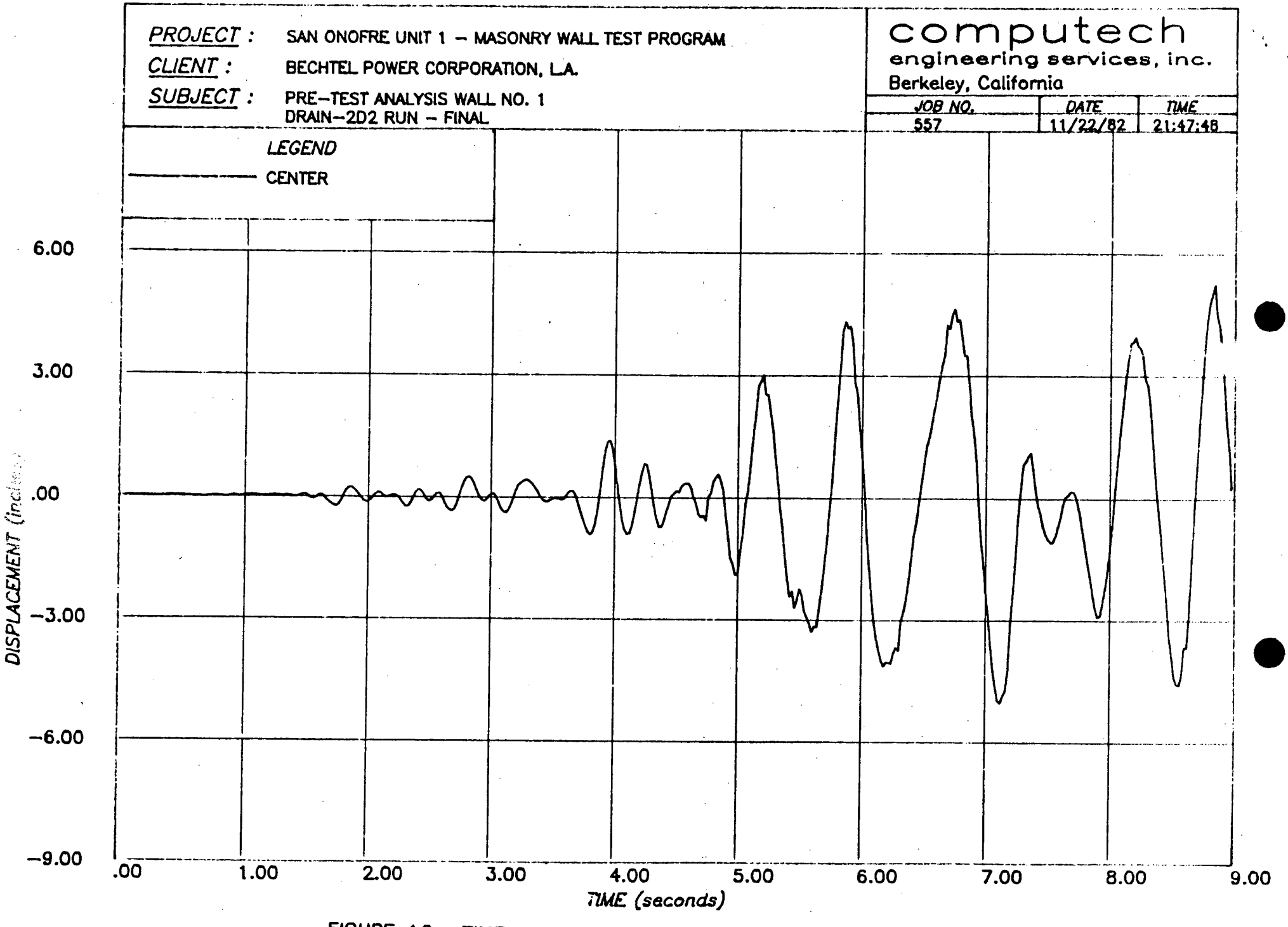

FIGURE 4.3 : TIME HISTORY OF DISPLACEMENT (0 to 9 Seconds)

5 EVALUATION OF RESULTS

The following observations indicate that the pretest analysis results presented in the preceding section conform to the expected wall behavior, and thus are acceptable:

a. The elastic, uncracked period is approximately 0.15 seconds as computed by the mass and stiffness properties.

b. The period gradually increases to approximately 0.4 seconds during cracking but preceding rebar yield. This compares with a period based on experimental evidence of a cracked overall stiffness of 1.5 times the transformed moment of inertia. The ratio of gross moment of inertia to 1.5 times the cracked moment of inertia is 7, and the period is expected to increase by the square root of this factor, i.e. 0.15 x 2.63 = 0.39 seconds.

c. After the central rebar yields further period elongation occurs. The extent of this elongation is unknown pending the results of the test program but the predicted doubling of the cracked period, from 0.4 seconds to 0.8 seconds, appears reasonable.

d. The maximum displacement from the original evaluation of the Fuel Storage Building using the conservative model was 10 inches [Reference 3]. The results reported here predict approximately 70% of this, thus indicating the measure of conservatism which is expected. Note that a direct comparison cannot be made as the time history used in this analysis does not exactly correspond to that used in the Fuel Storage Building analysis.

e. Higher steel strains occur at the base joint than at the upper plastic hinge. This conforms to the expected response because the elastic moment diagram indicates a higher base moment than the mid-height moment. Thus yielding would be expected to initiate at the base of the wall.

6 CONCLUSIONS

The predicted response of the Wall Panel Type 1 is for a maximum displacement of 7 inches and steel strain ratios of 12 at the base hinge and 7 at the upper hinge. Sensitivity studies suggest a possible variation of from 70% to 130% of these displacement values, i.e. from 5" to 9" maximum displacement. The possible variation in the steel strain ratios incorporates the additional variation in the effective rebar length and base steel strain ratios may vary from 6 to 20 and upper steel strain ratios from 4 to 12. It is expected that the actual response will fall within the central portion of these ranges.

7 REFERENCES

1. Computech Engineering Services, Inc. "Seismic Evaluation of Reinforced Concrete Masonry Walls", Volume 1 : Criteria, San Onofre Nuclear Generating Station Unit 1, Report No. R543-02, January 1982.

2. Computech Engineering Services, Inc. "Seismic Evaluation of Reinforced Concrete Masonry Walls", Volume 2 : Analysis Methodology, San Onofre Nuclear Generating Station Unit 1, Report No. R543-02, January 1982.

3. Computech Engineering Services, Inc. "Seismic Evaluation of Reinforced Concrete Masonry Walls", Volume 5: Fuel Storage Building Soil Backfill Condition Evalution, San Onofre Nuclear Generating Station Unit 1, Report No. R543-02, September, 1982.