SAN ONOFRE NUCLEAR GENERATING STATION
UNIT 1

SEISMIC EVALUATION OF REINFORCED CONCRETE MASONRY WALLS

VOLUME 2 : ANALYSIS METHODOLOGY

Prepared for:

BECHTEL POWER CORPORATION
Los Angeles, California

Prepared by:

COMPUTECH ENGINEERING SERVICES, INC.
Berkeley, California

January, 1982

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TABLE OF CONTENTS

1 INTRODUCTION

This volume details the methodology for the inelastic out-of-plane analysis of centrally reinforced masonry walls. The methodology was developed in a number of stages, and details of each of these stages are summarized in the following sections.

The basic formulation of the model was developed at an early stage and retained throughout the initial feasibility and parameter studies. The experimental data used in the model formulation is described in Section 2 and the principles used in developing the subsequent non-linear models are outlined in Section 3.

An initial study examined the feasibility of using existing non-linear computer programs to model the yielding response of masonry walls. This study used the ANSR-II program to compare the predicted response with experimental results for a cyclically loaded wall. Good correlation was obtained for the plastic hinge model although the elastic model required a factored stiffness to accurately reflect bending deflections. A summary of this feasibility study is given in Section 4 of this volume.

The feasibility studies indicated that (a) existing computer programs were adequate to model inelastic masonry walls and (b) maximum ductility demands would be within the ductile capacity of centrally reinforced walls. Based on these results more development work and a parameter study were subsequently performed. This study refined the basic model developed during the feasibility study phase and quantified parameters for the modelling of actual walls at the San Onofre, Unit 1. Section 5 details the work carried out at this stage.

In Section 6 a brief description of the computer programs used in the analysis is given, and Section 7 summarizes the methodology developed in the previous sections for modelling the actual walls at the San Onofre, Unit 1 plant.

2 EXPERIMENTAL DATA

While a considerable amount of research effort has been expended studying the effects of out-of-plane loads on masonry walls only a limited amount is directly applicable to the walls at San Onofre, Unit 1. Much of this data is summarized by Omote et al. in Reference 1. From this available information appropriate results have been extracted which meet the criteria of the San Onofre, Unit 1 walls, i.e.

(1) single wythe, centrally reinforced grouted walls
(2) low vertical loads, and
(3) loading into the inelastic range, both monotonically and cyclically.

Using these criteria two series of experimental results were selected from the literature for the verification of the models developed. One of these series was monotonic and one cyclic. It should be noted that each series of tests was pseudo-static. These results are used throughout the development of the methodology as a basis for which to verify the response predicted by the theoretical models.

An extensive series of out-of-plane monotonic tests has recently been performed by the Structural Engineers Association of Southern California. The tests indicated that the concrete block walls were capable of withstanding significant out-of-plane displacements.

2.1 Monotonic Load Tests

Dickey and Macintosh (Reference 2) tested a series of walls similar to those at San Onofre, Unit 1. In the tests 8'-0" wide strips of wall spanning 20'-0" vertically were loaded by an air bag, on one side of the wall, to deflections greater than the yield level. All walls tested had an equal amount of reinforcing and the variable was the size and spacing of the reinforcement. Examples of the load-deflection curves obtained are reproduced in Figures 2-1 and 2-2. It can be seen that each wall deformed beyond the yield level without strength degradation. In private communications, Dickey indicated that the walls were capable of deforming to much greater deflections than those indicated in Figures 2-1 and 2-2. The deflection measuring devices were removed at the stage indicated on the figures because the wall impacted the reference frame used to measure deflections.

In each test the wall "rebounded" when initial cracking occurred and the load reduced to less than 20% of the value carried in the uncracked condition. This illustrates the reduction in stiffness which occurs when the the tensile capacity of the mortar joints is exceeded. After cracking the stiffness effectively remains constant up to the yield level.

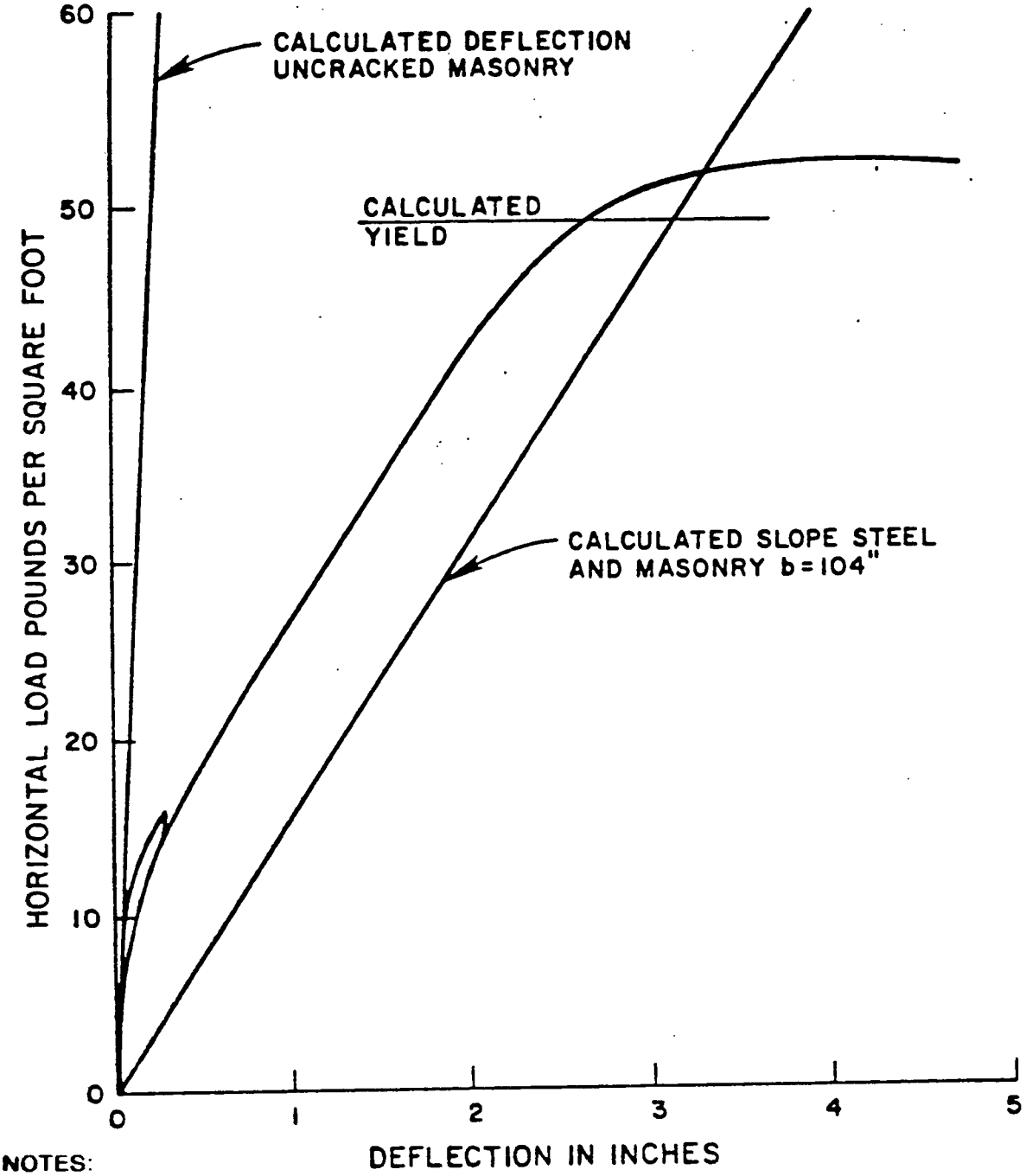
2.2 Cyclic Load Tests

Results reported by Scrivener (Reference 3) are included as part of the

model verification even though they refer to 4" reinforced brick rather than 8" concrete masonry. However, Scrivener notes that the response of brick units is very similar to that of concrete masonry and his results are used because of the limited available data on cyclic loading.

Figure 2-3 shows the results of Scrivener's tests. Very large deflections, up to 8" for a 4" wall, were sustained without strength loss. The hysteresis loop has a very different shape from that for doubly reinforced concrete or steel sections as the load deflection curve passes through the zero point on each load reversal and tends to "grow" with each yield excursion rather than follow a stable path. It is essential that any inelastic model for centrally reinforced masonry be able to track this load-deflection pattern.

Figure 2-3 also shows that the original gross stiffness prior to cracking is non-recoverable. In all cycles subsequent to first loading the stiffness is equivalent to the cracked stiffness no matter how low the level of load. Due of the central position of the reinforcement this ratio of cracked stiffness to gross stiffness is much less than for a doubly-reinforced section.



NOTES:

Vertical bars at 24" o.c.
Effective Depth = 3.9" (Measured)
Test Rebounded at 15.6 psf to 2.6 psf.

(From DICKEY & MACKINTOSH, reference 2)

FIGURE 2-1 : EXPERIMENTAL LOAD DEFLECTION CURVE FOR REINFORCED MASONRY

PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
DRAWN		2-1
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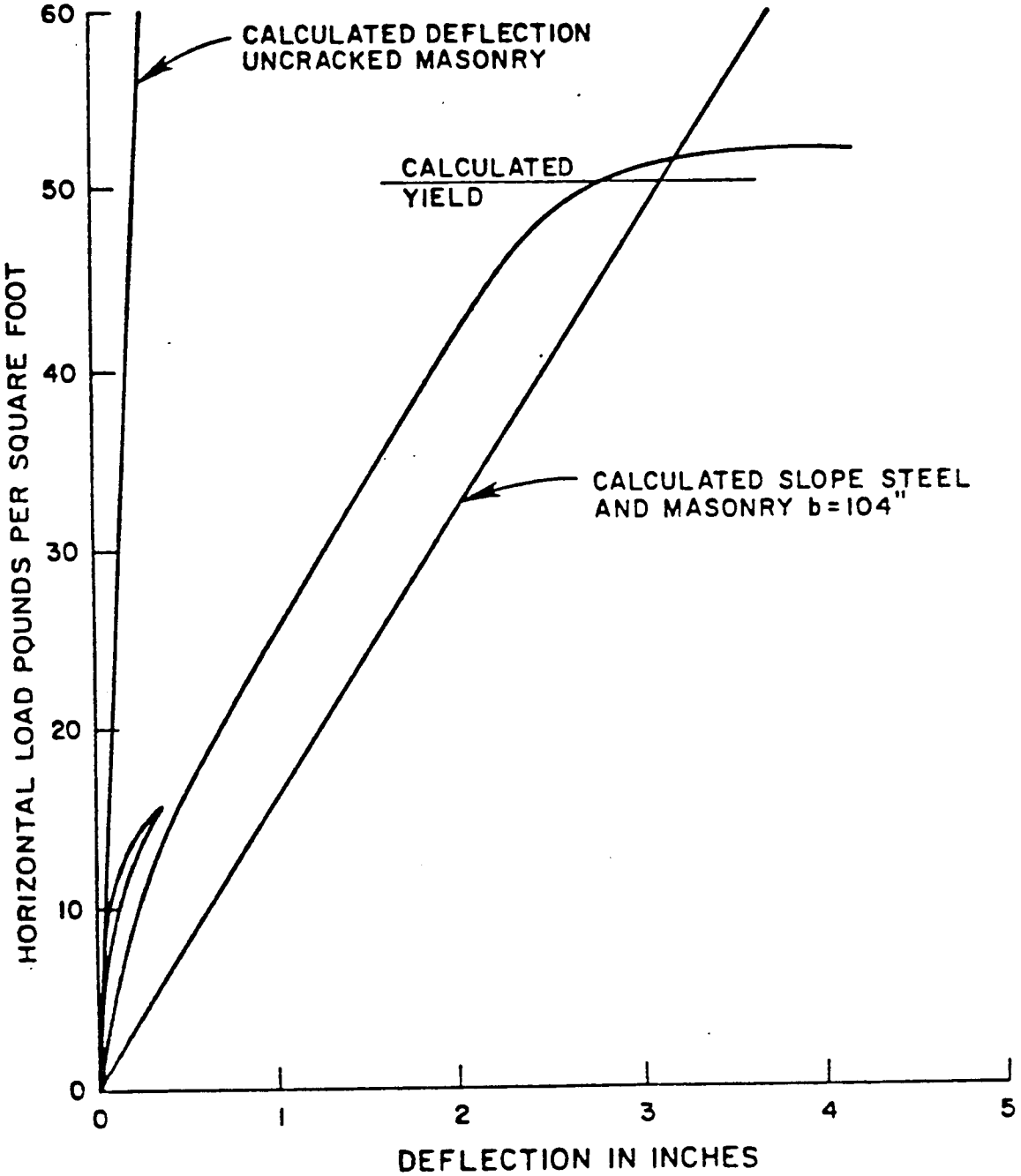

NOTES:

1- #7 Bar at each end of wall (Bars at 8'-0" o.c)
Effective Depth = 4.0" (Measured)
Test Rebounded at 15.6 psf to 2.6 psf.

(From : DICKEY & MACKINTOSH, reference 2)

FIGURE 2-2 : EXPERIMENTAL LOAD DEFLECTION CURVE FOR REINFORCED MASONRY

PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
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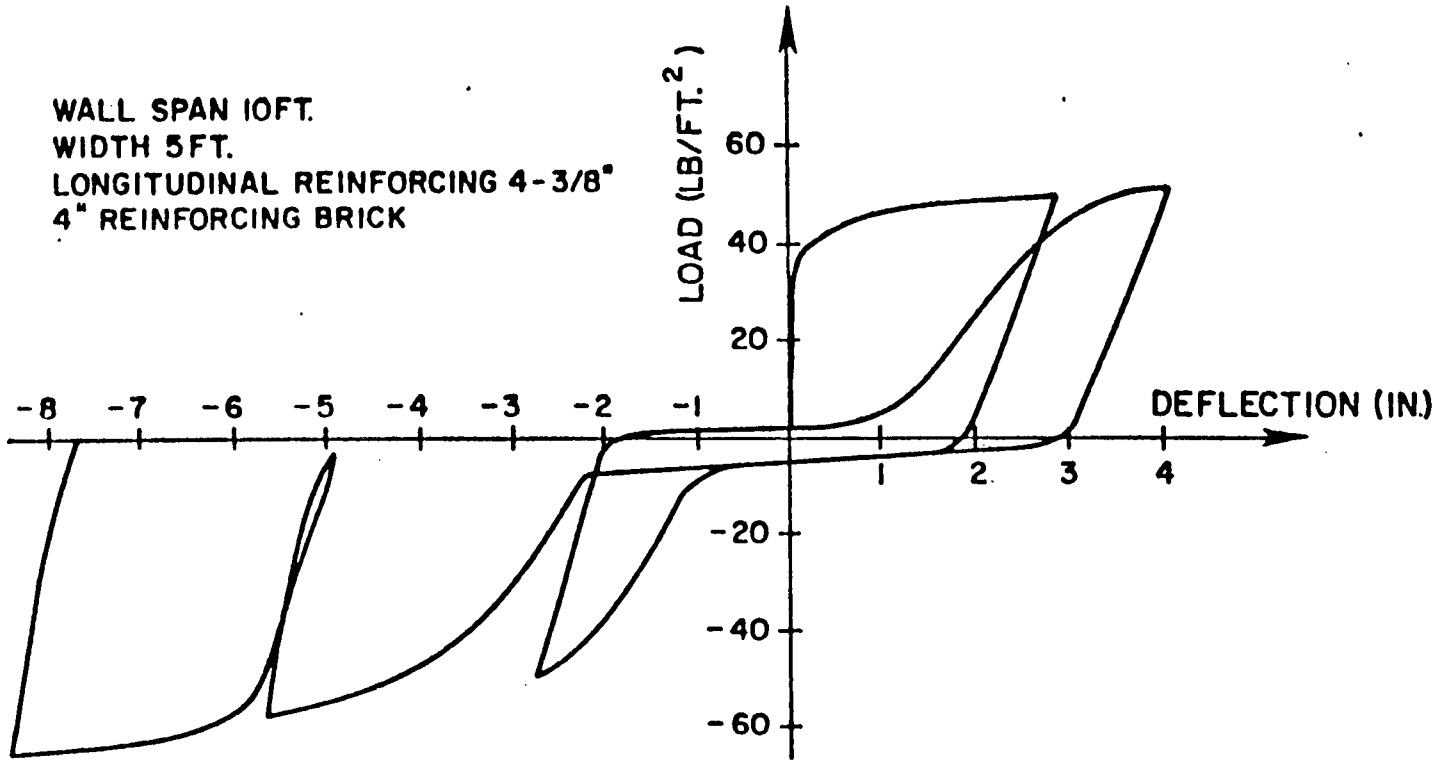

(From : SCRIVENER, reference 3)

FIGURE 2-3 : EXPERIMENTAL CYCLIC LOAD CURVE FOR REINFORCED BRICK

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SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1

FIGURE NO 2-3

3 MODEL FORMULATION

The experimental results reproduced in Figures 2-1 , 2-2 and 2-3 identified three stages in the deflection response of a masonry wall loaded beyond yield level:

(1) Uncracked, with gross stiffness
(2) Cracked, with transformed stiffness, and
(3) Yielded, with strain hardening stiffness only.

In the model derivation an attempt to duplicate the uncracked properties was not necessary since the gross properties apply only up to low load levels in the initial loading cycle. More important was that the model simulate the predominant bi-linear behavior with the two slopes formed by the cracked and yielded stiffness respectively. It is clear from Figures 2-1 and 2-2 that using purely cracked stiffness would overestimate the deflections up to yield, and so the model should reflect this actual stiffness behavior. The model should also account for the loss of stiffness under cyclic loading. In the following sections model requirements to incorporate each of these features are discussed.

3.1 Cracked Stiffness

A masonry wall bending out-of-plane is weakest at the mortar joints and cracks tend to form at the joints rather than being evenly distributed as in a monolithic reinforced concrete member. Once a crack forms at a joint the total stiffness at that section is formed by the face shell in compression together with the reinforcing bars which initially remain elastic in tension. Most rotation therefore occurs at the joints rather than in the masonry blocks which remain elastic.

A model of this cracked joint would therefore require three elements - the two face shells which are elastic in compression and carry zero tensile force plus the central reinforcing bar which initially remains elastic in tension and compression. Because of the lower moment of inertia at the joints, steel stresses are higher there than at the adjacent uncracked blocks. However, because a finite bond length is required for the steel to change from a high to a lower stress the effective length of the joint will be considerably more than its nominal thickness. The length of the central reinforcing element would therefore be a variable in any joint formulation incorporated in a model.

3.2 Yielding Stiffness

As the load on the wall increases the reinforcement at the point of maximum stress will reach its yield point and deform plastically. For mild steels a definite yield point is reached followed by varying amounts of strain hardening. Once yield level is reached (at the point of maximum moment for a uniformly reinforced wall) in a statically determinate system (such as a simply supported wall) the laws of statics dictate that no extra load can be carried by a system without strain hardening. With strain hardening the load carried may

increase slightly. In this respect behavior after the onset of yielding differs from that after the onset of cracking - with the initiation of cracking the load carrying capacity drops but rebounds with added deflection so that cracking progresses along much of the wall length. After yielding, the moments increase only slightly and the plastic hinging is restricted to the region adjacent to the point of maximum moment.

A model for the yield mechanism would therefore be similar to that for the cracked joint in that three elements would be required - two face shell elements remaining elastic in compression and having zero tensile strength plus a central element to model the rebar. In this case the rebar element would deform plastically in tension. The model would also differ in that because of the high stresses the effective bar length would be longer than in the joint model due to bond deterioration and the restricted area over which hinging occurs.

3.3 Cyclic Response

The bi-linear behavior of monotonically loaded masonry as discussed in the preceding sections is similar to other structural materials such as steel or reinforced concrete. Upon load reversal however Figure 2-3 shows that the hysteresis loop differs significantly from that for other materials.

The general form of this curve may be derived by considering a simple cantilever strip as shown in Figure 3-1. As the face shells can take only compression, equilibrium requires that the reinforcing always be either unstressed or in tension. Therefore the steel deforms plastically in tension for both positive and negative moments exceeding the yield capacity. With each yield excursion a gap at the plastic hinge position grows progressively larger and on load reversal the section has practically zero stiffness until the gap closes, i.e. until a deflection equal in magnitude but opposite in sign to that reached in the previous cycle is attained.The load deflection diagram for such a section has the form shown in Figure 3-2.

It is this unique hysteretic behavior that distinguishes centrally reinforced masonry from other forms of construction. The ability to capture this behavior is the most important aspect of the development of an appropriate analytical model.

3.4 Modelling

The previous sections have demonstrated that both the cracked and yielded stiffness could be modelled with three elements, two face shells with a central bar element. The elements modelling the yielded condition would be longer than those modelling the cracked joints because of the higher stresses at yield and the consequent longer length over which the full bar stress extends. However the cracked joints would extend over a greater length of the model Provided only one face shell was stressed at any particular loac step the hysteresis behavior would be correctly modelled. Figure 3-3 shows the general form of such a model.

Two general purpose non-linear computer programs were available, ANSR-II (Reference 4) and DRAIN-2D (Reference 5). These programs are discussed in more detail in Section 6 of this volume. Because of its greater versatility for equivalent static load analysis ANSR-II was chosen for the initial model verification work and then DRAIN-2D was used for time history analyses once the model had been refined.

Each program contained both plane stress and truss bar elements. The latter may be either elastic-plastic in both tension and compression or may be elastic-plastic in tension and buckle under compression. Thus they may be used to model the face shells if a change in direction of forces is admitted, i.e. elastic in tension with no compressive capacity. As the central rebar has equal properties in both tension and compression and as the section is symmetrical about this bar it is apparent that such a change does not violate equilibrium. Therefore a model may be constructed using existing elements in the available non-linear programs. Because the truss bars can carry no shear equal displacements must be enforced across the joint so that there is no shear deformation at the joint. Therefore the joints provide only rotation with no relative displacement. Because of this the model must be dimensioned so that the length of the elements between the joints is equal to the actual center-to-center joint spacing.

Within these constraints a model was developed and successively refined through the initial feasibility studies and the parameter studies reported in the following sections.

(a) P = 0

CANTILEVER UNLOADED

(b) P = Py
DOWNWARD YIELD LOAD
Steel at yield in tension

(c) P > Py
DOWNWARD LOAD GREATER THAN YIELD
Steel deforms plastically in tension.

(d) P = 0
ZERO LOAD

Elastic recovery but steel has permanent plastic deformation.

(e) -P << -Py
UPWARD LOAD MUCH LESS THAN YIELD
Cantilever returns to horizontal position but gap open at root due to steel plastic extension.

(f) -P < -Py
SMALL INCREASE IN UPWARD LOAD
Steel is unstressed until a negative deflection equal to the positive deflection of the previous cycle is reached.

(g) -P = -Py
UPWARD LOAD EQUAL TO YIELD
Steel yields in tension

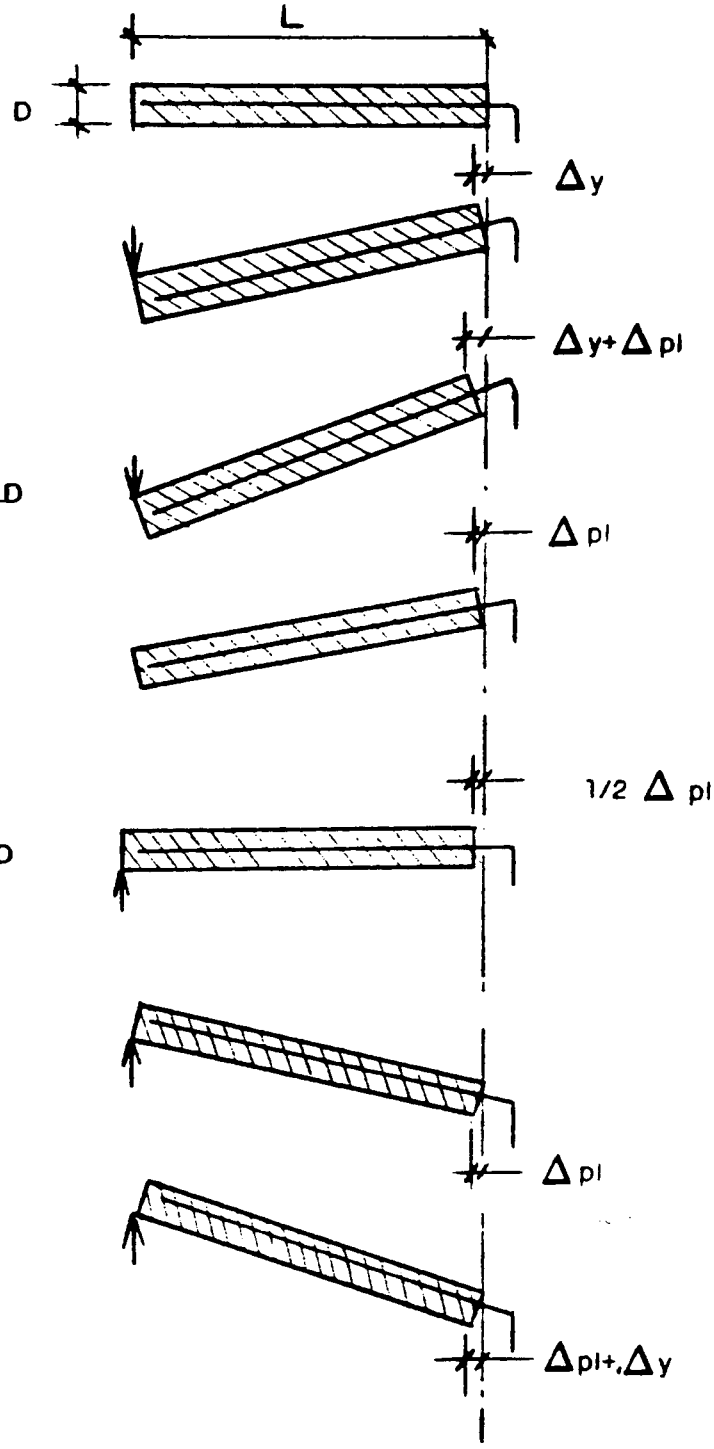

FIGURE 3-1 : IDEALISED BEHAVIOR OF A CENTRALLY REINFORCED CANTILEVER

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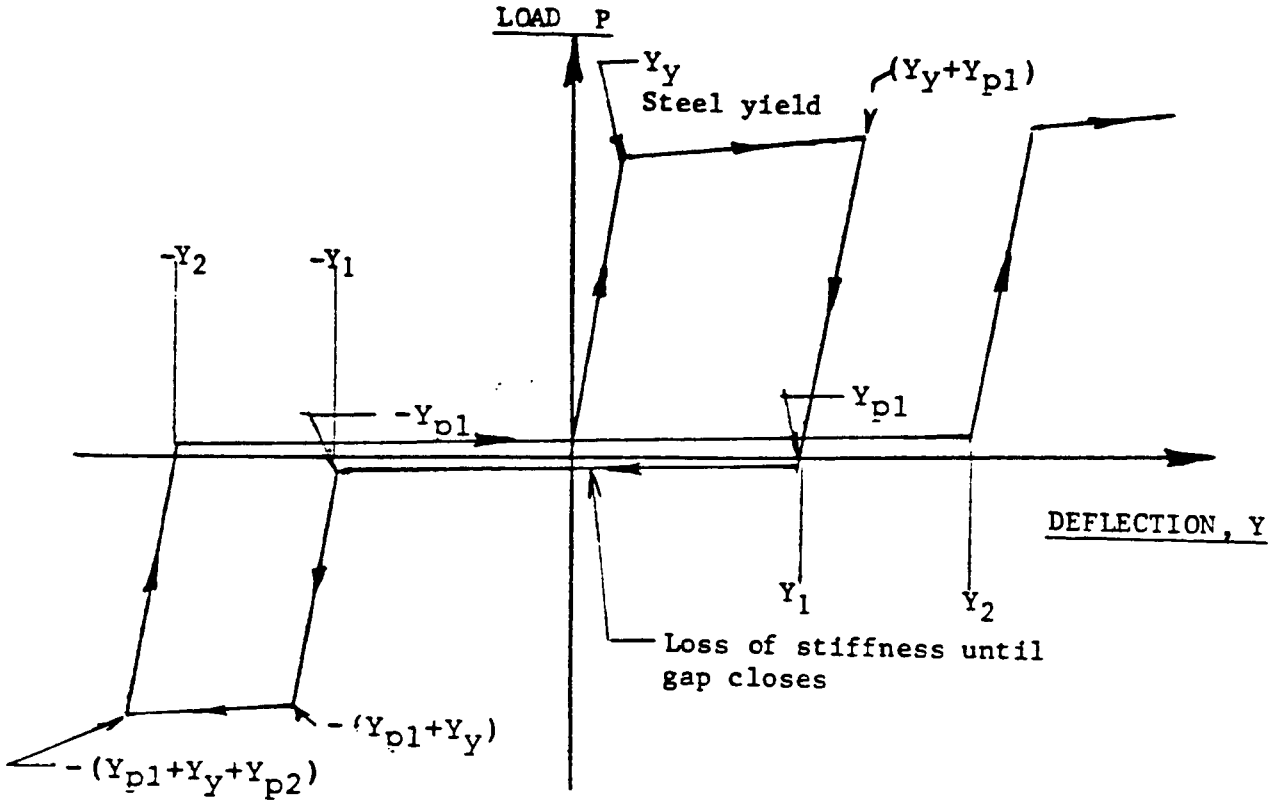

FIGURE 3-2 : LOAD DEFLECTION DIAGRAM FOR IDEALISED CANTILEVER

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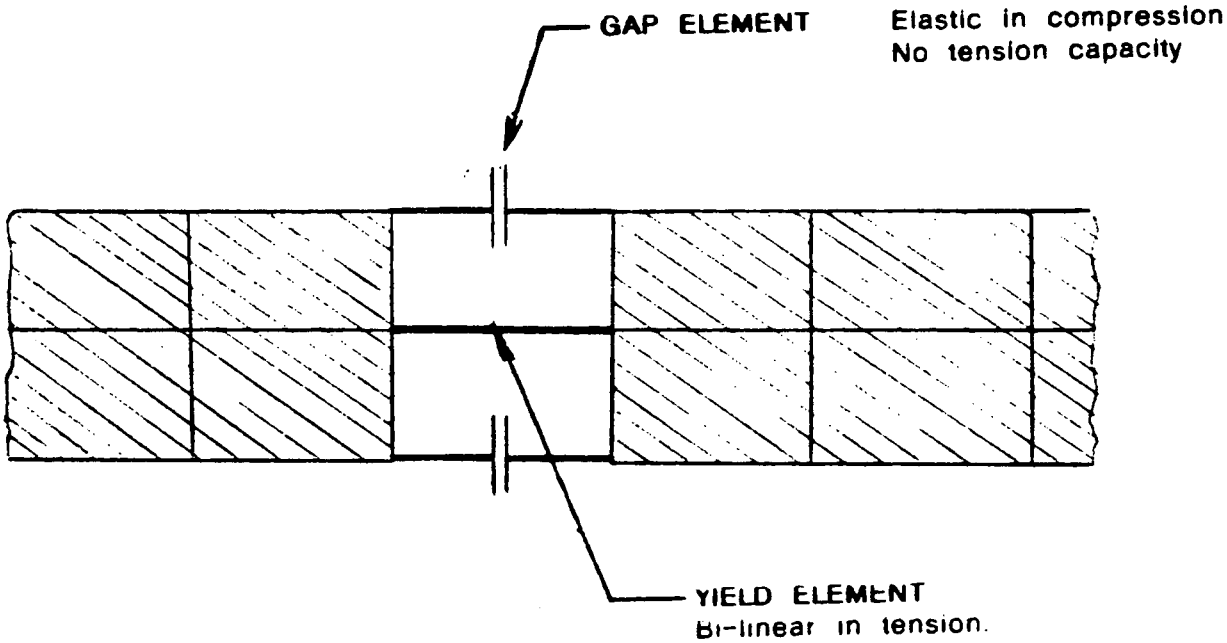

FIGURE 3-3 : MODEL FOR CRACKING AND YIELDING JOINT

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4 FEASIBILITY STUDY

4.1 Introdution

The purpose of this phase of the study was to determine the applicability of inelastic modelling techniques to the out-of-plane response of centrally reinforced walls, using existing computer programs with a minimum of modification. Accordingly, the ANSR-II program (Reference 4) from the University of California, Berkeley, was used and the walls were modelled using elements available in the program.

A simply supported beam model was studied in the following three steps:

(1) verification of elastic model
(2) verification of plastic hinge model
(3) trial time history analyses

A total of six time history analyses were run using a single ground acceleration record and varying the strength and stiffness of the model.

This summary of the initial studies includes the model formulation and verification and the results from a series of time history studies. The areas in which further model development was required were identified and this formed the basis for the more detailed parameter studies, which are summarized in Section 5 of this volume.

4.2 Derivation of Model

The model was formulated for the ANSR-II program using the theoretical basis and procedures derived in Section 3. Using the central truss bar with a gap element on each side a model was coded and model verification carried out in two stages:

(1) Elastic curves compared with experimental values
(2) Inelastic hysteresis curves compared with experiments

The results of these verification analyses are summarized in the following sections.

4.3 Model Verification

4.3.1 Elastic Component

A 1'-0" wide strip of wall was modelled as shown in Figure 4-1, using cracked properties for the section. The thichness of the plane stress elements was selected so as to give the correct cracked moment of

inertia of the wall as calculated from the transformed section. Similarly, the area of the concrete face shell was computed so that together with the central rebar element it would give the correct moment of inertia.

Two models were constructed using these properties, differing only in that model 1B had a mesh spacing one half that of model 1A. Each model was analyzed for a uniformly applied load, giving the central displacement shown in Figure 4-2. For model 1A the displacement was 51% of the theoretical value and for 1B 86% of theoretical.

As expected the finite element formulation was stiffer than the theoretical calculations would indicate, and the solution tended to converge as the number of elements increased. The elements as incorporated in ANSR-II do not include incompatible bending modes which tend to improve flexural properties.

Although model 1B provided a reasonably accurate stiffness it also required a large number of equations. Therefore the coarser mesh of model 1A was used and the thickness adjusted to give the correct stiffness, i.e. reduce B by a factor of 1/0.51. This was appropriate because the stiffness of the elastic elements remains constant.

4.3.2 Plastic Hinge Component

The model used to verify the proposed three component formulation for the plastic hinge is shown in Figure 4-3. The dimensions and sections properties were calculated to match as closely as possible the wall tested in Figure 2-3. Loading was applied in increments as shown in Figure 4-4 and was selected in an attempt to match the measured hysteresis loop. The load was applied in steps with the step size decreasing in the hinging region.

The computed load-displacement history was as shown in Figure 4-5 and this should be compared with the experimental results of Figure 2-3. The postulated model exhibits very similar behavior to the test results and numerical values are similar.

4.4 Time History Analysis

Model 1A was selected for the time history analysis. The area of reinforcement in the wall was assumed equal to #3 bars at 36" which provided a strength level such that first yield would occur at a uniform load of 0.2 times the wall weight, i.e. at 0.2g uniform acceleration. The analyses at this stage were intended mainly to confirm the validity of the modelling procedures. Only one time history was used, the first 5 seconds of the N-S component of the 1940 El Centro earthquake.

4.4.1 Wall Conditions Analyzed

All analyses assumed an 8" nominal thickness wall, simply supported and 16'-0" long with #3 bars at 36". For runs 2 and 5 a yield strength of 40 ksi was used. For runs 1 and 4 the steel strength was increased to very large values to obtain elastic response and for runs 3 and 6 the steel strength was taken as 60 ksi to give a yield level equivalent to 0.3g. Note that only the strength and not the stiffness was altered.

4.4.2 Displacements

The time history of central displacements for runs 1, 2, 3 and 4, 5, 6 are given in Figures 4-6 and 4-7 respectively. There is a marked difference in response between the two stiffness levels: using the low I value throughout the elastic displacements were larger than the yielding displacements whereas when the cracked I value was used only in the hinge region inelastic displacements were several times those of the elastic model.

The use of these two different stiffness values provide an upper and lower bound on the actual wall stiffness. In the parameter study described in Section 5 results of experimental tests are used to obtain more realistic values for the effective I.

4.4.3 Ductility Requirements

Table 4-1 lists maximum deformations and associated ductility requirements for each analysis. The displacement ductility, i.e. ultimate displacement divided by the first yield displacement was far greater for the runs with the stiffer element, runs 4 and 5 than for the "soft" elements in runs 2 and 3. However the maximum steel ductility was similar for both stiffness values implying similar plastic hinge rotations.

4.5 Concluding Remarks

The feasibility study was performed to determine the applicability of inelastic modelling techniques to the out-of-plane response of centrally reinforced walls. It is clear from the preceding sections that these techniques are applicable The following general comments are a result of the feasibility study.

(1) present techniques are satisfactory to model the inelastic out-of-plane response of masonry walls.

(2) use of cracked I over only the central portion is more reasonable than over the entire length

(3) inelastic deflections were a function of the

position of the elastic frequency of the wall on the appropriate response spectrum

(4) rotational ductility demands required similar ultimate steel strains for both fully cracked and partially cracked sections.

The feasibility study indicated that although existing programs provided a means of obtaining inelastic response of masonry walls a number of parameters needed further study before actual walls could be analyzed. These included:

- use of different time histories
- effect of damping
- extent of cracking
- plastic hinge length.

In the following section the effects of these parameters and means by which to include them in the model are studied.

	Yield Disp. (ins)	Ultimate Disp (ins)	Displ. Ductility	Steel Yield Disp.	Steel Ultimate Disp.	Steel Ductility Eu/Ey
(a) Fully Elastic	-	10.83	-	-	0.035	-
(b) Yield at 0.3G Static Load	3.64	8.28	2.3	0.0113	0.34	30.1
(c) Yield at 0.2G Static Load	2.51	8.17	3.3	0.0076	0.32	42.1

(a) Using Cracked Moment of Inertia

	Yield Disp. (ins)	Ultimate Disp (ins)	Displ. Ductility	Steel Yield Disp.	Steel Ultimate Disp.	Steel Ductility Eu/Ey
(a) Fully Elastic	-	1.30	-	-	0.056	-
(b) Yield at 0.3G Static Load	0.29	4.06	14.0	0.0113	0.315	27.9
(c) Yield at 0.2G Static Load	0.19	4.83	25.9	0.0076	0.383	50.4

(a) Using Uncracked Moment of Inertia Except at Hinge

TABLE 4.1 : DUCTILITY REQUIREMENTS

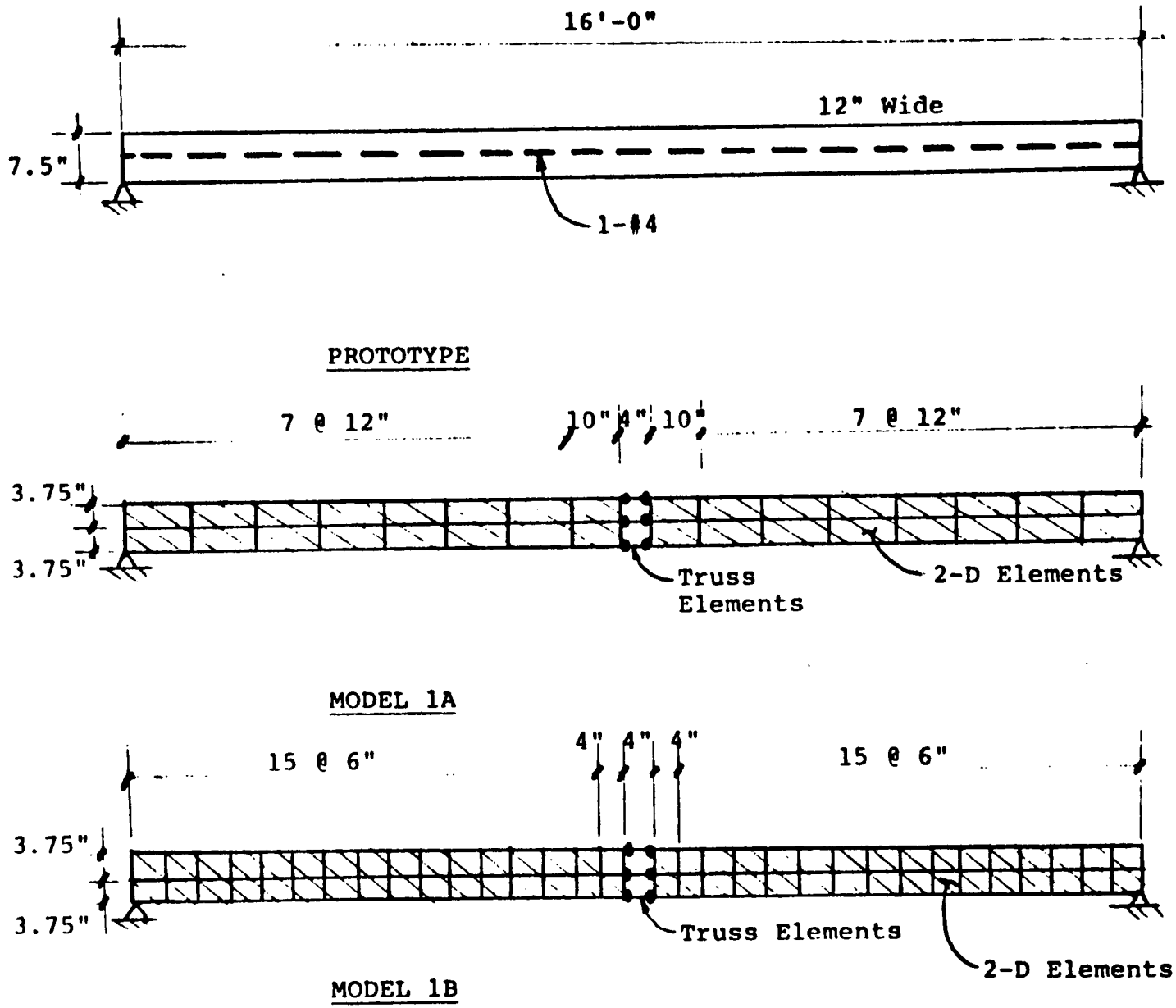

FIGURE 4-1 : MODELS USED FOR ELASTIC VERIFICATION

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FIGURE NO 4-1

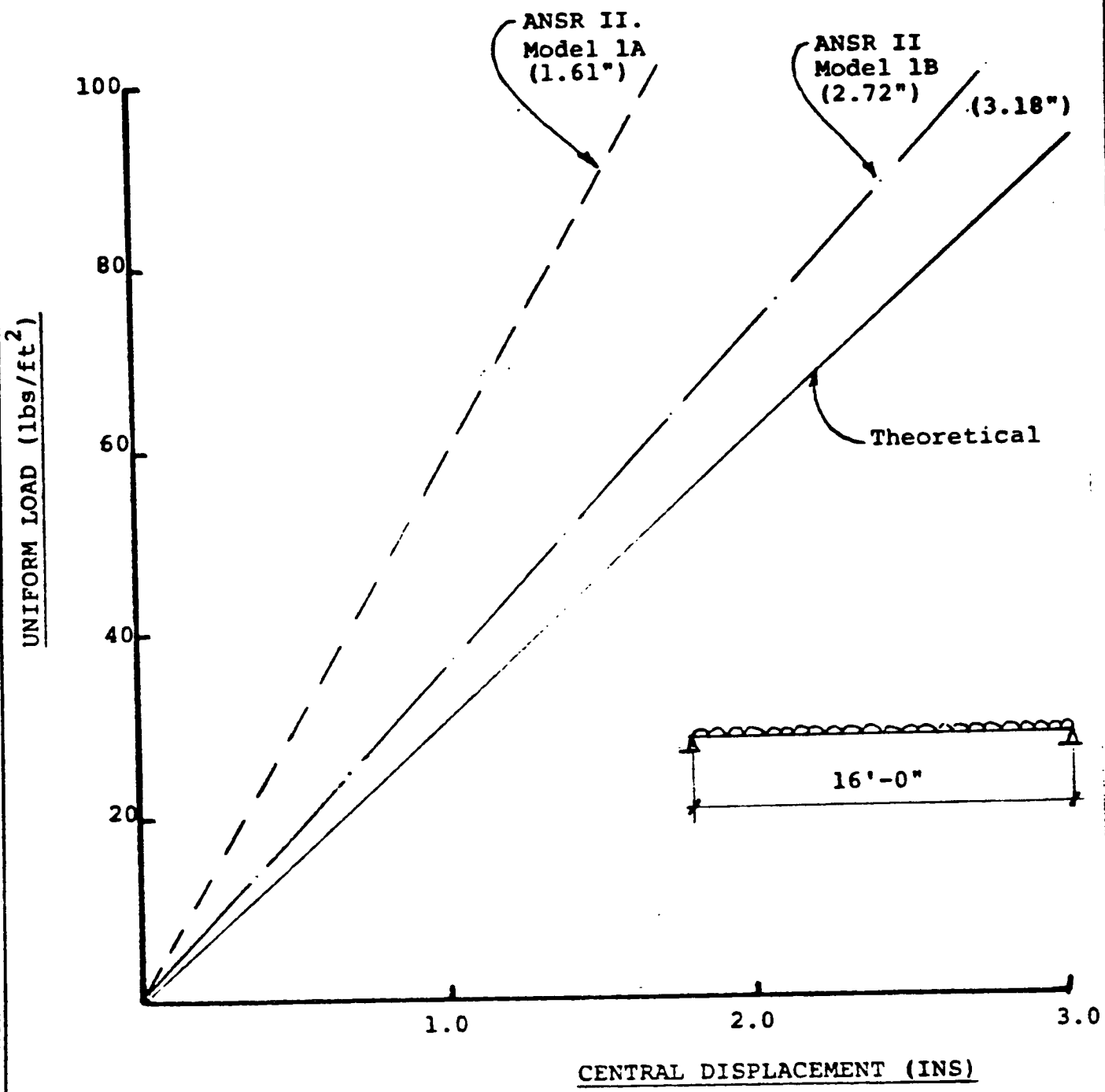

FIGURE 4-2 : MODEL vs THEORETICAL DISPLACEMENTS

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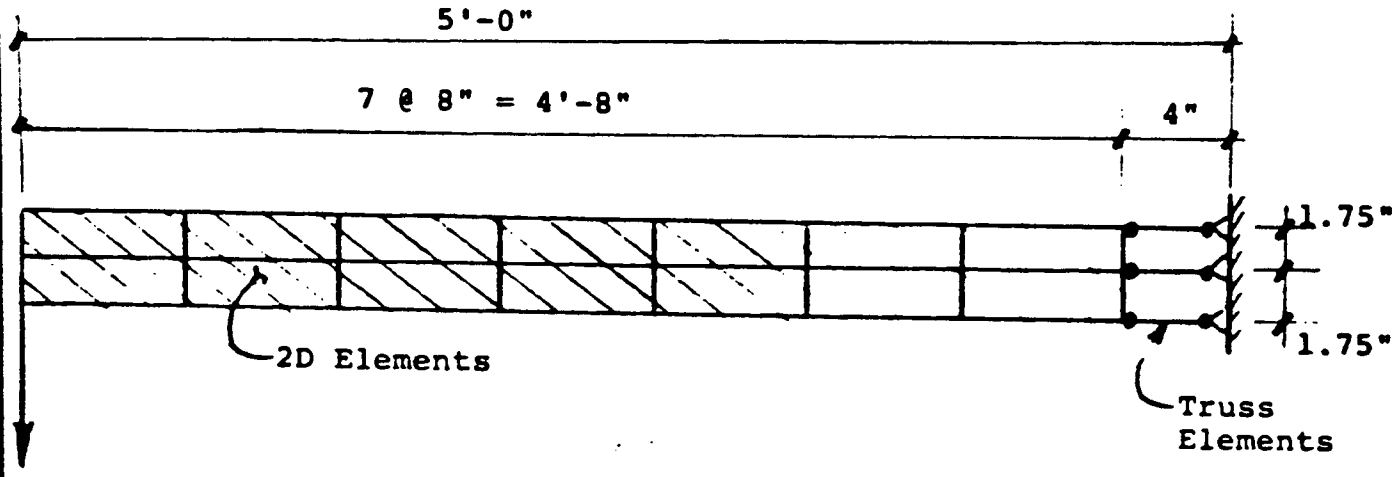
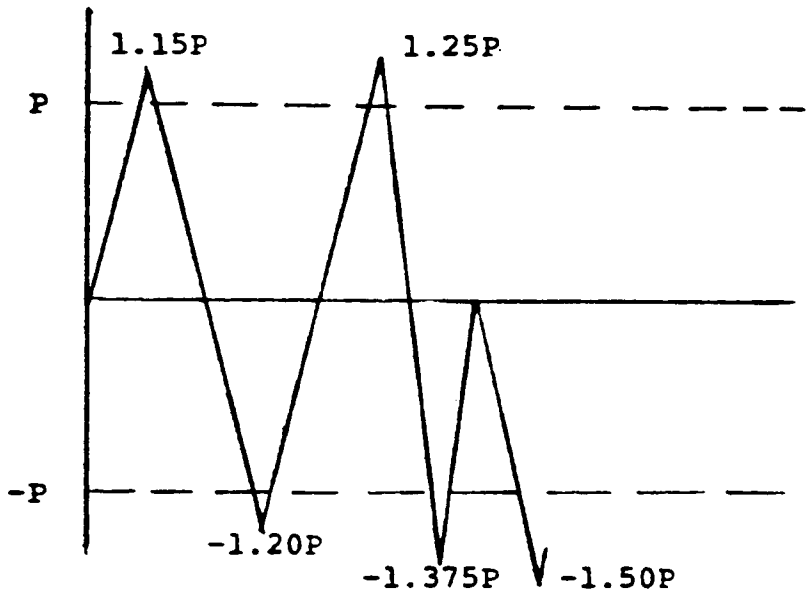


FIGURE 4-3 : MODEL FOR HINGE VERIFICATION



P = Tip Load To Cause Steel Yield

FIGURE 4-4 : LOAD INCREMENTS

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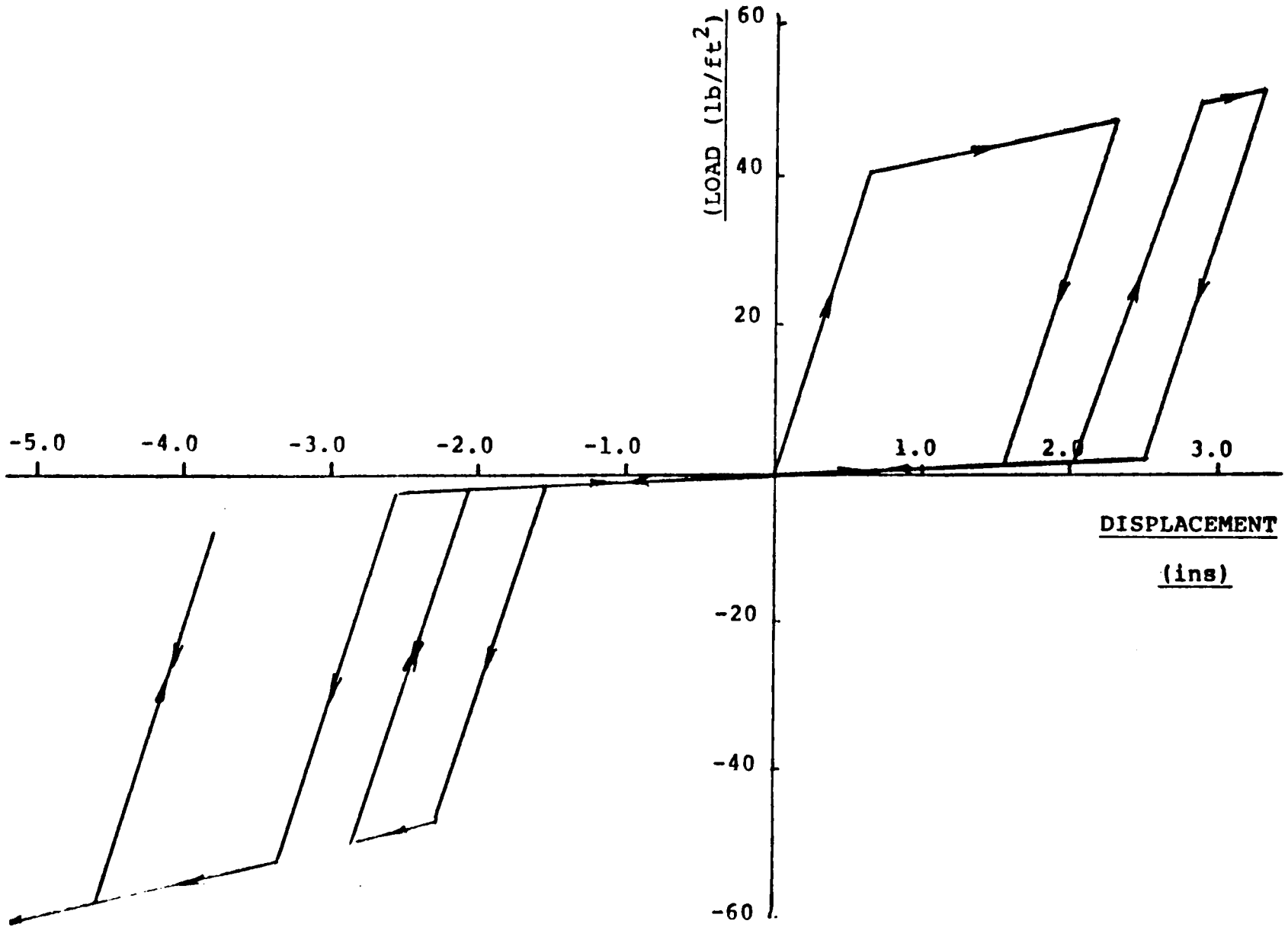

FIGURE 4-5 : RESPONSE UNDER IMPOSED LOAD

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FIGURE NO 4-5

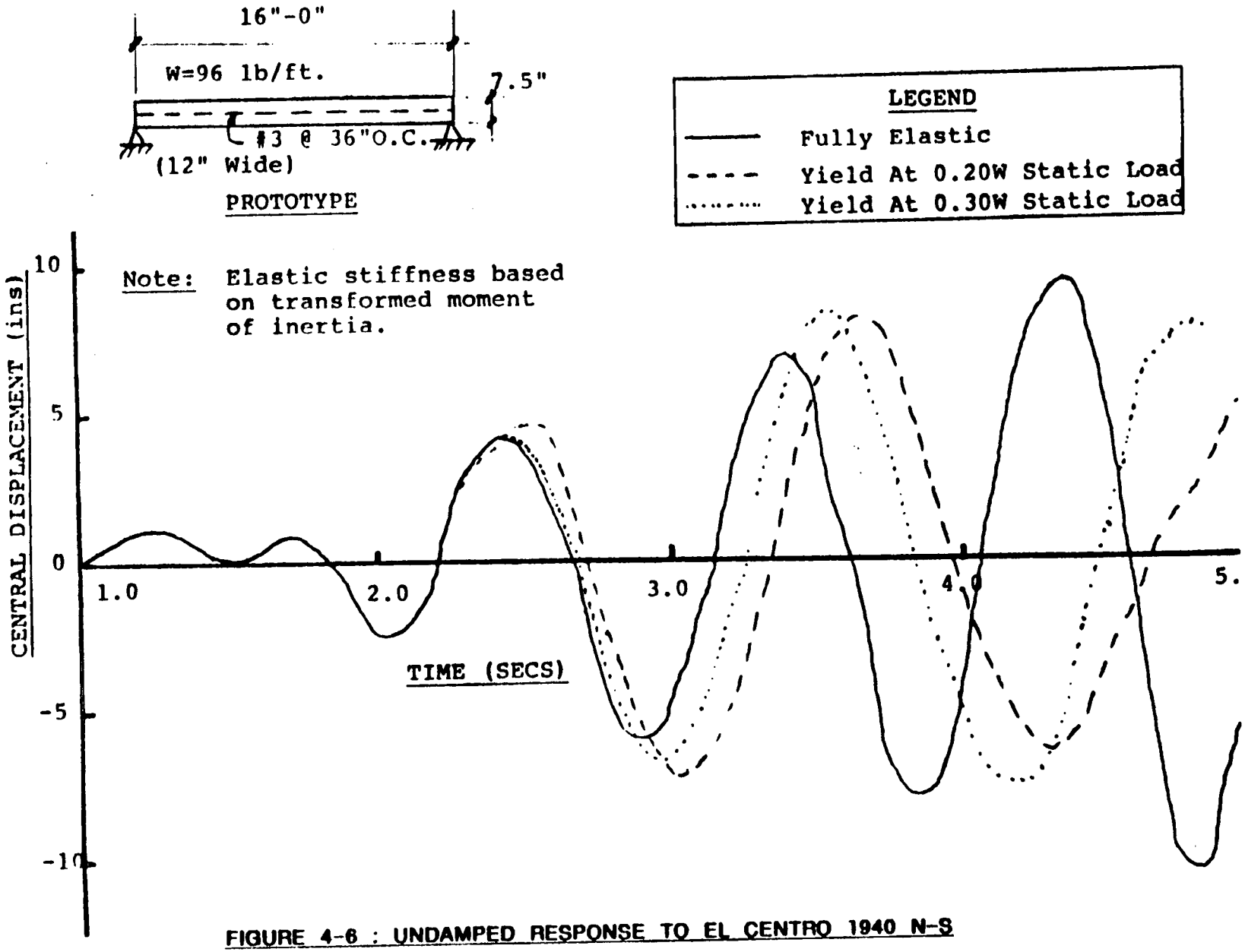

FIGURE 4-6 : UNDAMPED RESPONSE TO EL CENTRO 1940 N-S

(Cracked Stiffness Throughout)

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FIGURE NO 4-6

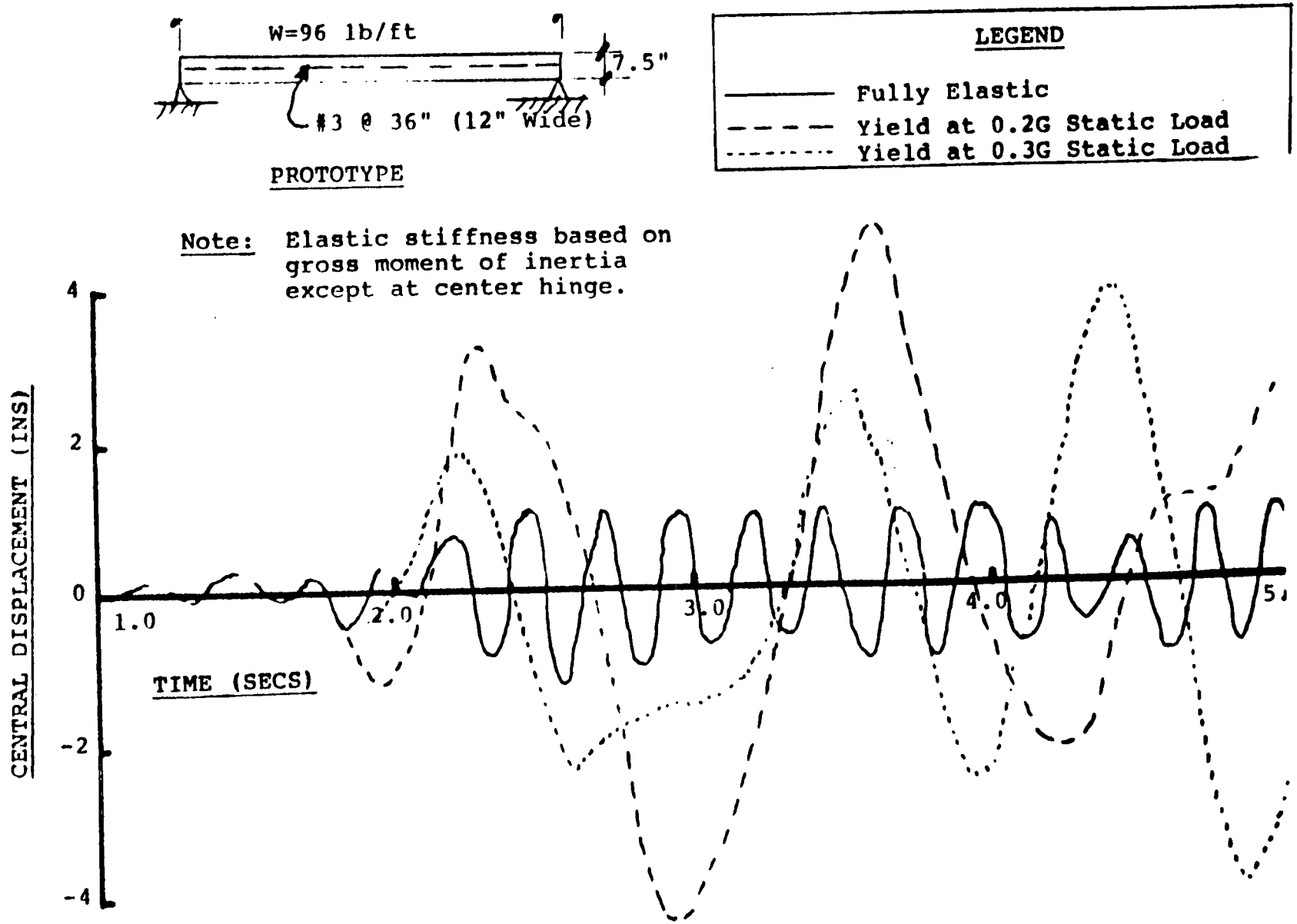

FIGURE 4-7 : UNDAMPED RESPONSE TO EL CENTRO 1940 N-S

(Uncracked Stiffness Except at Hinge)

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FIGURE NO 4-7

5 PARAMETER STUDY

5.1 Introduction

The initial feasibility study summarized in Section 4 of this volume demonstrated that it was practical to model the inelastic response of masonry walls using existing computer programs. The results also indicated that parametric studies were required to refine model properties for actual wall analyses. In this section the results of such parameter studies are reported.

Two basic criteria were used to develop and verify the model:

(1) The physical characteristics of the observed inelastic behavior should be recognizable in the model
(2) Results obtained with the model should closely predict those obtained experimentally.

A model fulfilling these criteria could then be used to study the response of individual walls to a number of time histories, thus providing an envelope of maximum inelastic response. In this report the model derivation is refined and the response predicted by the model is compared with that measured in tests.

Once the model was adequately refined a series of time history analyses were carried out to identify important numerical parameters and solution strategies.

5.2 Derivation of Model

The model was derived using the principles outlined in Section 3 of this report. This basic model was then successively refined by considering elastic elements, cracking elements, plastic hinge length and the cyclic response. In the following sections each of these stages is discussed.

5.3 Refinement of Model Properties

5.3.1 Elastic Elements

The inelastic model used 4 node plane stress finite elements to model the elastic segments between joints. As implemented in the ANSR-II program (and also in DRAIN-2D) these elements do not incorporate incompatible bending modes necessary to accurately model bending behavior with only two elements through the thickness of the wall. That is, the plane stress elements provide an effective EI which is higher than the theoretical value. Rather than substantially increasing the number of elements in the model the same results could be obtained by adjusting

the effective EI of the section to produce the correct bending behavior.

To determine the effective EI a series of analyses was carried out using cantilever and simple beam models. A standard length of 120" was used and the depth varied over 6", 8" and 12" to give L/D ratios of 20, 15 and 10 respectively. Two elements over the depth of the member were used in all cases. Figure 5-1 shows the models used. The cantilever was modelled with a point load and the simple beam with a uniformly distributed load.

Figures 5-2 and 5-3 show the ratio of the deflections computed by ANSR-II to the theoretical deflections plotted against the number of elements. The results proved independent of the width of the elements.

The deflection ratios tend towards a value of 1.0 as the number of elements increases, with faster convergence for lower L/D ratios. The simple beam deflections are more accurate than the cantilever deflections for a given number of elements. The results show that to achieve an accuracy of 90% or better would require at least 30 elements with considerably more for high L/D ratios.

The alternative approach adopted for this study is to adjust the element thickness based on the known ratios of effective stiffness to theoretical stiffness so that the model accurately models deflections. This approximation is acceptable because of two factors:

(1) The elements remain elastic, and so the adjusted stiffness remains constant throughout the analysis

(2) The aim of the analysis is to investigate critical stresses and ductilities at the joints where the masonry is modelled by strut elements. Therefore accuracy of stresses in the plane stress elements is not as critical.

On the basis of these analyses that the thickness of the plane stress elements was adjusted by the ratio of computed to theoretical deflections for the applicable support conditions, number of elements and L/D ratio.

5.3.2 Cracked Joint Elements

To test the ability of the proposed joint model to predict test deflections one example from the Dickey and Mackintosh test series (Reference 2) was selected as a benchmark. This prototype wall is dimensioned as in Figure 5-4(a) and its load deflection curve is as given in Figure 2-1.

For the initial analyses model 2A as shown in Figure 5-4(b) was used with all joints modelled and the joint width being a variable. The model was loaded monotonically in increments of load up to 60 psf and the analyses repeated for joint widths of 1", 2", 4" and 6". Plots of the

load deflection curves are given in Figure 5-5.

Because the model uses the cracked stiffness at all load levels it is not capable of following the experimental load deflection diagram at all points up to yield level - the objective is to match the experimental deflection at the inital yield level and beyond. From Figure 5-5 a joint length of 4" best achieves this aim. The physical significance of this is that for 2" into the block on either side of the joint the steel stress is equal to the joint steel stress and beyond that point the stress reduces because the masonry block remains elastic.

Because cracking and yielding is most significant near the point of maximum moment models 2B and 2C were then analyzed. These models have similar joint details to model 2A in the central portion but the remainder of the beam is modelled with plane stress elements whose thickness is adjusted so as to model the cracked condition. Because cracking is not constant over the length of the beam these "cracked" plane stress elements will have a thickness somewhat greater than that required for the cracked moment of inertia.

Figure 5-6 shows that model 2B, with three joints, closely parallels the response of model 2A when 1-1/2 I cracked is used for the plane stress elements. Model 2C with only one joint did not give good agreement with test results and its use was discontinued. Because model 2B gave similar results to model 2A with fewer elements and nodal points this form was adopted as being sufficiently accurate for further analysis. In Figure 5-7 the effect of varying strain hardening ratios on model 2B is shown. A value of 1% of the original bar stiffness appeared to give best correlation with experimental results and so was adopted for the subsequent analyses.

5.3.3 Yielding Joint Elements

For modelling the cracked joint stiffness the same bar length was assumed effective both before and after yielding. However after yield the spread of hinging is much less than the spread of cracking due to the very low post-yield stiffness Also, because of the high steel stress and bond deterioration the effective length of bar at or near yield stress would be longer. Therefore model 2B was modified as shown in Figure 5-8 to better relate to the physical behavior of the wall at yield.

Because previous analyses had shown that 3-4" joints fitted the experimental results approximately the same total effective bar length, 12", was used. However in this case it comprised 1-10" length plus 2-3/8" lengths. Figure 5-9 shows that the deflections using this model agreed closely with those of the previous model 2B. Because of the better match with physical behavior this modified joint model was adopted throughout the cyclic and time history analyses.

5.3.4 Cyclic Response

To test the ability of the model to predict cyclic behavior a model based on a prototype tested by Scrivener (Reference 3) was used. This model, number 3, was dimensioned as shown in Figure 5-10. Nodal loads simulating uniform loading were applied in the pattern shown in Figure 5-11(a) and the resulting deflection pattern was as shown in Figure 5-11(b).

The load-deflection diagrams from the ANSR-II model and from Scrivener's test have been superimposed in Figure 5-12. The model predicts actual test behavior very well and closely matches the loss of stiffness which occurs under reversing loads. The inital stiffness in the test results is not duplicated but in all subsequent loading and unloading cycles the analytical model stiffness closely parallels that obtained from the test. Note that in this model similar parameters to those found to give a "best-fit" to the monotonic load results were used, i.e. 1-10" yielding bar plus 2-3/8" joints, 1% strain hardening and 1.5 I cracked effective stiffness in the plane stress elements.

At this point it was concluded that a valid model had been obtained for use in subsequent dynamic analyses. An initial model had been developed based on an understanding of the physical behavior, its parameters had been refined using the results of the monotonic tests and the resulting model was shown to reproduce the results of the cyclic tests very well. Thus the basic requirements for acceptability of an analytical model had been satisfied.

5.4 Dynamic Parameter Studies

In Section 5.3 the validity of the postulated model for both monotonic and cyclic loading was confirmed. In this section solution procedures for subjecting the model to a time-history of base accelerations are examined. For this portion of the study the model shown in Figure 5-13 was used. This has the same general proportions as the walls at San Onofre, Unit 1. The model used was similar to those developed in the preceding sections to match the experimental results.

The N-S component of the 1940 El Centro earthquake was used for all runs. To ensure sufficient inelastic response to fully test the model the record was scaled by 1.50 to give a maximum ground acceleration of approximately 0.5g. Only the first 5 seconds of the record were used. The full El Centro record is shown in Figure 5-14 and the response spectrum derived from the first 5 seconds in Figure 5-15. The model had a computed elastic frequency of 1.97 hz.

5.4.1 Elastic Correlation

For the wall analyzed elastically the maximum displacement should be approximately equal to the spectral displacement for a

single-degree-of-freedom oscillator of similar frequency. The spectral displacement was computed based on the product of the displacement times the participation factor times the mode shape component for a distributed mass system. This gave computed wall displacements of 2.46" and 3.81" for damping values of 7% and 0% respectively. Figure 5-16 shows the time history response as computed, giving maximum displacements of 3.12" and 6.78" for the two respective damping values for El Centro scaled by 1.5. Dividing by 1.5 gives values of 2.08" and 4.52" for 7% and 0% damping respectively. The undamped value is thus considerably higher than expected. This could be partly because of higher mode effects but is most likely due to a frequency slightly different than computed. Figure 5-15 shows that a frequency change of plus or minus 10% would increase accelerations by up to 50%. In fact from Figure 5-16 the period in the final cycle measures 0.45 seconds giving a corresponding theoretical spectral displacement of 4.78" which compares well with the computed value of 4.52".

5.4.2 Integration Time Step

For all analyses the step-by-step integration method with equilibrium correction at the end of each time step was used. To determine the sensitivity of the results to the size of the time step three different values were used to obtain the response of the 7% damped wall. Time steps of 0.01, 0.005 and 0.0025 seconds were used corresponding to T/50, T/100 and T/200 respectively where T= the fundamental period.

Figures 5-17 and 5-18 depict the displacement and steel strain response respectively. The latter plot is the ratio of maximum steel strain to yield strain, which gives a measure of the local ductility demand. These figures show very little difference in response for the varying time step size and it was concluded that a time step of 0.02 times the fundamental period was satisfactory.

5.4.3 Damping

A comparison of the effects of damping is shown in Figures 5-19 and 5-20. Damping was introduced using both mass dependent and stiffness dependent damping. The latter damping was applied only to the plane stress elements. This is a good representation of the physical behavior because damping within the joint region is in the form of hysteretic damping which is already accounted for in the yield function.

As implemented in numerical integration procedures in both ANSR-II and DRAIN-2D damping gives rise to unbalanced nodal forces at the end of each time step. An equilibrium correction is therefore made at the beginning of the next time step. Provided the step is small enough this procedure does not effect the numerical stability. However for the reinforced masonry model the unbalanced forces become very large at the time of load reversal when the system has very low stiffness. To overcome this possibility of numerical instability it was found necessary

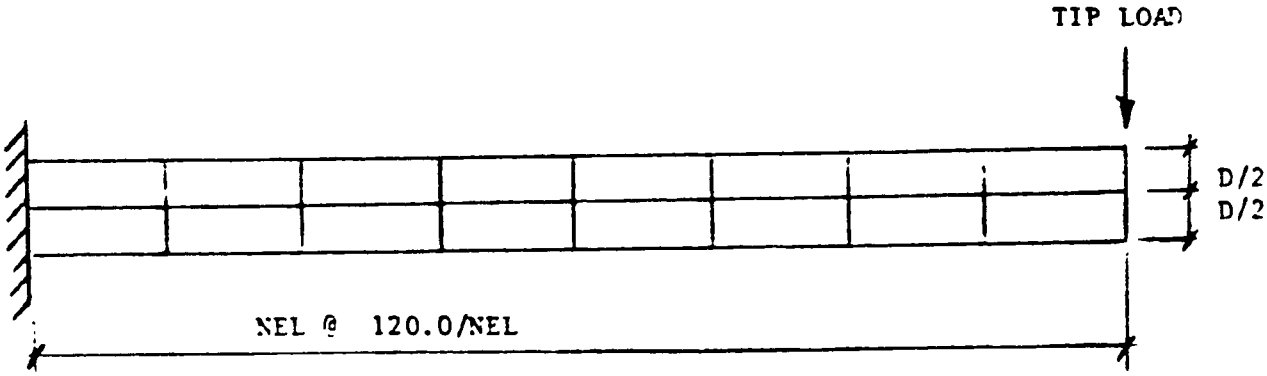
to iterate within ANSR-II whenever unbalanced loads exceeded 1000 lbs. In DRAIN-2D incorporation of the time step repetition feature removes all unbalanced loads and ensures stability (see Section 6.3).

5.4.4 Large Displacement Formulation

The analysis of the undamped yielding model was repeated using the large displacement formulation of ANSR-II. The results of this are given in Figures 5-21 and 5-22. The large displacement formulation generally reduced response. As for damping large displacement effects were applied at the element level and were used for the elastic elements only. The time penalty for incorporating these effects is considerable and therefore they were not used.

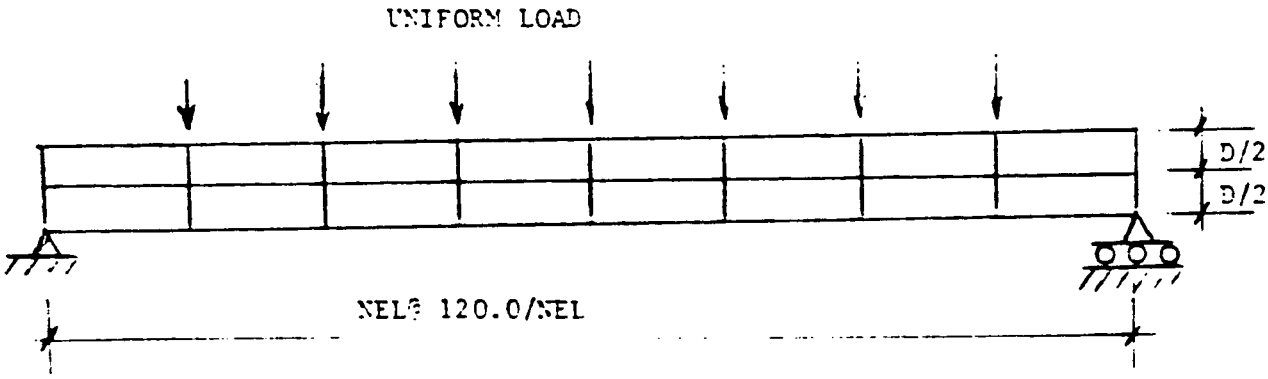
5.5 Summary and Concluding Remarks

This study completed the methodology development stage for the non-linear analysis of masonry walls at the San Onofre. Unit 1 plant. The model was formulated for use with either the ANSR-II or the DRAIN-2D programs and was sufficiently refined to allow the walls at San Onofre. Unit 1 to be analyzed.



(a) CANTILEVER MODEL

D= 6",8" and 12"
NEL=6,8,12,15 and 30 } FOR EACH MODEL



(b) SIMPLE BEAM MODEL

FIGURE 5-1 : MODELS FOR ELASTIC ANALYSIS

PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO 5-1
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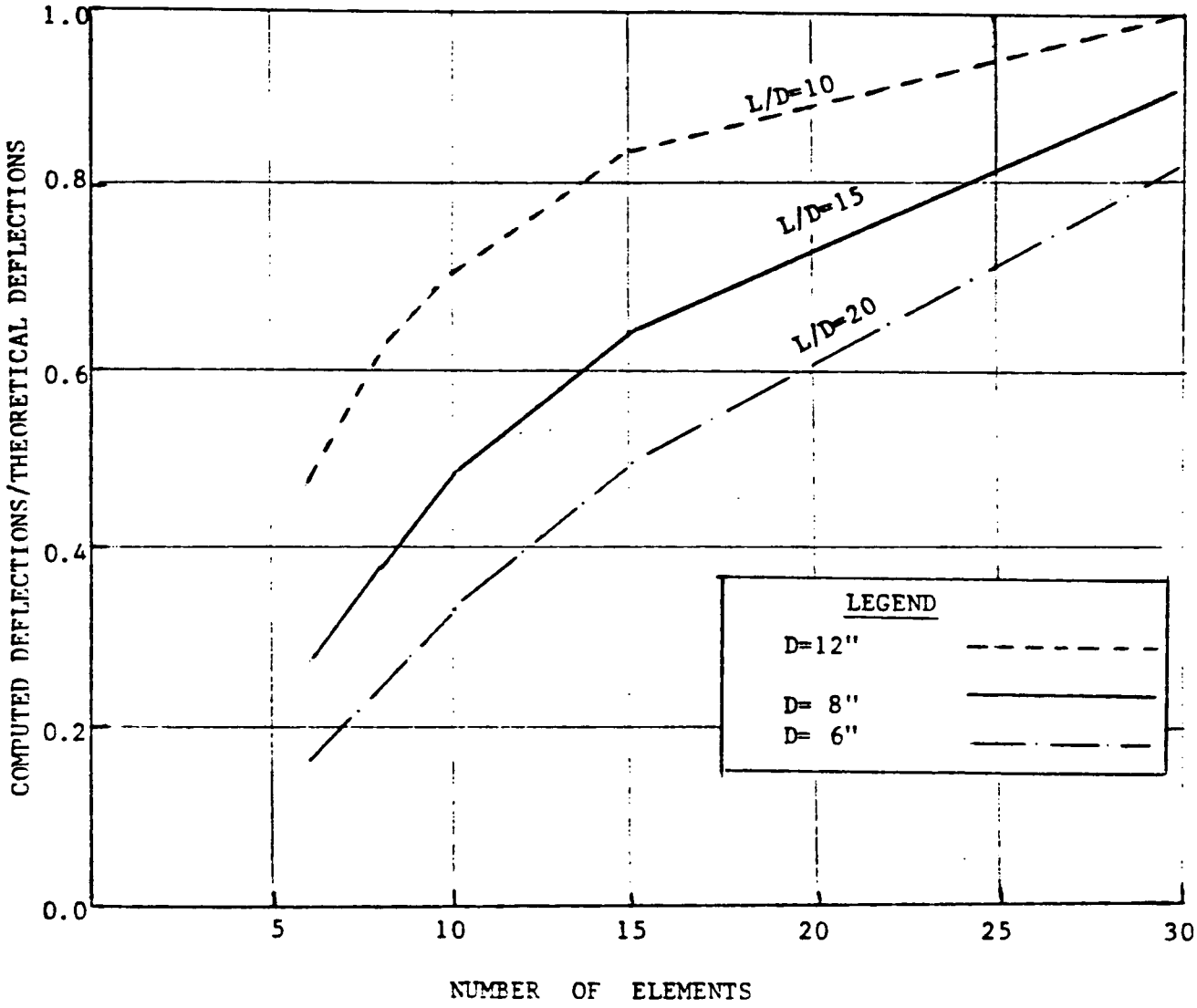

FIGURE 5-2 : SIMPLE BEAM DEFLECTIONS

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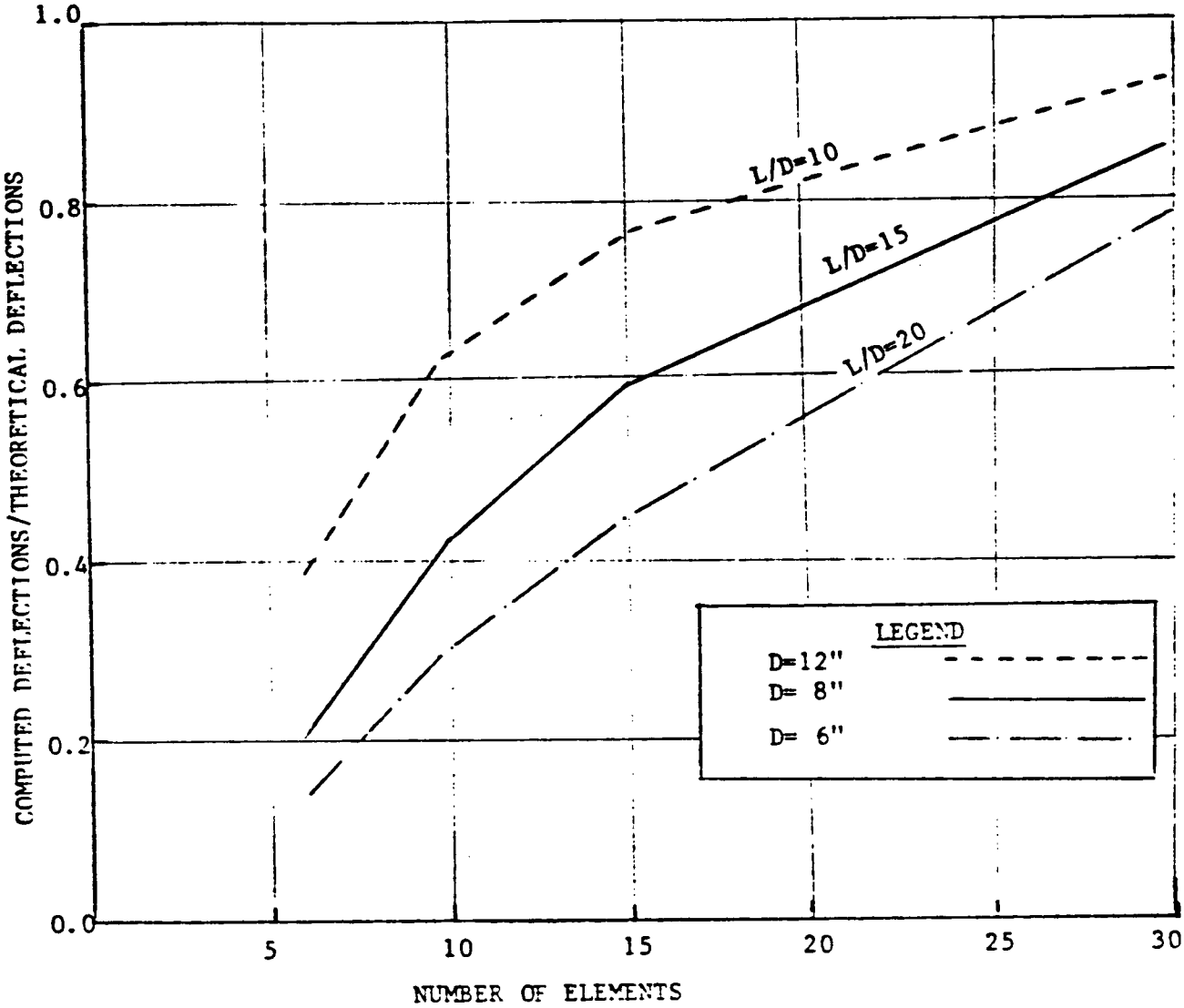

FIGURE 5-3 : CANTILEVER DEFLECTIONS

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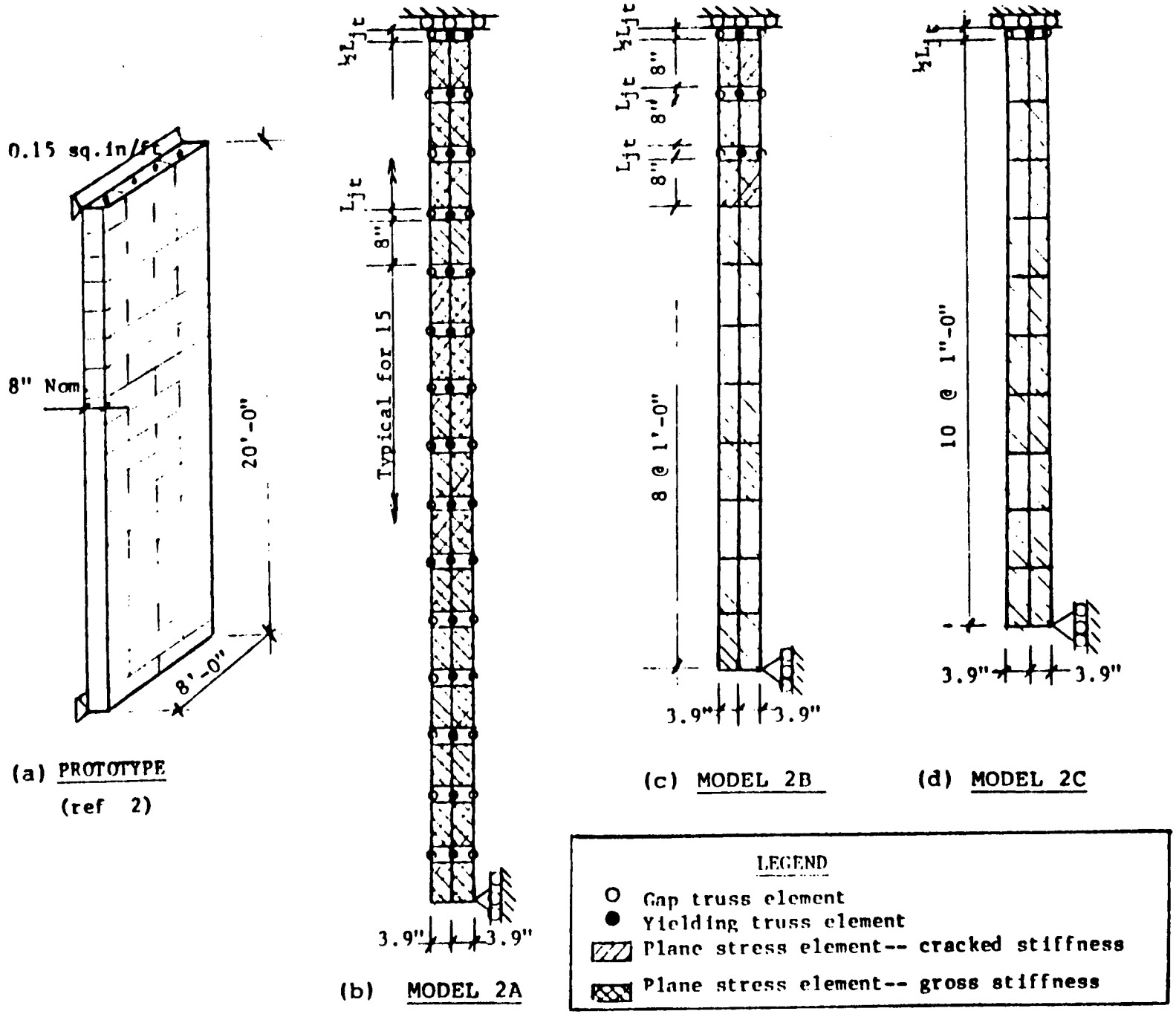

FIGURE 5.4 : MODELS USED TO DETERMINE CRACKED STIFFNESS

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SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1
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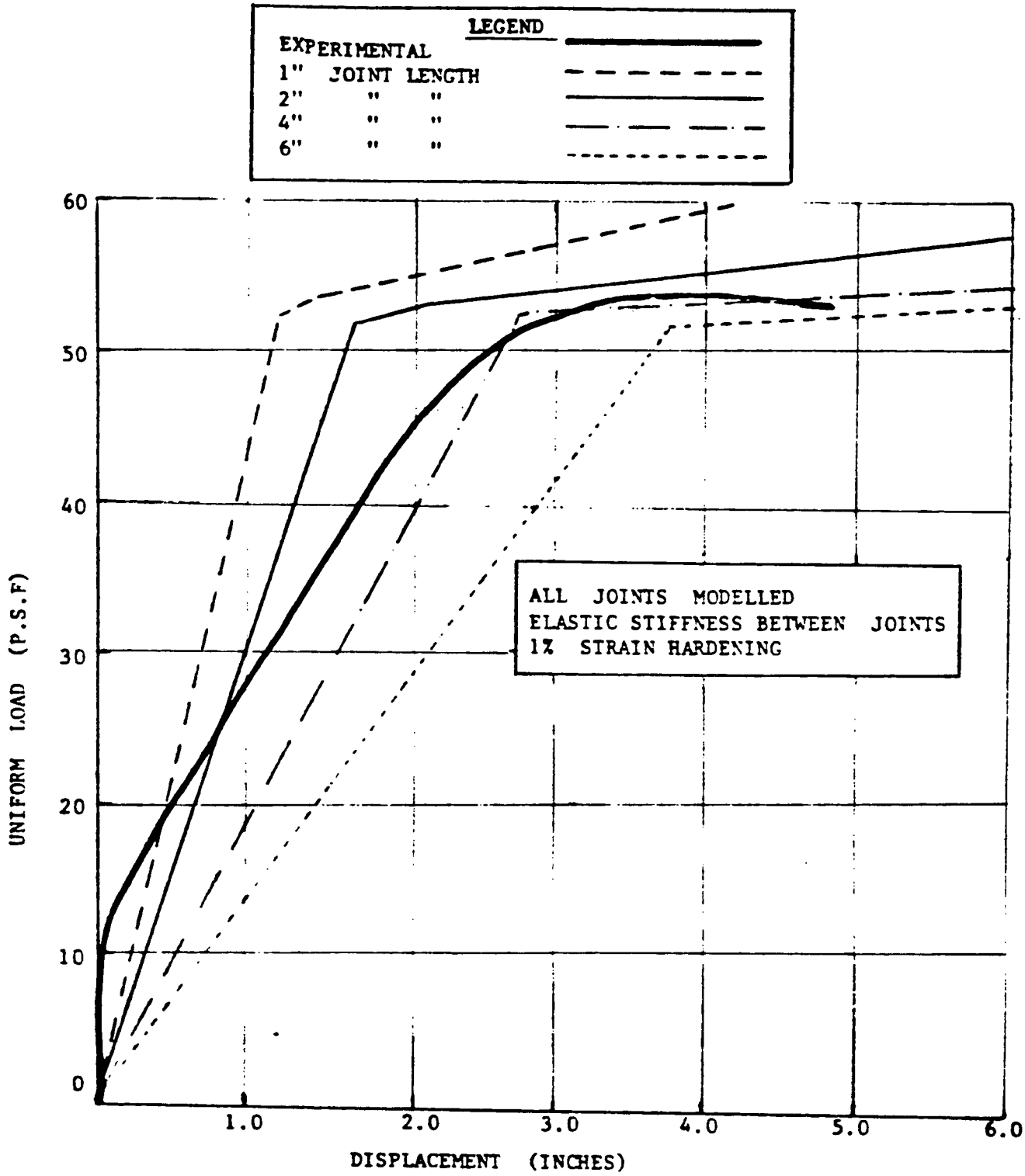

FIGURE 5-5 : YIELDING MODEL — ALL JOINTS INCLUDED

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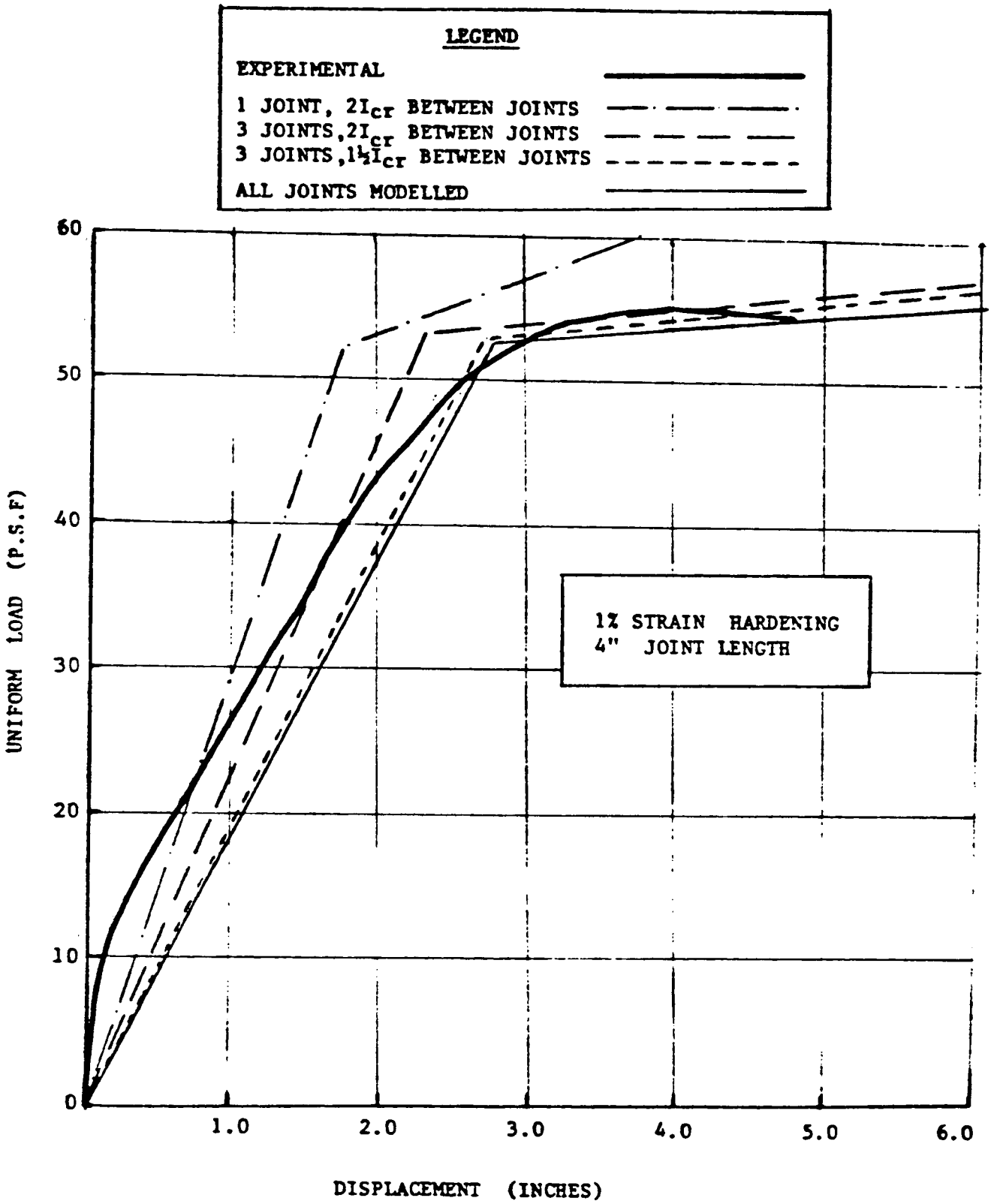

FIGURE 5-6 : YIELDING MODEL --- VARIABLE NUMBER OF JOINTS

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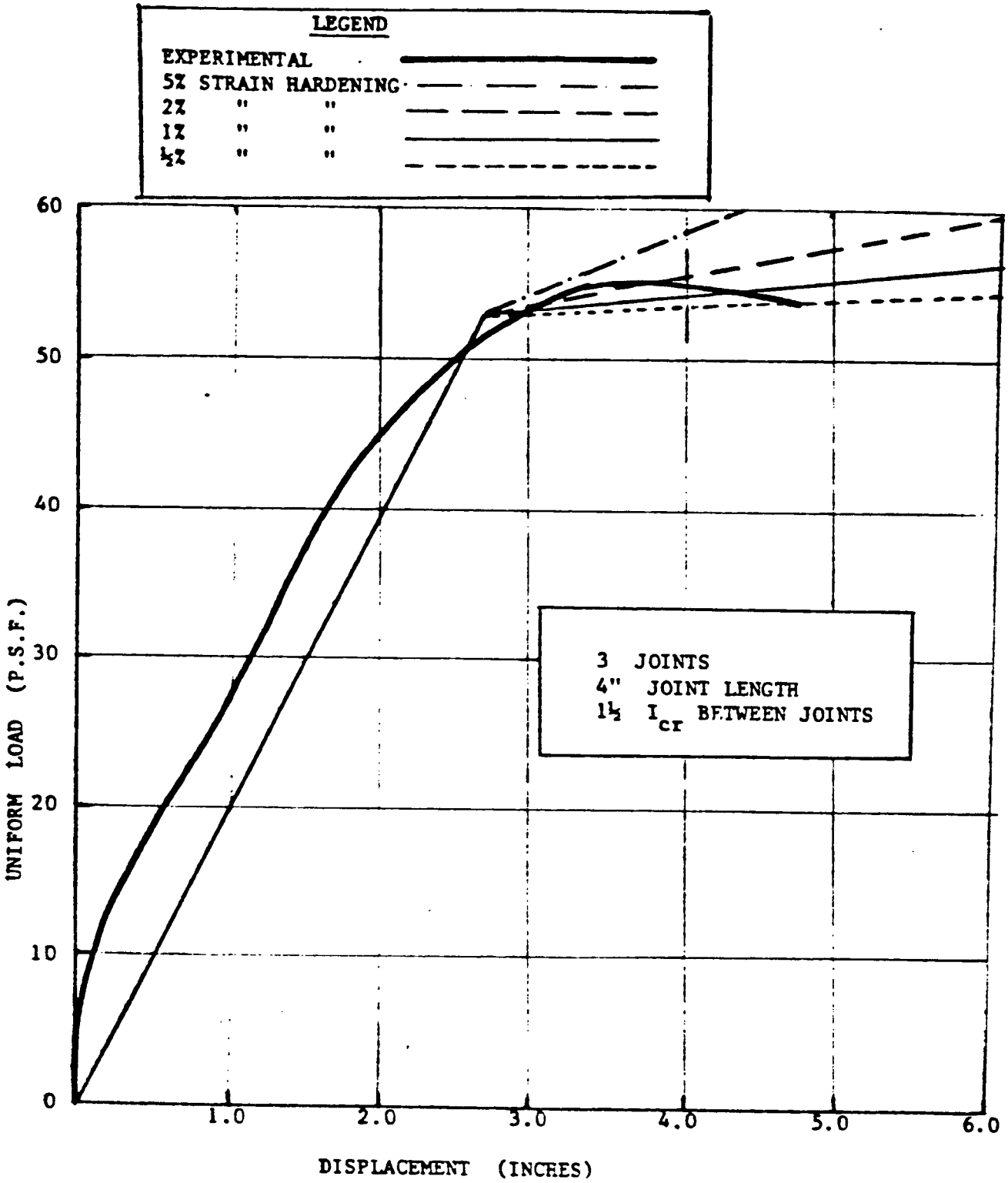

FIGURE 5-7 : YIELDING MODEL --- VARIABLE STRAIN HARDENING RATIO

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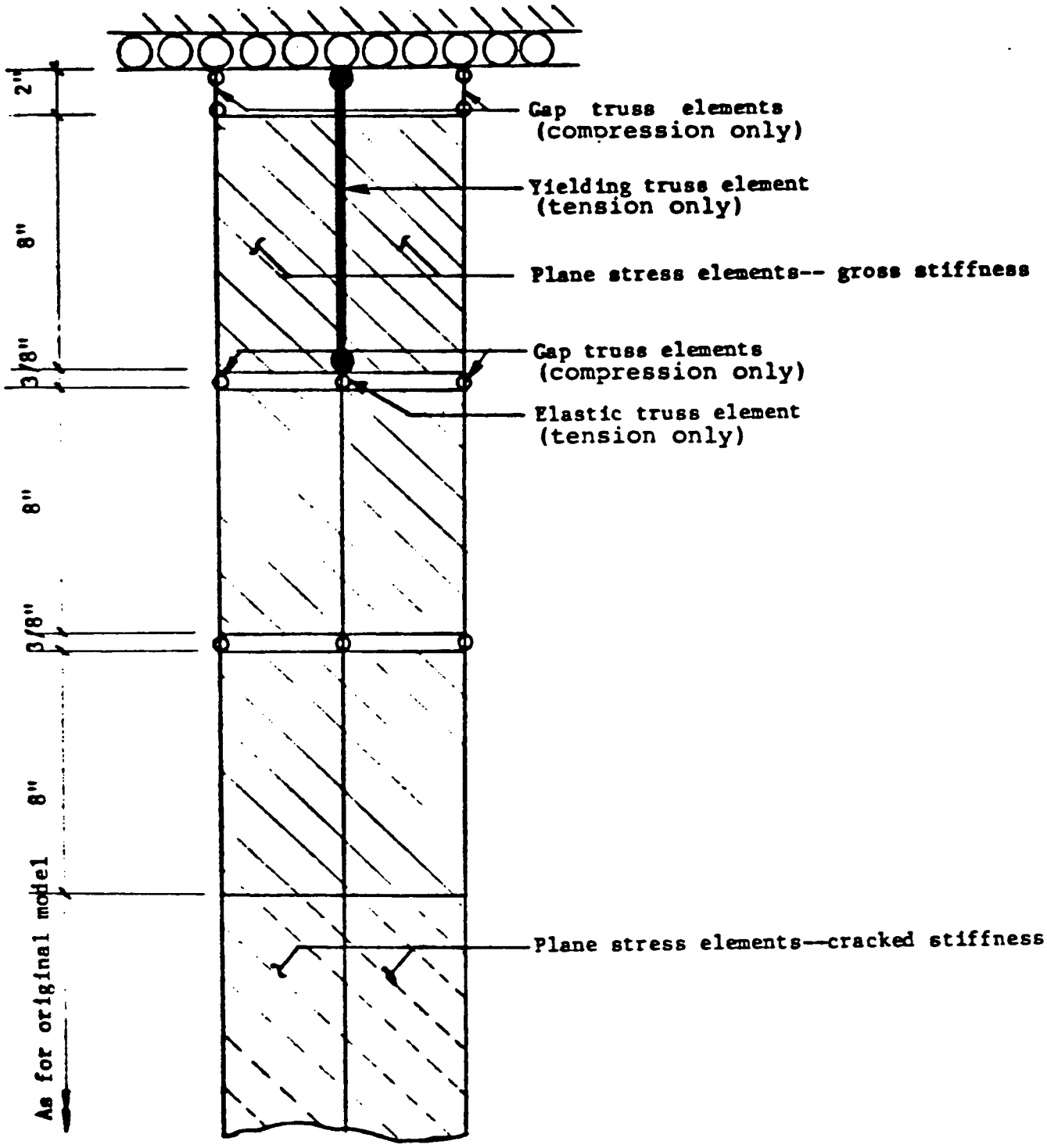

FIGURE 5-8 : JOINT MODEL MODIFIED FOR YIELDING

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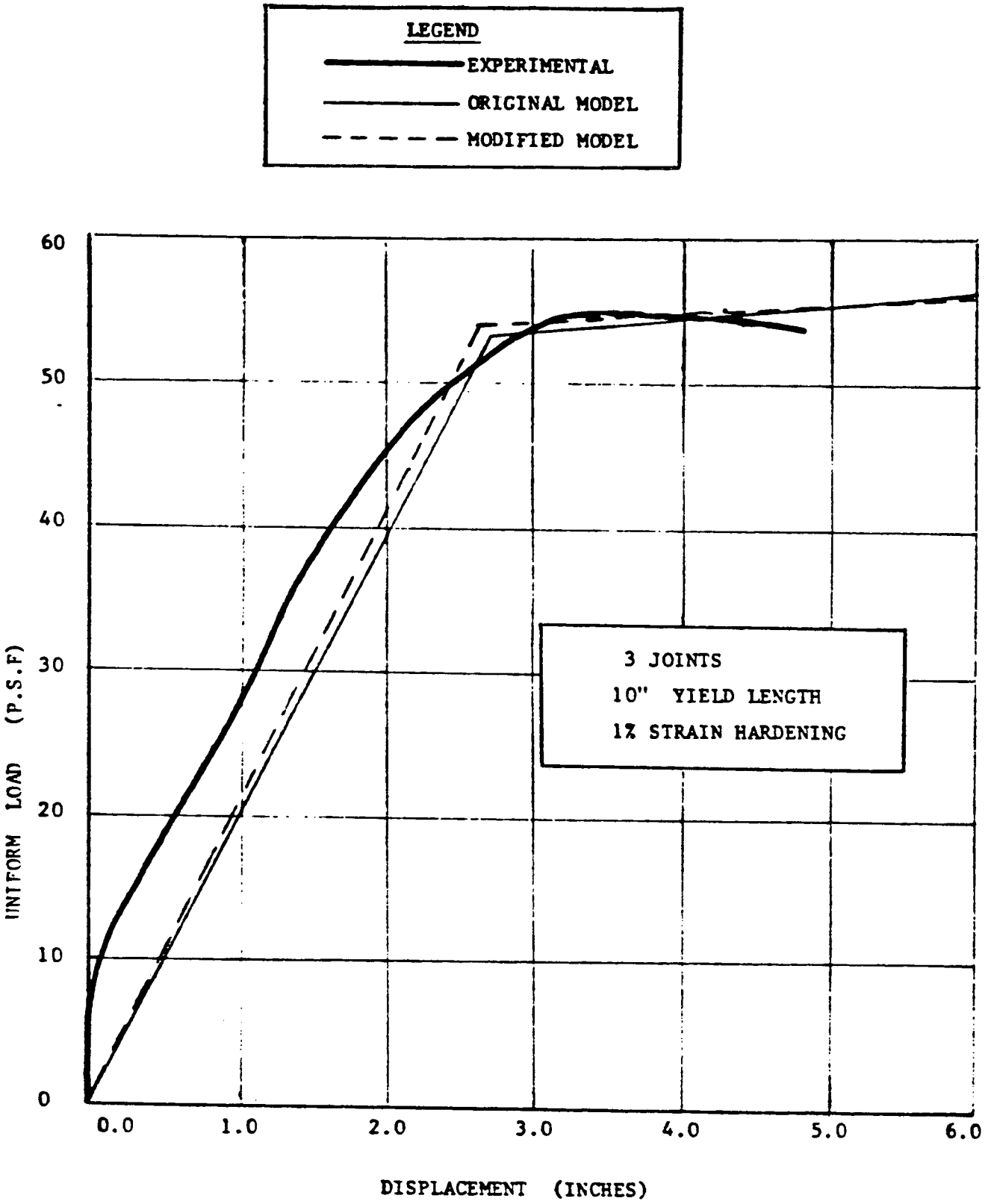

FIGURE 5-9 : ORIGINAL vs MODIFIED MODEL DEFLECTIONS

PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
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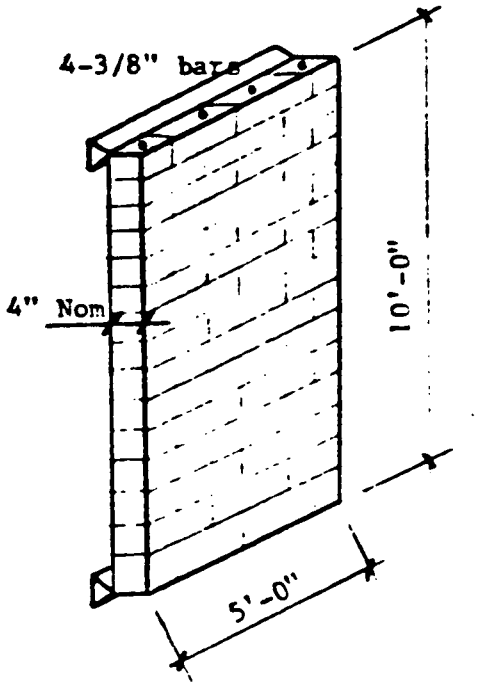
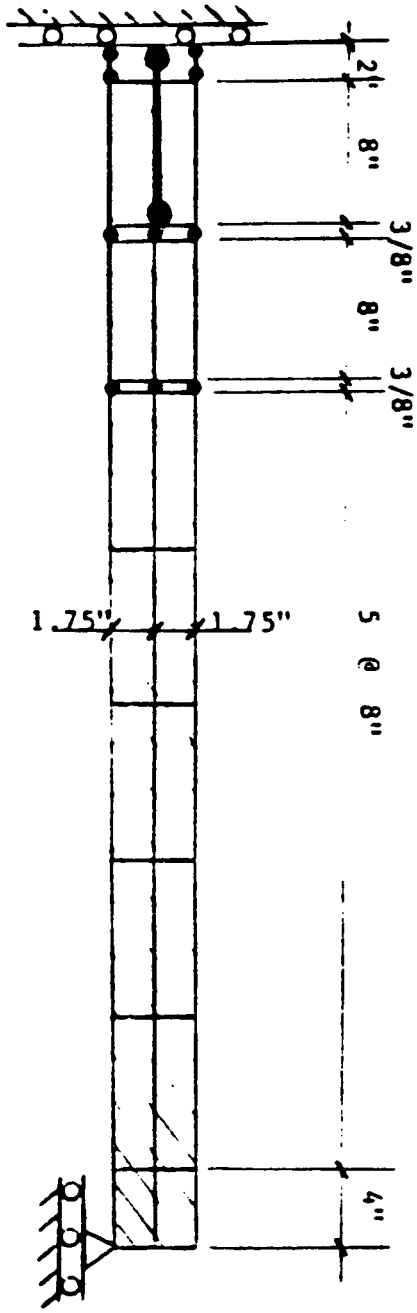


(a) PROTOTYPE
(ref (3))



(b) MODEL 3

FIGURE 5-10 : MODEL FOR CYCLIC LOADING

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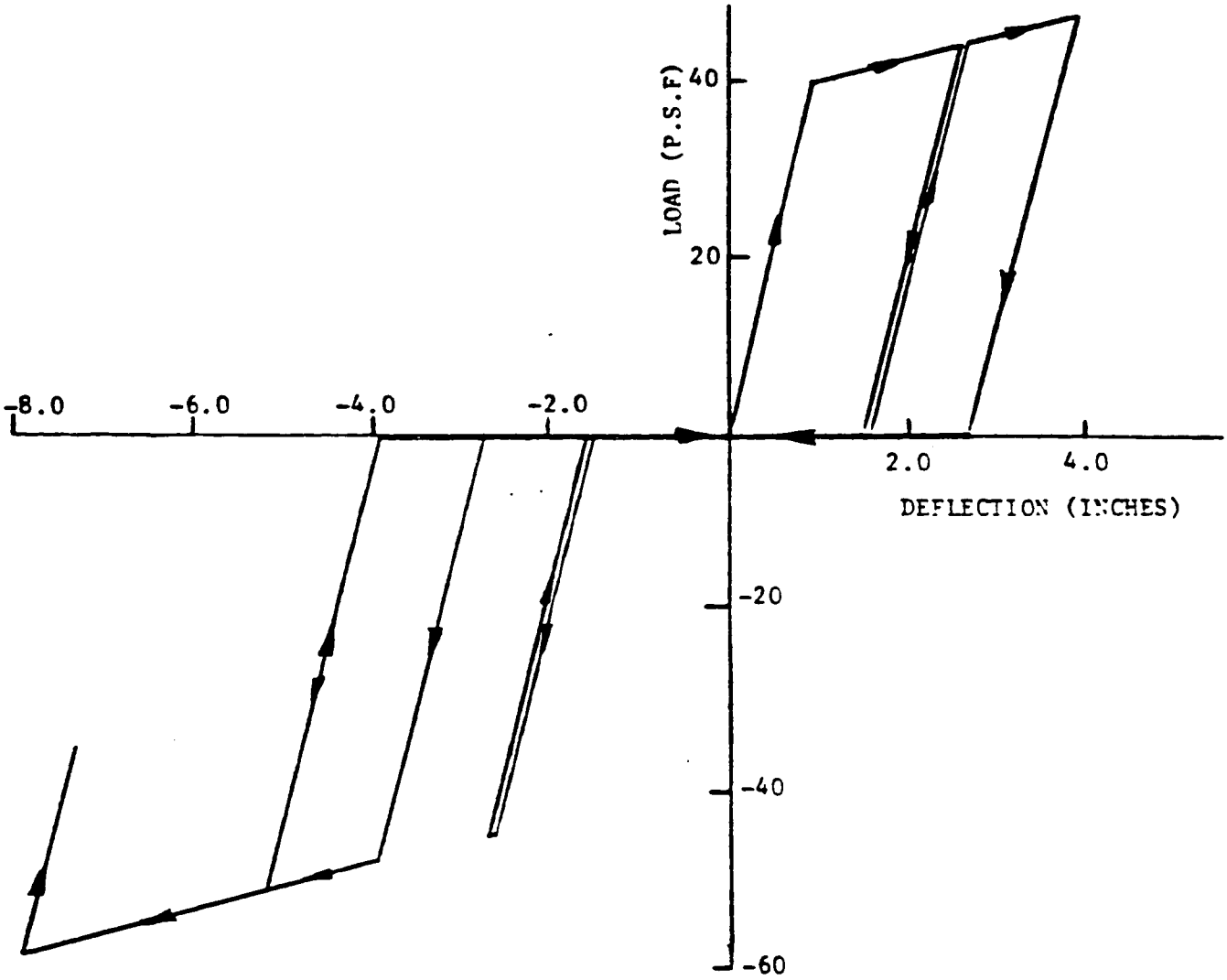
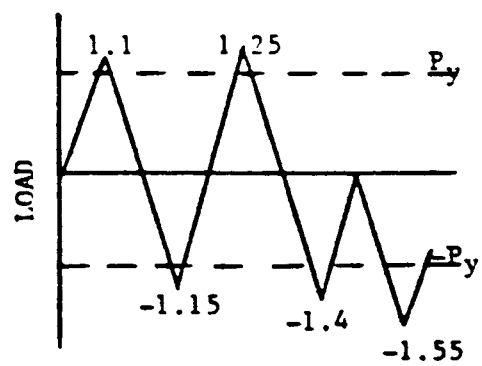


(b) DEFLECTIONS



(a) APPLIED LOAD PATTERN

FIGURE 5-11 : COMPUTED CYCLIC LOAD-DEFLECTION CURVE

PROJECT NO 643	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
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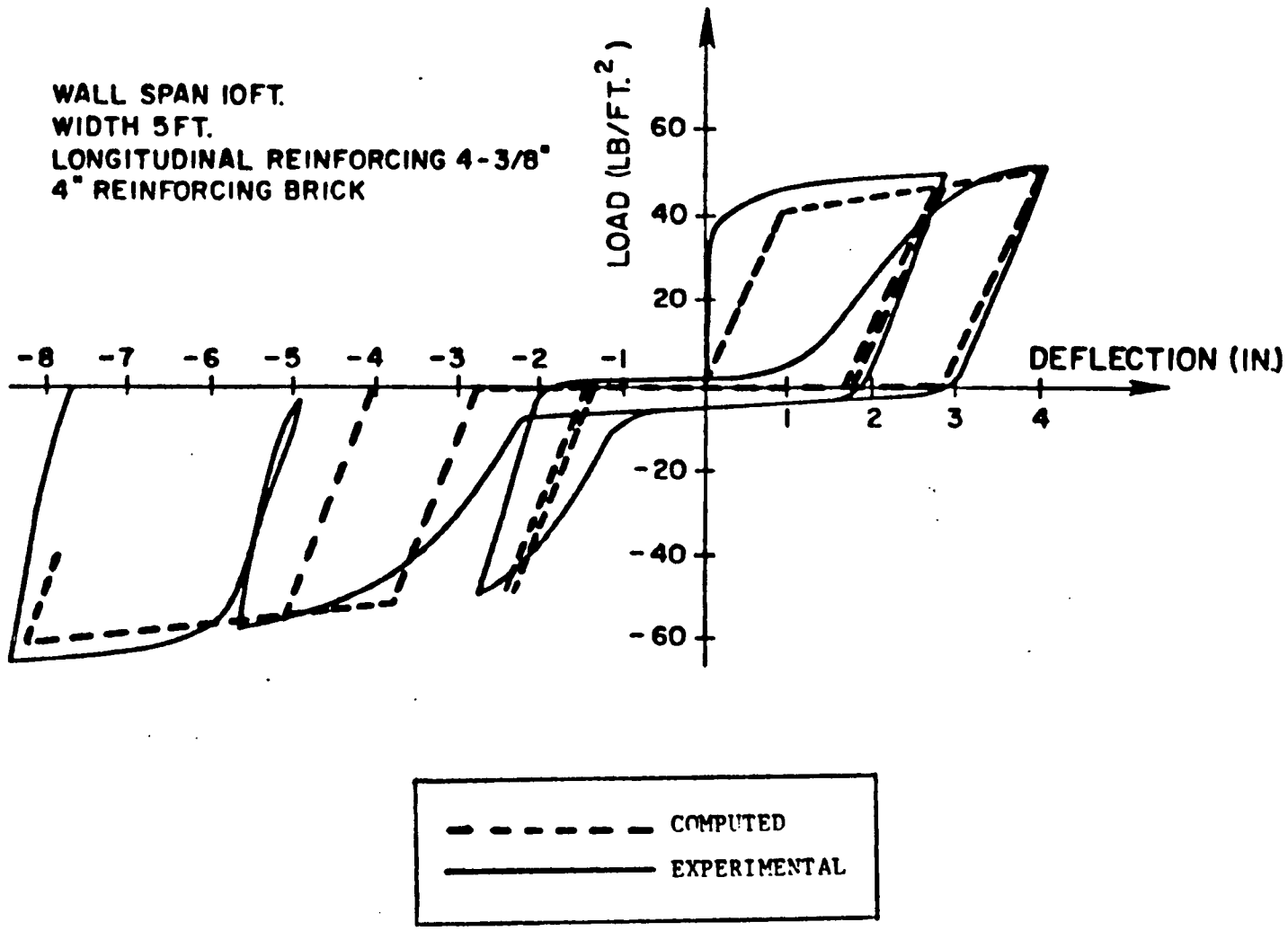

FIGURE 5-12 : COMPARISON WITH EXPERIMENTAL RESULTS

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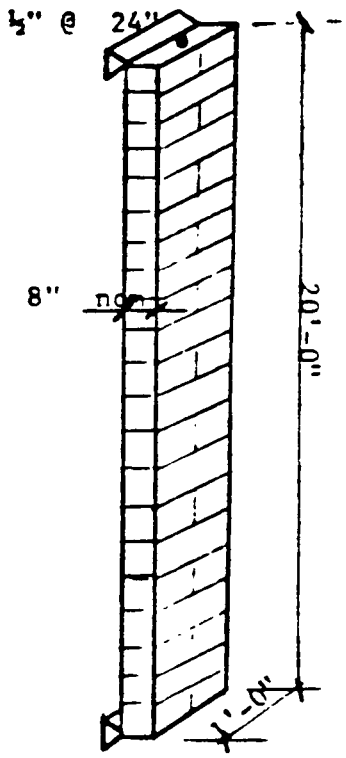
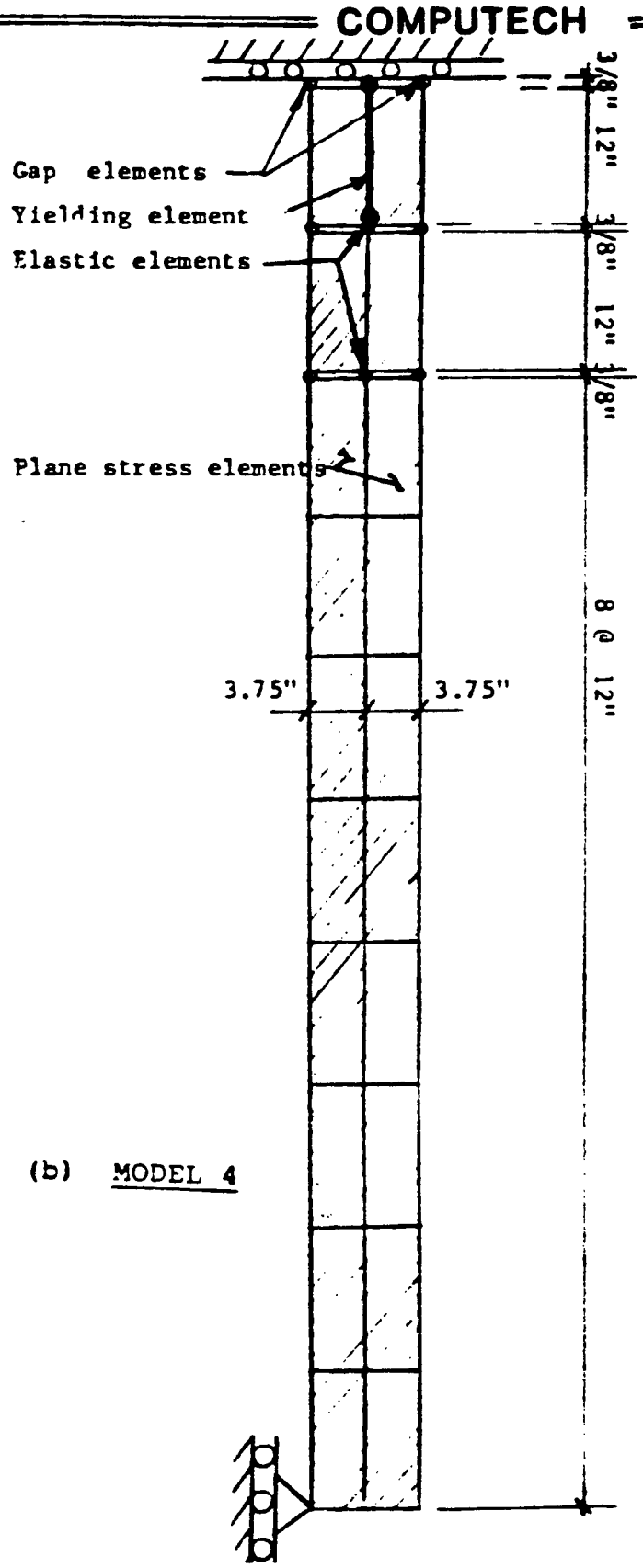
FIGURE NO 5-12



(a) PROTOTYPE



(b) MODEL 4

FIGURE 5-13 : MODEL FOR TIME HISTORY ANALYSIS

PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
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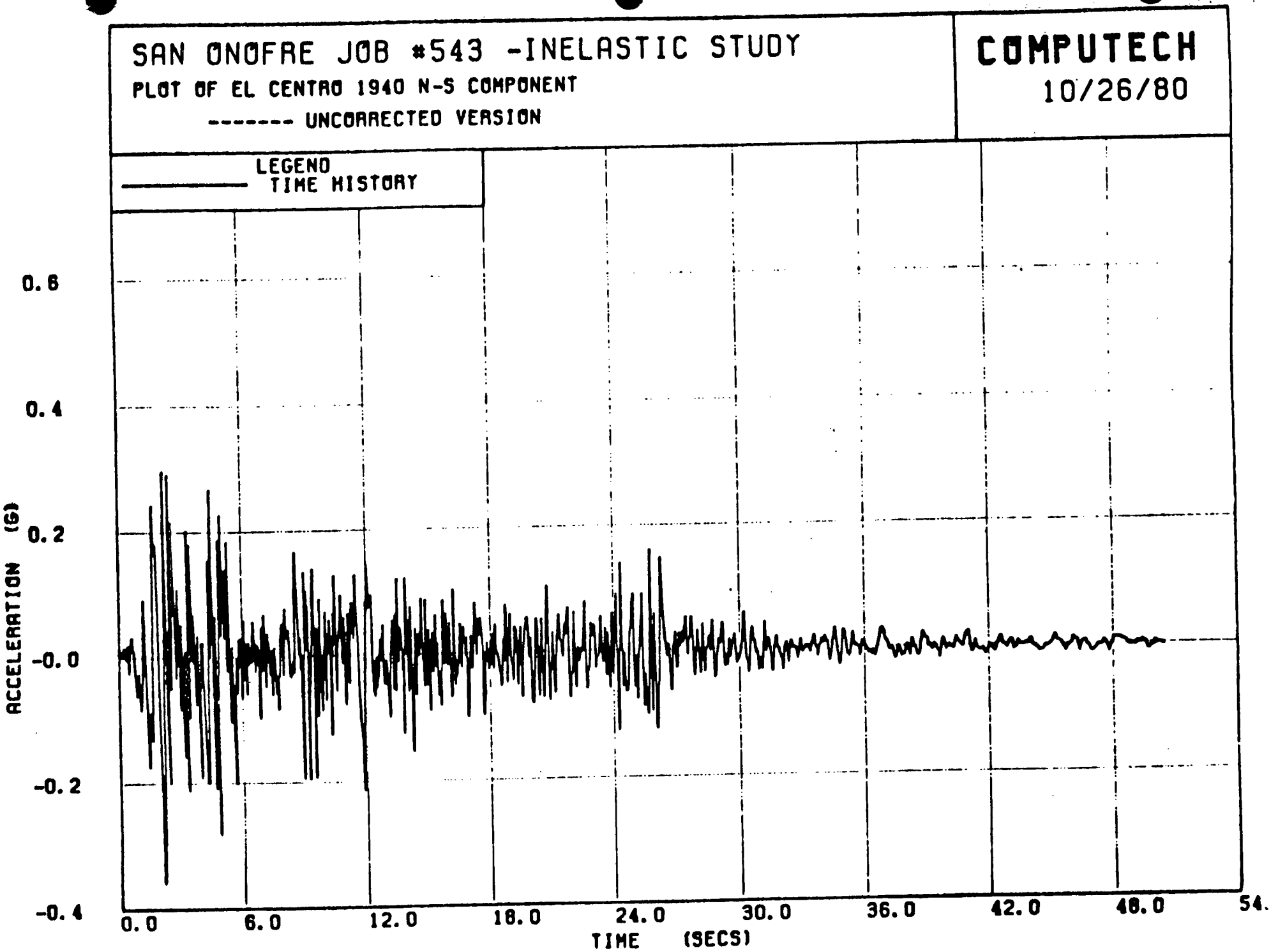

FIGURE 5.14 : EL CENTRO 1940 N-S COMPONENT

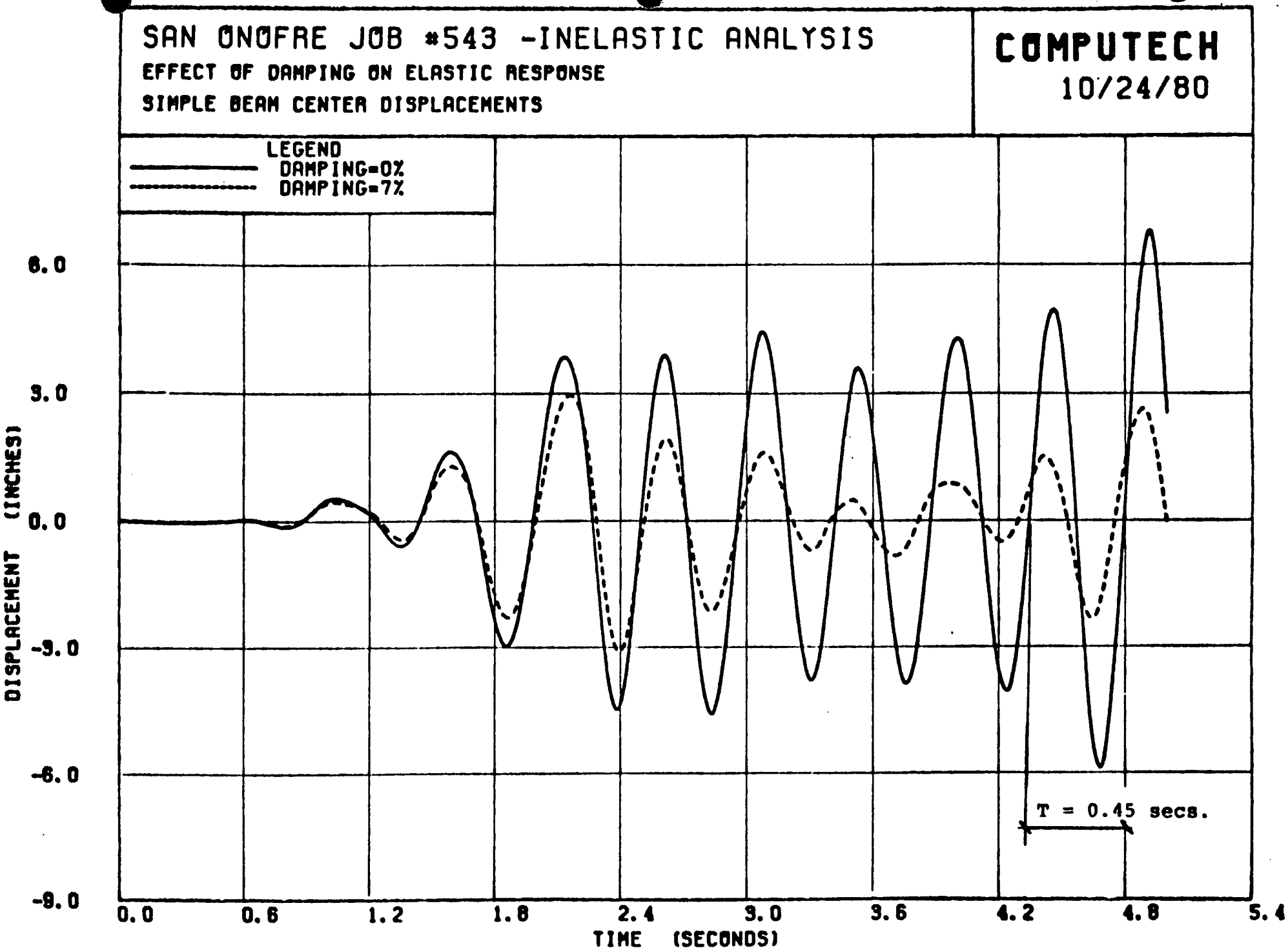

FIGURE 5.15 : RESPONSE TO 1.5 EL CENTRO - ELASTIC MODEL

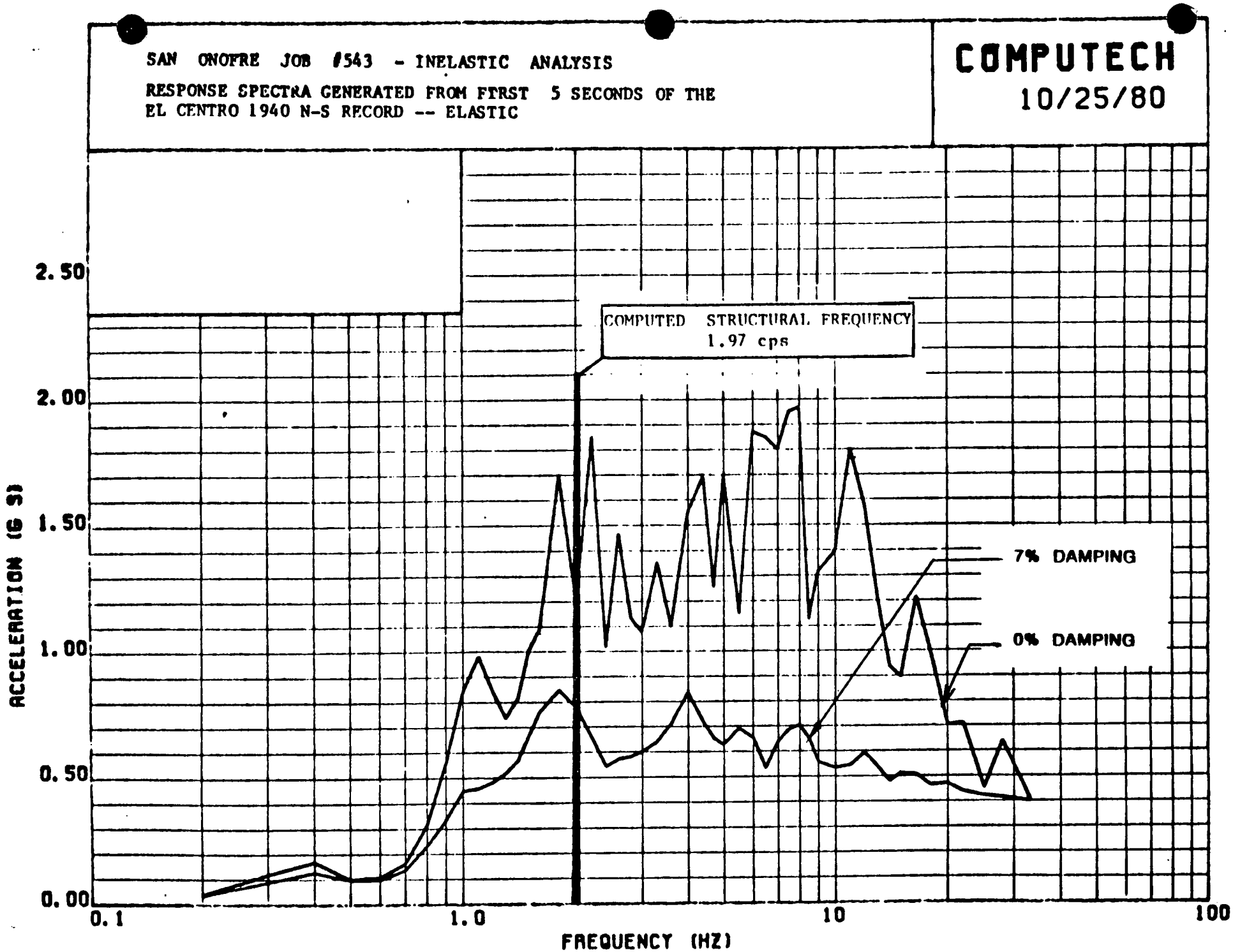

FIGURE 5.16 : RESPONSE SPECTRUM FROM FIRST 5 SECONDS OF EL CENTRO

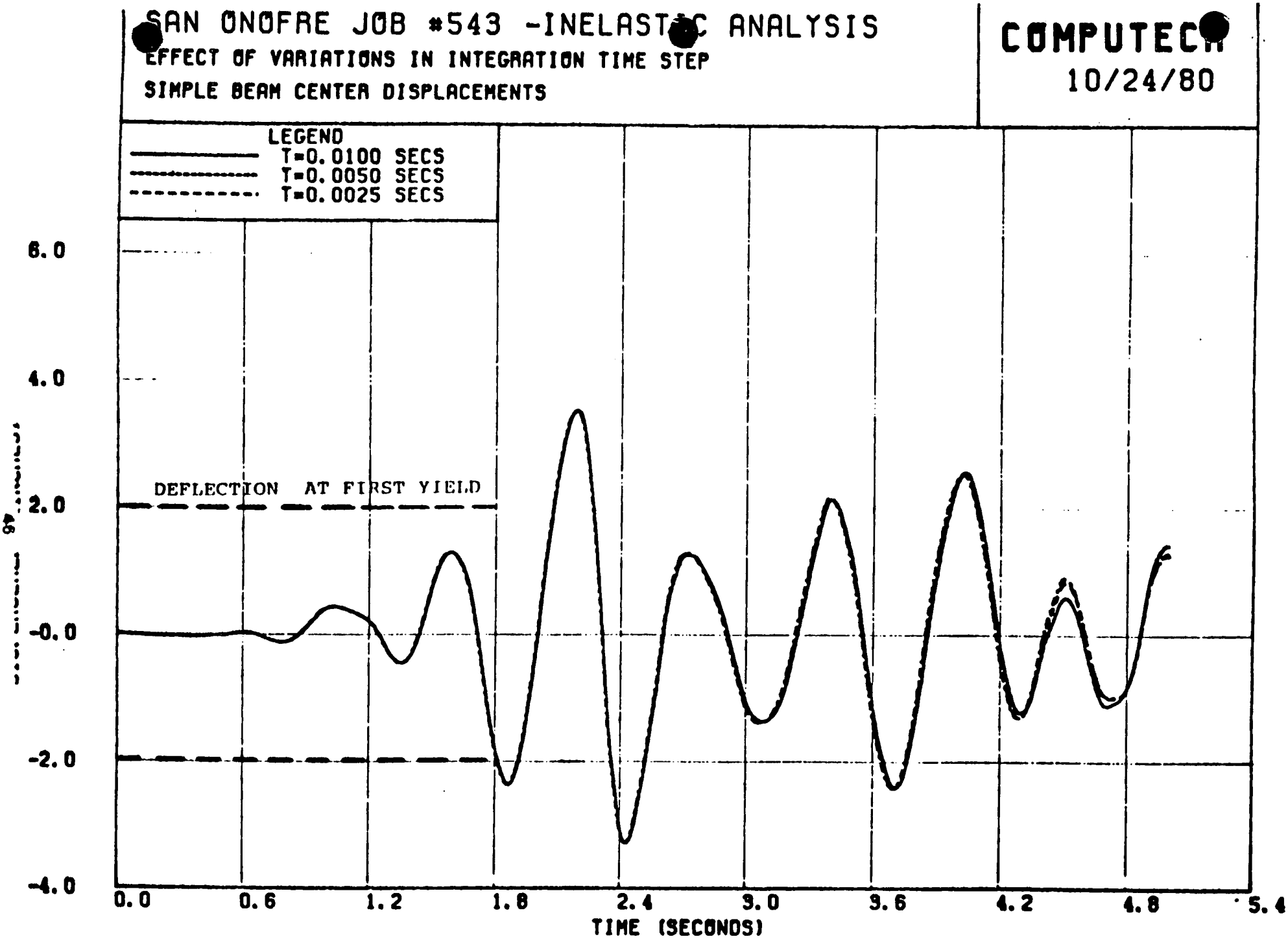

FIGURE 5-17 : RESPONSE TO 1.5 EL CENTRO -- YIELDING MODEL

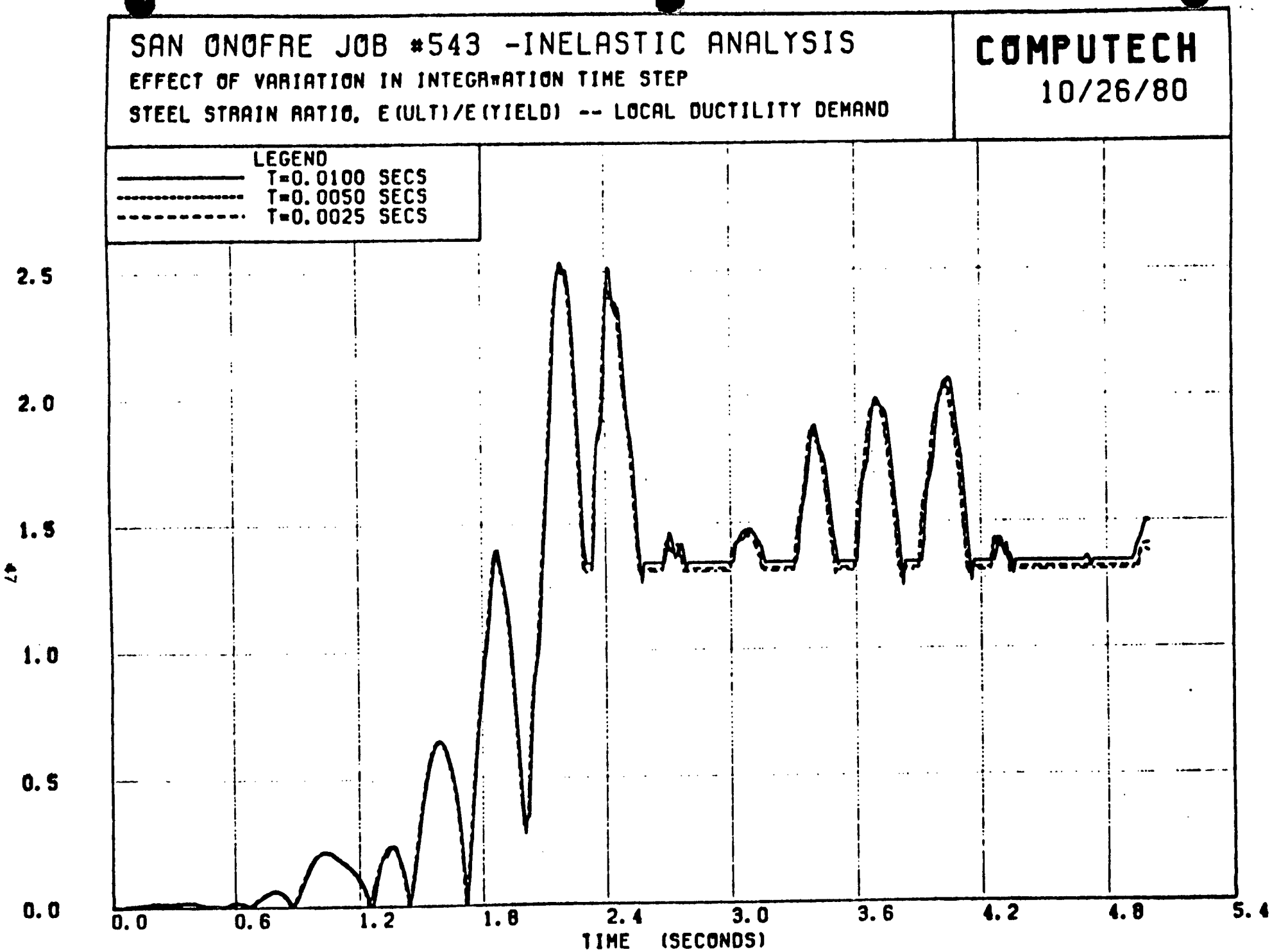

FIGURE 5.18 : RESPONSE TO 1.5 EL CENTRO - YIELDING MODEL

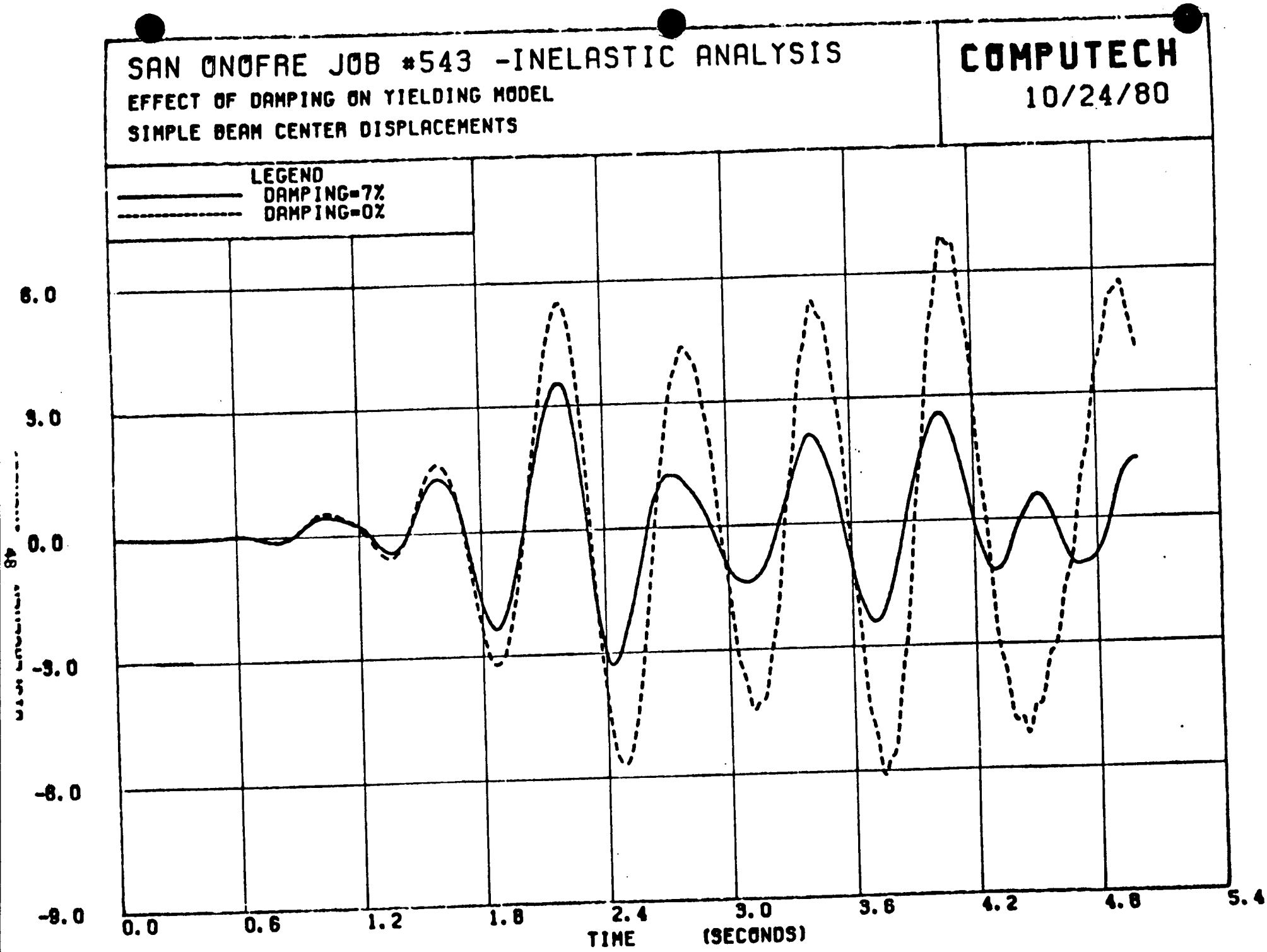

FIGURE 5-19 : RESPONSE TO 1.5 EL CENTRO -- YIELDING MODEL

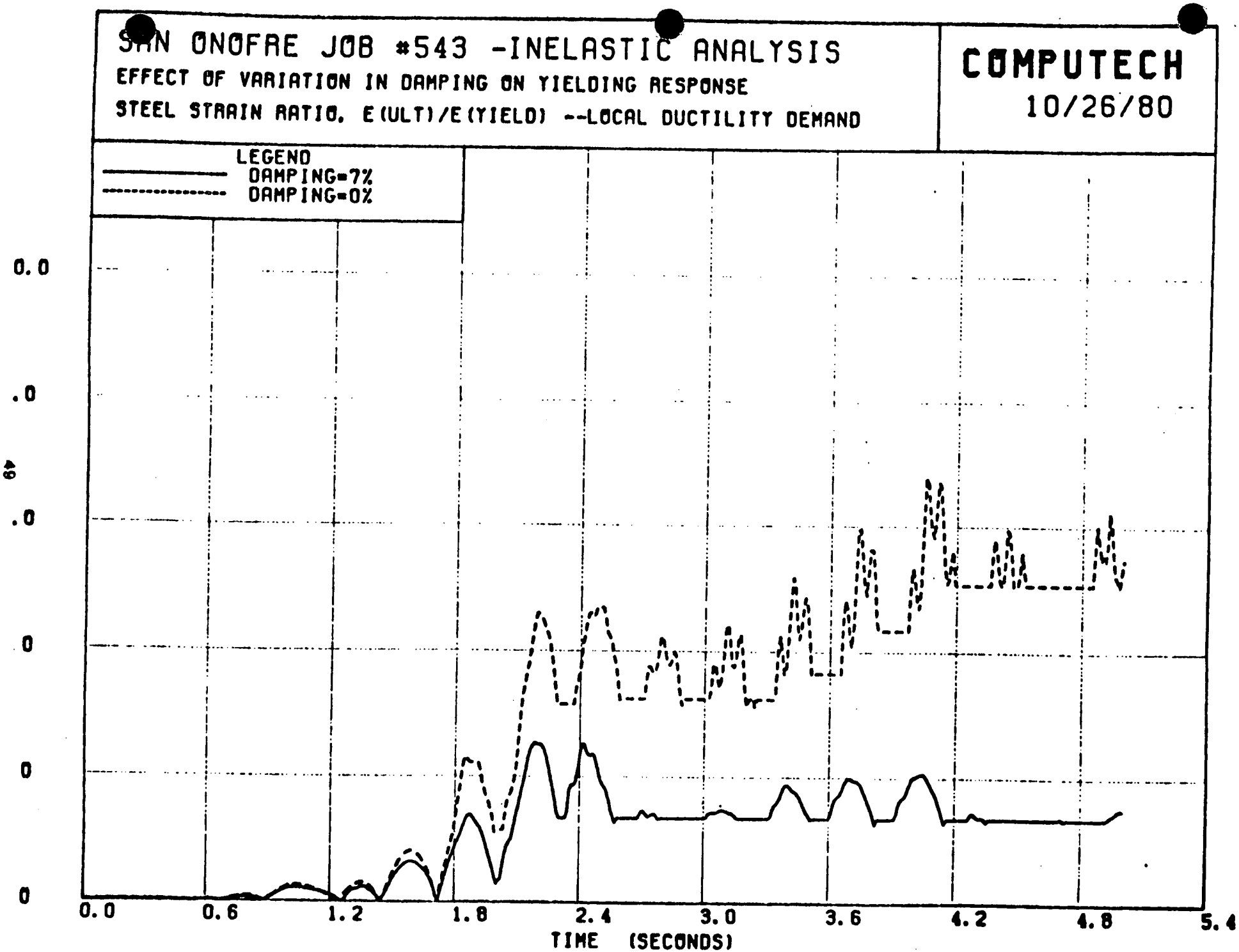

FIGURE 5-20 : RESPONSE TO 1.5 EL CENTRO -- YIELDING MODEL

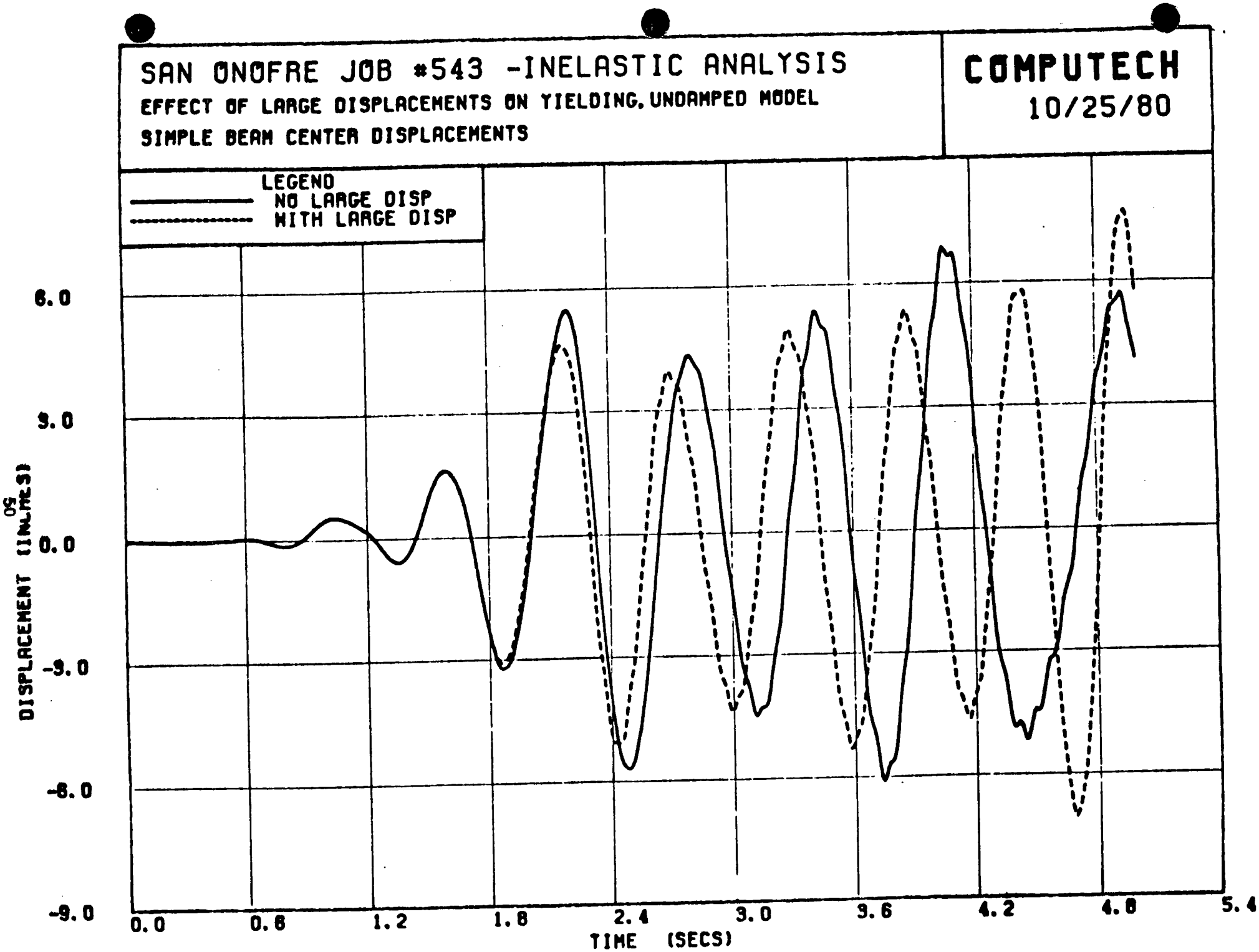

FIGURE 5.21 : RESPONSE TO 1.5 EL CENTRO - YIELDING MODEL

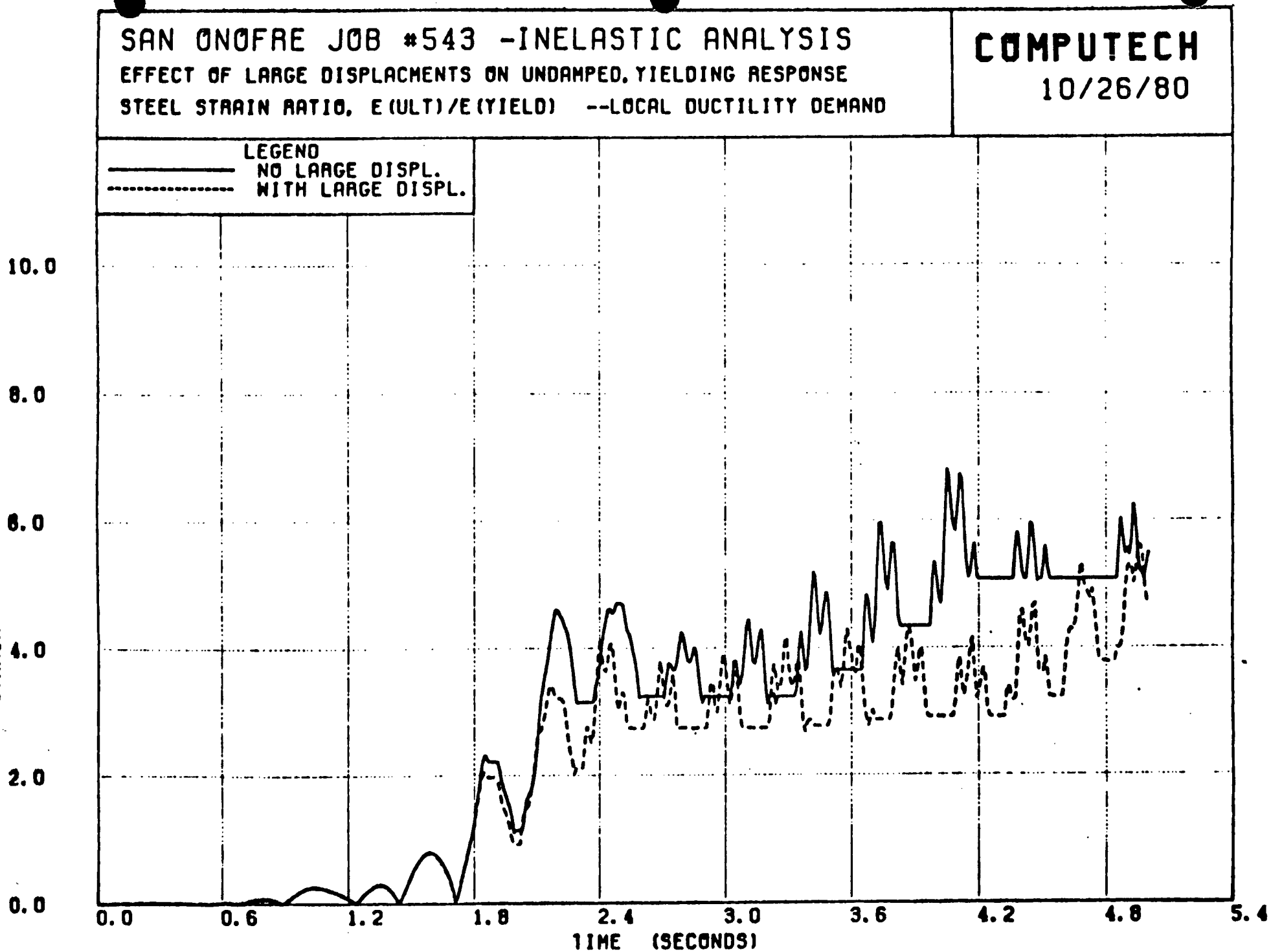

FIGURE 5-22 : RESPONSE TO 1.5 EL CENTRO — YIELDING MODEL

6 COMPUTER PROGRAMS

6.1 Introduction

The studies from which this methodology was derived used two non-linear analysis programs. ANSR-II and DRAIN-2D. Both of these programs were developed at the Univeristy of California, Berkeley, and have been in general use for a number of years.

In the following sections each of these programs is briefly discussed and a comparision is made between the response predicted by each program.

6.2 ANSR-II

ANSR-II is a general purpose computer program for the static and dynamic analysis of non-linear structures including both large displacement and inelastic effects. The original program, ANSR-I, was released for general use in 1975 and was superceded by ANSR-II in 1979.

Three-dimensional structures of arbitrary configuration may be analyzed with the program using any number of static and dynamic loadings. An out-of-core equation solver may be specified for the analysis of large systems and the user has a large degree of control over solution procedures and iteration parameters.

6.3 DRAIN-2D

The original DRAIN-2D program for the dynamic response analysis of inelastic plane structures of arbitrary configuration was released in 1973 and a revised version issued in 1975. In 1980 Schricker (Reference 6) published a report detailing modifications to improve the stability and energy balance for elements with large changes in stiffness

DRAIN-2D is limited to the analysis of two-dimensional systems under the action of seismic loads specified as in-phase ground accelerations acting at all support points. The sole solution method is the Newmark Beta method of numerical integration with unbalanced loads applied at the end of time steps. The Schricker modification applied at the element level allows a time step to be subdivided when a change in yield status occurs and an additional solution step to be inserted. This completely removes unbalanced loads resulting from "overshoot".

6.4 Comparision of ANSR-II and DRAIN-2D

In general ANSR-II is a much more powerful analysis tool than DRAIN-2D in that it includes full three-dimensional effects, large displacement theory, non-linear static analysis and a variety of user-selectable solution schemes.

However these extra computational capabilities are obatined at the expense of a considerable increase in run time and in computer storage requirements.

For the static analyses used to verify the model against experimental results ANSR-II was found superior because of its versatility in specification of loadings and solution techniques. To verify that the two programs produce comparable results a masonry wall time history analysis was repeated using both DRAIN-2D and ANSR-II. The resultant displacements and ductilities are shown in Figures 6-1 and 6-2 respectively. These results show that the response predicted by the two programs is very similar. In general DRAIN-2D predicts a slightly higher response than ANSR-II and is therefore more conservative. for non-linear analysis.

6.5 Program Verification

Both programs are available in the public domain and are supplied by the National Information Service for Earthquake Engineering (NISEE). The versions used by Computech are fully verified in accordance with the company's quality assurance program. Benchmark programs supplied by NISEE have been checked on the programs as implemented and the resultant output checked against the supplied results.

6.6 Summary and Concluding Remarks

The initial feasibility studies and the parametric studies have shown that both ANSR-II and DRAIN-2D are capable of analyzing masonry walls for input support acceleration ground histories. For static analyses ANSR-II has greater versatility and is preferred over DRAIN-2D.

It is essential that the version of DRAIN-2D used have the modification allowing repetition of a time step within which a change of yield status in the yielding rebar element occurs. This prevents excessive unbalanced force vectors and consequent numerical instability.

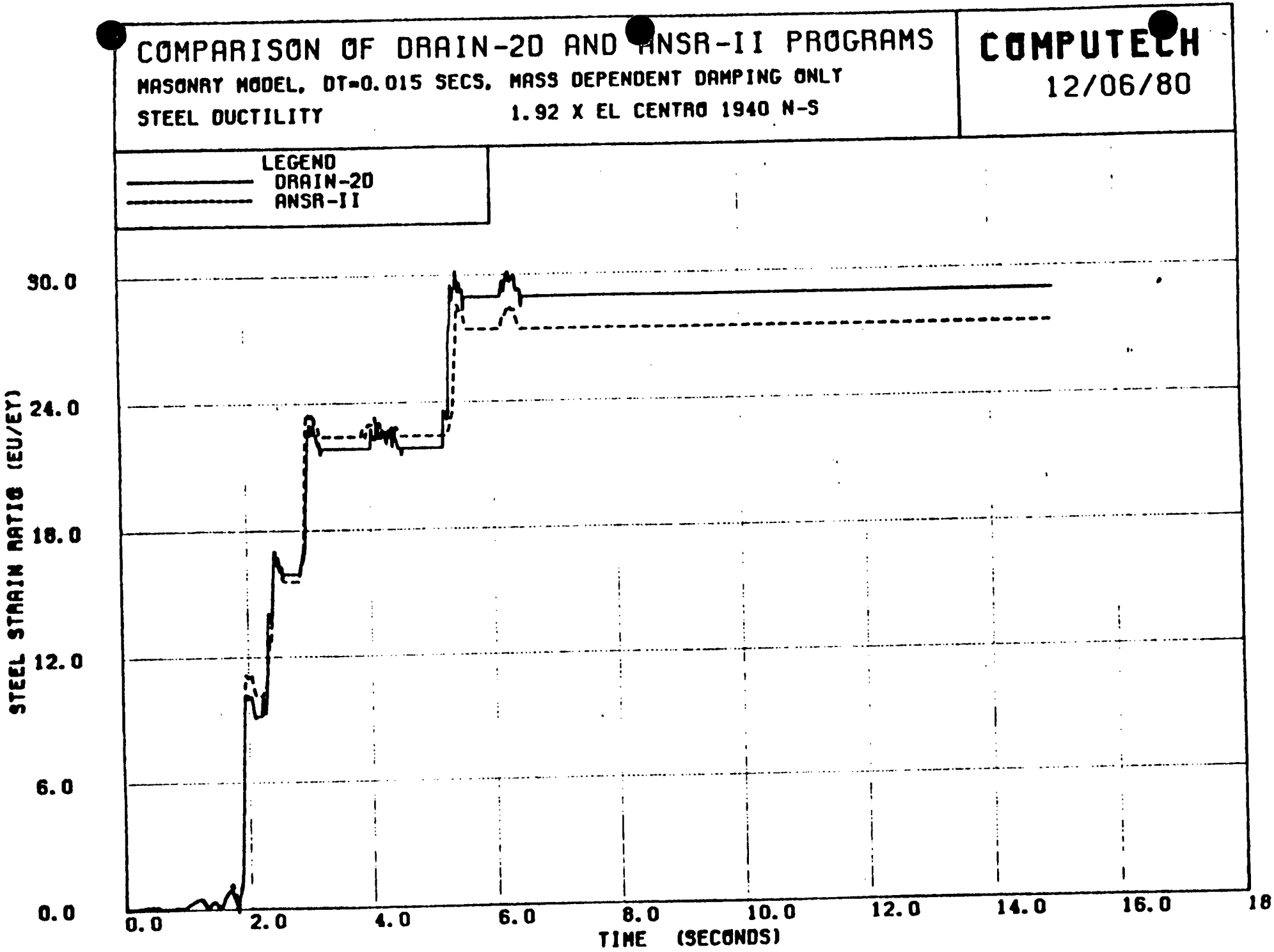

FIGURE 6-1 : COMPARISON OF DRAIN-2D AND ANSR-II -- DISPLACEMENTS

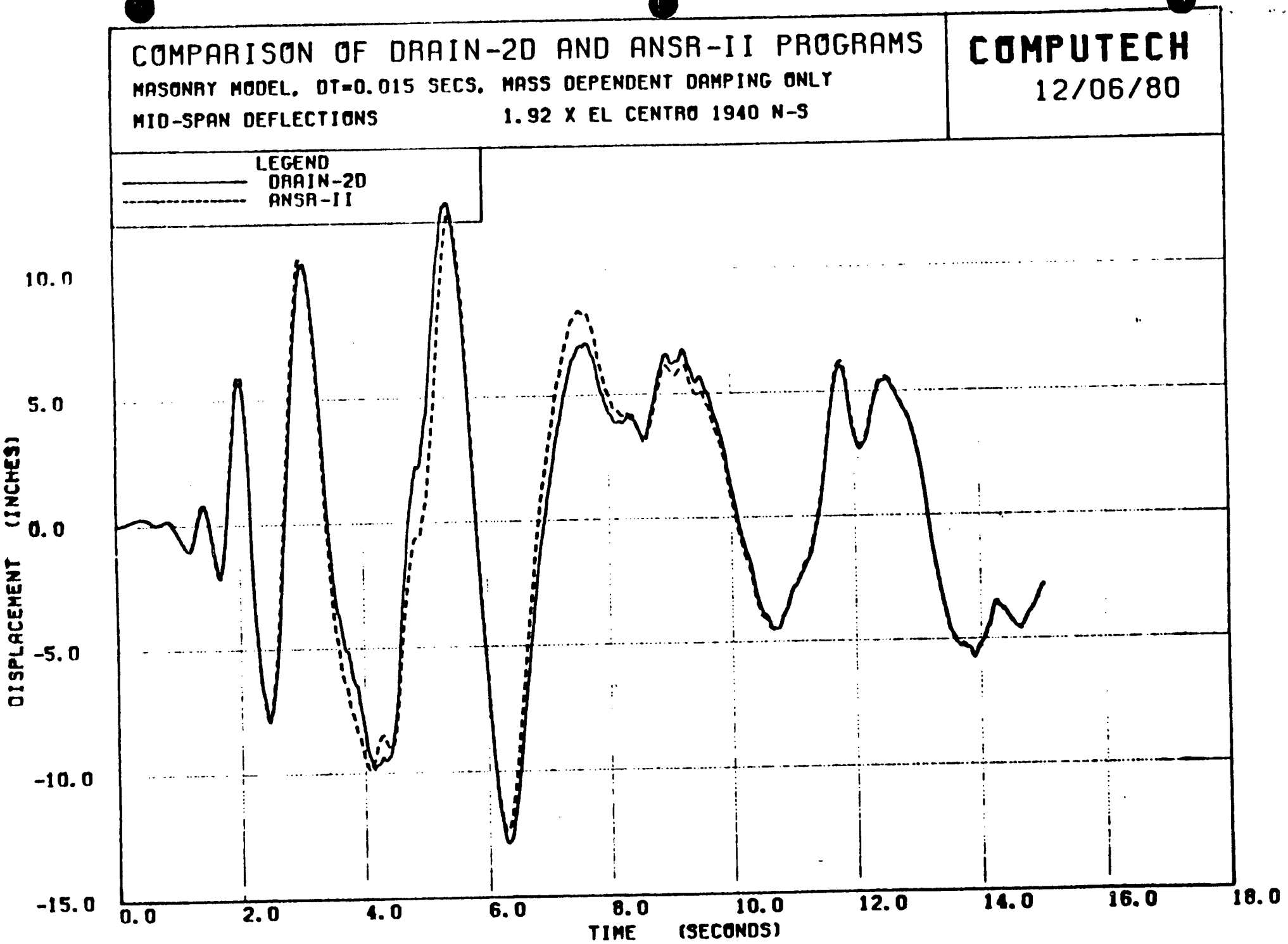

FIGURE 6-2 : COMPARISON OF DRAIN-2D AND ANSR-II -- DUCTILITY

7 MODELLING PROCEDURES

From the initial feasibility study and the parameter studies the following procedures were developed for the non-linear modelling of the masonry walls at the San Onofre, Unit 1 plant:

- Model a minimum of two joints on either side of the points of maximum moment.

- At joints model the face shells as truss bars elastic in compression with zero tension capacity. Model the central rebar as an elastic truss member.

- Model the plastic hinge as a truss bar which yields in tension.

Blocks between joints should be modelled as plane stress elements with properties based on the gross uncracked masonry properties.

- Outside the joint region model the wall with plane stress elements of stiffness equivalent to 1.5 times the transformed moment of inertia.

- Strain hardening should be typically 3 to 5% of the stiffness prior to yield, which will typically be provided by using a strain hardening ratio of 0.01 in the yielding member.

- Damping and large displacement formulation if required should be applied at the element level to the plane stress elements only.

- A time step less than or equal to 0.02 times the fundamental period should be used unless a larger value is found satisfactory on a wall-by-wall basis.

- The time step repetition option in DRAIN-2D must be used for the yielding element to avoid instability.

- If ANSR-II is used for the time history analyses iteration within a time step should be used for unbalancec loads greater than 1000 lbs.

8 CONCLUSIONS

The development of a methodology for the nonlinear analysis of masonry walls due to out-of-plane loads has been presented. The methods are based on the use of the existing computer programs DRAIN-2D and ANSR-II. Model parameters were derived using both theoretical considerations and the results from a series of monotonic tests. The resultant model was verified by comparing analysis results with those obtained from a series of cyclic loading tests. Good correlation was obtained.

A series of time history analyses were performed to investigate the influence of various parameters on the solutions obtained. These analyses indicated conditions under which the solution remained stable. A methodology was established for use in constructing models of the actual walls at San Onofre, Unit 1.

This stage of the study was to develop a methodology, not to draw conclusions on the actual performance of the San Onofre, Unit 1 walls. The studies did indicate that centrally reinforced masonry walls exhibit significant ductile behavior and retain their structural integrity well beyond the initial yield point.

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SAN ONOFRE NUCLEAR GENERATING STATION
UNIT 1

SEISMIC EVALUATION OF REINFORCED CONCRETE MASONRY WALLS

VOLUME 3 : MASONRY WALL EVALUATION

Prepared for:

BECHTEL POWER CORPORATION
Los Angeles, California

Prepared by:

COMPUTECH ENGINEERING SERVICES, INC.
Berkeley, California

January, 1982

REPORT NO. R543.02

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TABLE OF CONTENTS

1 INTRODUCTION

Volume 1 of this report details the criteria to be used for the evaluation of the SONGS-1 masonry walls, and Volume 2 describes the methodology developed for the inelastic out-of-plane analysis of the walls. This volume reports the results of the evaluation of the walls in three buildings of the plant using the criteria and methodology presented in Volumes 1 and 2.

The evaluation of walls presented in this volume relates to those walls for which the specified ground accelerations cause inelastic actions when subjected to out of plane forces. Walls that remain elastic when subject to out of plane forces and also the evaluation of the combined in-plane and transverse loads are covered in separate evaluations. In addition, the walls in the Fuel Storage Building are covered in a separate evaluation

The scope of the evaluation is summarized in the following part of this section. Section 2 of this volume details the methodology used for the wall analyses, time history selection and wall evaluation including the effects of openings and added mass. The evaluation is based throughout on the methodology contained in Volume 2 and the criteria contained in Volume 1.

Sections 3 through 5 report the evaluation of individual walls on a building by building basis. For each building the masonry walls are described and the inelastic analysis results are tabulated. Based on these results the walls are evaluated in terms of the criteria. Conclusions are given in Section 6.

1.1 Scope of Evaluation

The evaluation presented herein relates mainly to walls responding inelastically in the transverse direction. Such walls are present in the following three buildings:

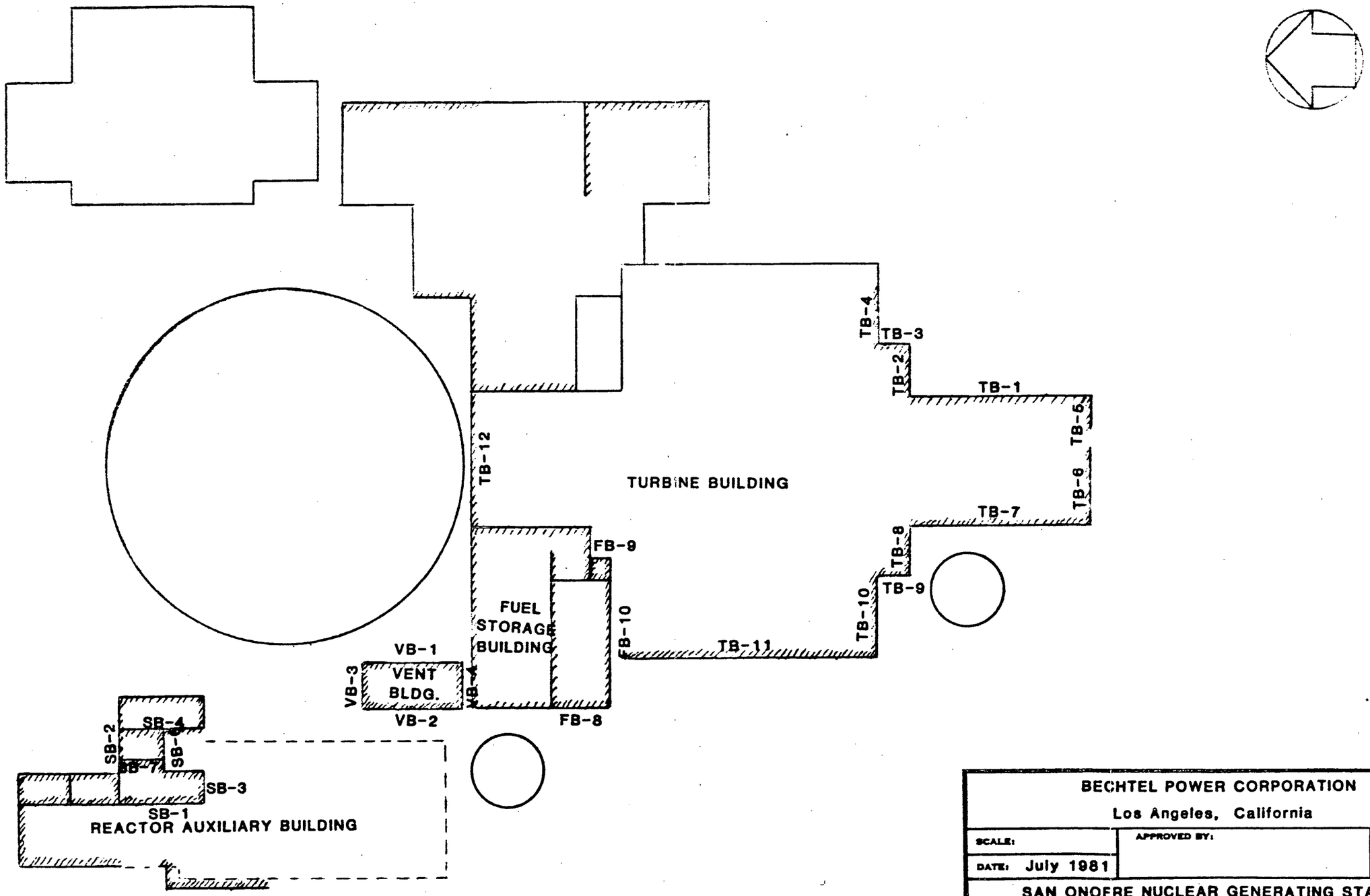
- Turbine Building (12 walls)
- Ventilation Equipment Building (4 walls)
- Reactor Auxiliary Building (6 walls)

The walls in each building that are included in this evaluation are identified and located in Figure 1-1 Details of walls, reinforcing details and equipment masses were obtained from a series of field prepared sketches and from the original structural drawings for the plant.

The wall numbers include all walls in the Turbine Building and the Ventilation Building For the Reactor Auxiliary Building they include all safety related walls for which preliminary analyses indicated inelastic action - the remaining 9 walls respond elastically to the specified load levels or were not safety related

For the walls listed above a full evaluation of the transverse response was performed, including the effects of added equipment mass and openings in the walls The effect of the transverse response on the walls capacity to sustain concurrent vertical and in-plane loading was assessed. The final

evaluation of the combined in-plane and out-of-plane loading response is to be made by BPC in conjunction with the overall structural analyses of these buildings. In addition, reaction loads are provided in this report for use by BPC in the check of connection capacities.



BECHTEL POWER CORPORATION Los Angeles, California		
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DATE: July 1981		REVISED
SAN ONOFRE NUCLEAR GENERATING STATION, UNIT 1 MASONRY WALL IDENTIFICATION		
Computech Engineering Services, Inc. Berkeley, California		DRAWING NUMBER FIGURE 1-1

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2 METHODOLOGY

2.1 Method of Analysis

Volume 2 of this report details the methodology developed for the inelastic transverse analysis of centrally reinforced masonry walls. This methodology is based on the use of the non-linear computer programs DRAIN-2D and ANSR-II. The walls are modelled with plane stress elements away from the hinging region and by a combination of yielding bar elements and compression only gap elements at the masonry joints adjacent to the hinging region.

The procedures contained in Volume 2 were used and models were developed for each wall type of each building. These were then analyzed for three different time histories of ground acceleration. The selection of these time histories is discussed in the following section. From the results of these time history analyses, wall displacements, steel ductility and an evaluation of wall stability was obtained. These were then evaluated in terms of the criteria of Volume 1 of this report to assess the wall adequacy when subjected to out of plane loadings.

2.2 Time History Selection

The Design Basis Earthquake motion used to evaluate the San Onofre, Unit 1 masonry walls was the Housner Response Spectrum normalized to 0.67g. As the methodology for the transverse analysis required time history analyses, earthquake acceleration records compatible with this design spectrum were required. The Housner Spectra is a composite smoothed spectra derived from the two horizontal components of the El Centro 1934, El Centro 1940, Olympia 1949 and Taft 1952 earthquakes. Thus suitable time histories should be selected from this list Bechtel Power Corporation, Los Angeles carried out the selection based on the following steps:

1. Each horizontal component for the four earthquake events was scaled to have an equal spectrum intensity to the Housner spectra. This was achieved by computing a scale factor such that the area under the zero damped velocity spectrum curve for the particular component was equal to the equivalent area under the Housner velocity spectrum The integration was carried out over the range of periods from 0.1 to 2.5 seconds, where the bulk of the earthquake energy input is concentratec

2. A single peak of the scaled time history was raised to 0.67g so as to obtain the same zero period acceleration as the normalized Housner Response Spectrum

3. The time histories so obtained were used to produce the 7% damped acceleration response spectra.

4. These 7% damped spectra were then compared with the Housner spectrum for the equivalent damping.

5. The three earthquake components which collectively enveloped the Housner curve over the complete frequency range were selected for the inelastic analyses. These were:

El Centro	May 18,1940	S00E Component
Taft	July 21,1952	S69E Component
Olympia	April 13,1949	N40W Component

In Figure 2.1 the 7% damped Housner Spectrum is plotted together with the three earthquake components listed above. It can be seen that the three components collectively envelope the Housner spectrum throughout the frequency range of interest.

The acceleration time histories used were obtained from the records digitized, filtered and corrected by the Earthquake Engineering Research Laboratory at the California Institute of Technology (EERL). The three components are listed below together with the EERL designation assigned to the records and the actual scaling factor applied to the acceleration time histories.

EARTHQUAKE		COMPONENT	EERL DESIGNATION	SCALING FACTOR
EL CENTRO	May 18 1940	S00E	A001/S00E	1.57
TAFT	July 21,1952	S69E	A004/S69E	2.90
OLYMPIA	April 13,1949	N40W	B029/N40W	2.51

It should be noted that the El Centro May 18 1940 N00E time history by itself almost completely envelopes the Housner 7% damped response spectrum and that the envelope of the three time histories exceeds the Housner Response Spectrum by a substantial margin in the amplified region of the spectrum (in some cases by nearly 100%). On this basis it was concluded that the selected time history inputs conservatively respresent the 0.67g Housner Response Spectrum

2.3 Added Mass

For the analytical models the mass was lumped at the nodal points so that

the correct distributed mass for the grouted masonry walls was obtained. Most walls have additional mass due to conduit, cable trays, junction boxes etc. For each wall the length with the greatest amount of added mass was selected for the analysis. The total weight of equipment at this critical point was then computed from unit weights. The equipment weight was based on conservative assumptions, e.g. that the conduits were all full and that all cable tray weights were equal to the heaviest. These weights were converted to mass units and added to the closest equivalent nodal point in the analytical model.

This method of applying added weights essentially assumes that both the equipment and its support is fully rigid and thus is subjected to the same level of acceleration as the wall to which it is attached. This is considered to be appropriate especially for the heavier items such as junction boxes, large diameter conduit and cable trays.

For many walls, particularly in the Turbine Building, the mass due to the equipment formed a substantial portion of the total mass and in some cases exceeded the weight of the wall itself. These masses are generally localized in extent and so the analysis results using them are upper bound values. The analysis for some walls was carried out both with and without the equipment mass and in these instances the effect of the added mass on the wall response is given in the summary of results.

2.4 Wall Openings

For relatively small openings the effect of the openings was considered not to adversely effect the wall response provided the following two criteria were met:

1. The area of additional reinforcement provided by the trimmer bars around the openings is at least equal to the amount of distributed vertical steel terminated because of the presence of the opening

2. The total width of opening across any plane in a wall is such that the remaining area of masonry face shell is sufficient to resist the full steel force without exceeding the allowable masonry stress of 0.85 f'm.

The effect of all openings within the length of any one wall was evaluated using the two criteria above. The length of a wall was defined as the distance between vertical control joints. This was generally between 32'-0" and 40'-0".

For the first condition listed above it was assumed that a minimum of one bar was terminated by the opening, but generally this did not prove a severe constraint as trimmer bars were generally specified as 2-#5 bars at each side of the opening and even a 4'-0" wide opening implied a loss of a maximum of two bars at the typical spacing of 32".

The second constraint, that of the maximum masonry stress, was generally checked by expressing the total width of openings across a plane as a percentage of the total wall width. For checking the masonry stress an ultimate strength approach was used, consistent with the levels of load in the inelastic analysis. The width of compressive face shell in a 12" strip was reduced in proportion to the reduction in wall width due to openings. The depth to the neutral axis was then computed based on the maximum rebar force from the time history analysis, the reduced section width and a uniform masonry stress of 0.85f'm across a rectangular stress block. If the neutral axis position computed based on these assumptions was within the thickness of the masonry face shell the wall was considered satisfactory.

For a number of walls the size and/or location of multiple openings was such that the simplified procedures listed above were not considered sufficiantly refined. For such cases a more detailed analysis of the load paths was computed using plate theory and the effect of the openings thus assessed. These procedures are discussed on a building by building basis in the subsequent sections of this volume.

2.5 Evaluation in Terms of Criteria

The acceptance criteria for transverse loading given in Volume 1 of this report cover limitations for reinforcement ductility, masonry stress, supports and wall stability. The steel ductility and masonry stress have numerical limits and so conformance with the criteria may be obtained directly from results on a quantitative basis. Evaluation of the supports may similarly be carried out using numerical force and/or displacement values obtained from the inelastic analyses. However, the wall stability evaluation requires that engineering judgement be exercised to rate the response of the wall in terms of its overall stability.

Two response parameters are available to aid in the judgement of the wall stability - the time history of central displacements and the ratio between the wall overturning force and the inertial force restoring the wall to its undeflected position. As the walls are vertically reinforced and supported top and bottom the absolute value of deflection is not in itself a critical parameter as test results have shown masonry walls capable of resisting large displacements relative to their height without serious distress.

The primary method of judging wall stability in these analyses has been the examination of the time history of central displacements. These displacements would indicate instability by increasing displacements with a bias in one direction and the base line about which oscillations occur would shift from the zero displacement line to one side of the wall. A second means of judging stability is an equivalent static procedure for examining the P-d moments at time steps at which the displacement exceeds one-half the wall thickness. The procedure for carrying out this check is described in Section 3 of Volume 1. The ratio of restoring force to overturning force was evaluated from the results of each analysis and these are discussed in the individual wall results. As noted in the commentary to the criteria

given in Volume 1 engineering judgement must be exercised in evaluating the significance of values of this ratio.

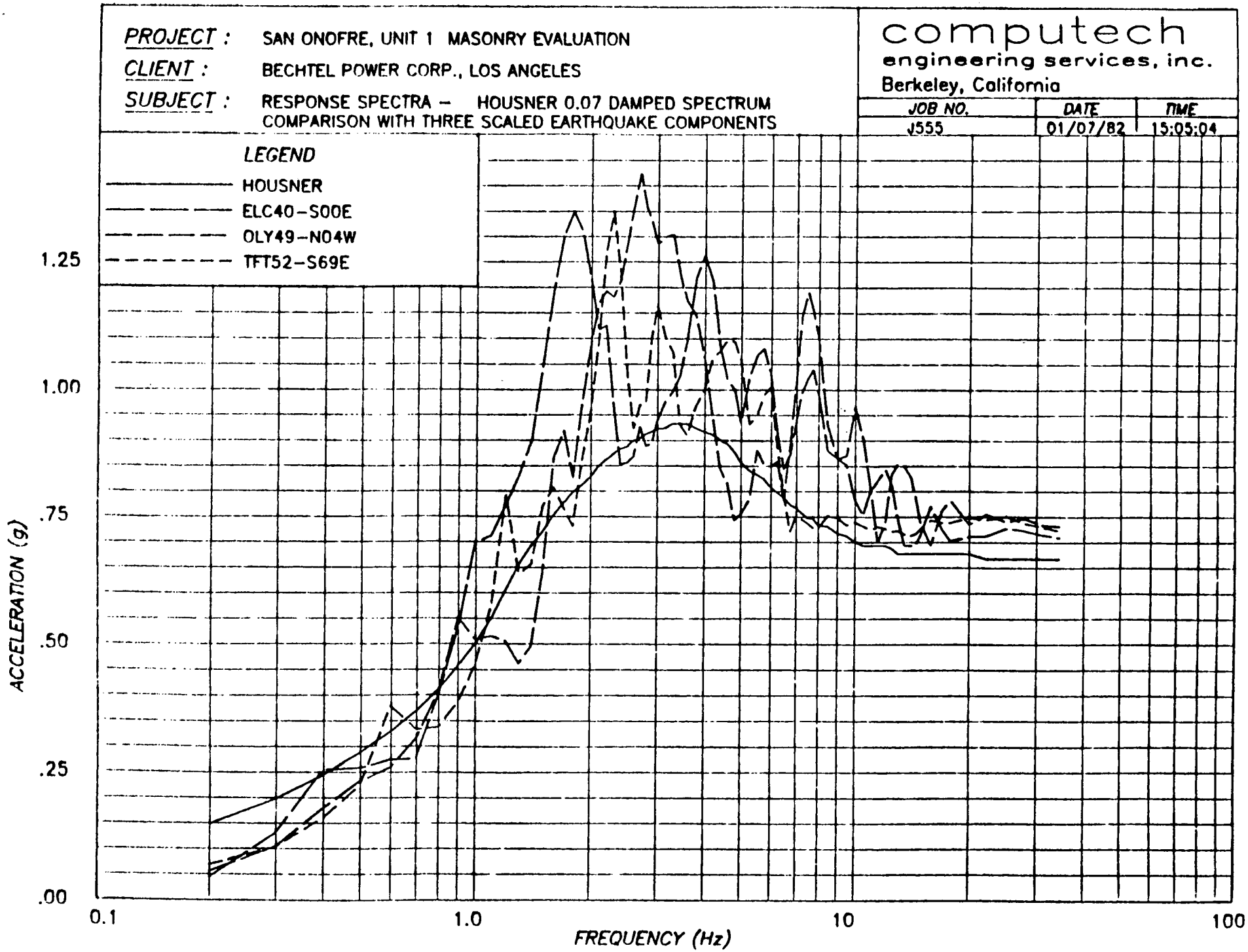


FIGURE 2.1 : COMPARISON OF SCALED EARTHQUAKES WITH HOUSNER

3 TURBINE BUILDING

3.1 Description of Walls

The Turbine Building contains a total of 12 masonry walls. These walls have been classified into three groups. I. II and III. according to their heights of 21'-4", 14'-8" and 10'-0" respectively. The walls in group II have one sub-group. IIa. to distinguish between the two types of top support conditions applicable to the 14'-8" walls. Table 3.1 lists the various walls and identifies the height. reinforcement and group number to which each wall belongs. In Figures 3.1 to 3.4 typical sections for the different wall types are shown. The walls in Groups I and II are simple spans. supported on the ground and to a steel girder at their top. The 10'-0" wall in Group III is a cantilever span Except for the cantilever wall all walls have reinforcement consisting of #5 bars at 32" centers vertically and #5 at 48" horizontally The cantilever wall has similar horizontal reinforcing but the vertical steel is #5 bars at 24" centers.

Four of the walls contain significant openings and all walls carry at least some added weight from equipment. cable trays and conduit. The effects of these openings and the added equipment masses have been taken into account in the wall evaluation.

3.2 Inelastic Analysis

For each group a DRAIN-2D model was constructed in accordance with the procedures developed in Volume 2 of this report. Three time histories scaled as described in Section 2.2 were used for the analyses and the results obtained included the displacements and forces at critical locations. The added mass due to equipment loads was included in the inelastic analysis and the effect of openings was determined based on the output results. In the following sub-sections a brief description of the models used for each wall group is given

3.2.1 Model for Group I Walls

The basic model for the Group I walls contained 124 nodal points and was pinned at the base with an elastic support at the top. The stiffness of the top support was a combination of two components. the frame stiffness of the building as a whole and the local flexibility of the support girder The frame stiffness was calculated based on the insitu configuration of the Turbine Building frames in the area of interest. The girder flexibility was computed from the existing member section properties and support conditions

The method used to model the contribution of the two components is shown in Figure 3.5. The building is reduced to an equivalent single

degree of freedom system and its mass and stiffness properties incorporated in a truss member adjacent to the fixed support. The inclusion of the building mass allows the approximate building deflections to be incorporated in the analysis results.

For this group the most significant added equipment weight was on wall TB-10 which had a number of cable trays and conduit which gave a most severe loading of 300 lbs. per foot of wall.

3.2.2 Model for Group II Walls

The model for the Group II walls was similar to that for Group I except for a lesser number of nodal points due to the shorter length. A similar method of modelling building and support flexibility was used. Two different values of support stiffness were used to include the two distinct support conditions for walls of this height. For the typical Group II walls values similar to that for Group I were used and for sub-Group IIa a much less stiff support based on the spanning of the steel section shown in Figure 3.3 was computed.

The greatest added equipment weight in this group was from wall TB-8 which had a total added weight of 1300 lbs mainly concentrated near the top of the wall. This is about one and one-half times the weight of the wall itself.

3.2.3 Model for Group III Walls

The Group III wall, TB-12, was modelled with a simple cantilever model of 63 nodes. Three cracked joints with a hinging rebar were included at the base of the wall.

The total added weight on this wall was 750 lbs. or slightly more than the wall weight. However, the bulk of the equipment contributing to this was located towards the base of the wall which lessened its impact on the overall response

3.3 Results of Analysis

The time history analyses, typically over a minimum of 1200 time steps, produced voluminous output. For this report this has been reduced to a tabulated summary of maximum response parameters and time history plots of the response of some of these parameters during the duration of the earthquake record.

In the following sections the results for each wall group in the Turbine Building are given. Tabulated results include maximum displacements, steel ductility, masonry stresses and support reactions where applicable. Plots of the variation in displacement, ductility and reaction force with time are also presented

The tables and plots refer to each of the three earthquake components listed in Section 2.2

3.3.1 Group I

A summary of maximum results for the walls in this group is given in Table 3-2 and time history plots in Figures 3.6 through 3.14. The figures represent the time history response for displacements, steel ductility and reaction forces for the El Centro, Olympia and Taft earthquake records.

The maximum mid-span displacement of 10.23" was reached during the analysis for the Olympia record, which also produced the highest steel ductility of 24.5 and masonry compressive stress of 314 psi. The Olympia record in general produced slightly higher response values than the El Centro earthquake record with the Taft record producing considerably lower response than the other two records. The maximum support displacement during the Olympia time history analysis was 2.57" and the associated maximum reaction force was 992 lbs on the one foot strip of wall.

Because of the method of modelling the wall structure interaction as shown on Figure 3.5 the maximum support response is the product of a multi-mode system, and so instantaneous values may reach considerably higher values than would be predicted from a single degree of freedom type model. Figure 3.11 shows that the high reaction force is of very short duration and apart from the two high peaks the next highest value is only about two thirds the magnitude. The different frequency of the wall after yielding and the elastic support frequency are shown on Figure 3.9, where the wall period as illustrated by the center displacements elongates while the support displacements retain essentially the same frequency content.

For this wall analysis the added mass was assumed distributed over a 4'-0" strip of wall and the one foot strip analyzed had an added mass approximately one quarter the weight of the wall itself.

The analysis of the same group I walls to the same earthquake records but with only the mass of the wall itself gave maximum center displacements of 9", steel ductility of 20 and reaction force of 620 lbs. These displacements and ductilities are of the same order of magnitude but it is apparent that the added mass has a substantial effect on the maximum reaction forces.

3.3.2 Group II

Table 3.3 summarizes the maximum response parameters for the Group II walls. In Figures 3.15 to 3.23 the displacements, ductility and reaction force for each of the three earthquake records are plotted.

Maximum displacements of 8.6", steel ducility of 25.0 and masonry compressive stress of 314 psi were similar to the corresponding values for the Group I walls. For this group the maximum wall response occurred during the El Centro analysis but the maximum support displacement and force occurred under the Olympia ground acceleration.

The support reaction force for these walls was particularly high, with a maximum value of 2829 lbs; over twice the value obtained from the Group I wall analysis. The added weight on this wall was very high, about 1300 lbs. This arises from the equipment mounted on wall TB-8, which carries the contributary weight from the 6'-7" span of three 24" cable trays. This provides a lumped weight of 700 lbs and in addition a number of large diameter conduits provide an additional 585 lbs. Both these weights are concentrated near the top of the wall and thus have maximum effect on the top reaction.

This wall was also analyzed for the condition without any added weights. For this case maximum displacements were less than 2" and a maximum steel ducility of 3 indicated very low levels of inelastic action. The maximum support reaction in these analyses was only 833 lbs, or less than one third the values determined when the equipment mass was included. Therefore, the added mass affects the dynamic properties and the interaction between the elastic support and the yielding wall such that the increase in the reaction force is greater than the proportion of added mass.

Figure 3.20 illustrates the instantaneous nature of the maximum reaction, where an isolated peak has a magnitude about 50% greater than the next highest peak

Two of the walls in Group II, TB-1 and TB-7, had different support conditions than the remainder of the walls in the group. These walls were supported on a simply supported steel beam (see Figure 3.3) and because of the louver above were not connected directly to the roof diaphragm. This support was considerably more flexible than the "typical" support condition. The analysis for the El Centro earthquake motion was therefore repeated with the lower elastic support values. The results from this analysis are summarized in Table 3.4 and time history plots are presented as Figures 3.24 to 3.26

Due to the reduced frequency of the system with the more flexible support the response was much lower than with the stiffer support The maximum displacement and steel ductility reduced to only about one half the previous values The support displacement was over twice the magnitude but the reaction force was reduced Note that the added equipment weights on these walls were much less severe than on the wall TB-8.

As the response parameters were reduced to a large extent it was concluded that this support configuration did not produce the critical case for the walls in Group II and the Taft and Olympia time histories were not analyzed

3.3.3 Group III

Wall TB-12, the only wall in Group III, differed from the other walls in the Turbine Building in that it was unsupported at its top edge. Therefore the response parameters listed in Table 3.5 consider only top deflection, steel ductility and masonry stress. In Figures 3.27 through 3.32 the displacement and ductility time history is plotted for each of the three earthquake records.

The El Centro record produced the greatest wall response, with a maximum displacement of 7.9" and steel ductility of 16.8. The Olympia response was slightly lower and the Taft reponse only about one half that of the other two records. Masonry compressive stresses were all of similar magnitude as once the rebar reached its yield force the only increase in tension, and thus in masonry compression required to equilibrate it, could arise from strain hardening.

3.4 Evaluation of Results

The acceptance criteria for transverse analysis of masonry walls at the San Onofre, Unit 1 plant are given in Section 2.1 of Volume 1 of this report. Commentary and justification for this criteria is presented in Section 3.1 of the same volume. In the following sections the results of the Turbine Building walls are evaluated in terms of these criteria.

3.4.1 Reinforcement Ductility

The acceptance criteria provide for a maximum steel ductility of 45, based on one-half the specified minimum elongation for reinforcing steel. The maximum values obtained from the analyses are as listed below:

Group I	24.5
Group II	25.0
Group IIa	8.5
Group III	16.8

Therefore all walls in the Turbine Building are within the limits for steel ductility.

3.4.2 Masonry Compressive Stress

The acceptance criteria restrict the maximum masonry stress to 0.85 f'm, based on a uniform distribution. The maximum stress for each wall has been computed from the results of each analysis as:

$$fm = F/(t \times b)$$

where

F = Maximum rebar force

t = Face shell net thickness
b = Section width = 12"

Computed on this basis the maximum values were:

Group I	314 psi
Group II	314 psi
Group IIa	300 psi
Group III	362 psi

For the San Onofre, Unit 1 walls the value of f'm is 1350 psi giving a maximum allowable masonry stress of 1147 psi. Therefore all the walls were within the criteria limits.

3.4.3 Top Wall Support

The criteria require that flexible supports be capable of accomodating the vertical component of displacement of the analysis and further that rigid supports be capable of resisting the maximum force. The supports for the Turbine walls in Groups I and II are flexible and so must accomodate the maximum vertical deflection of the walls. This is unlikely to cause problems because of the several sources of flexibility inherent in the support conditions. The most significant of these would be flexure of the bolt connecting the wall to the girder, which has a typical outstanding length of over 6", and the torsional flexure of the W24 girders which have low resistance to rotation due to applied torque.

More critical for the support evaluation would be the level of horizontal support loads indicated by the analyses. These loads form the basis of evaluating the adequacy of individual connections. As discussed in the presentation of results the magnitude of the support reaction increases markedly due to the effect of added equipment weights on the wall. For the inclusion of these weights in the Group II analyses the total weight between equipment supports was added to the one foot strip analyzed. Therefore the support reaction applies only to the one foot strip and not to the wall strips on either side. It would be erroneous to multiply this value directly by the length of wall to obtain the total load contributing to a particular support.

For the Group II walls the support adequacy was assessed using a weighted average of the results both with and without the added mass over a length of wall equal to the distance between supports for the heaviest supported equipment, typically about 6'-0" for these walls. This method would give an average support force of:

$Fs = (2829 + 5x833)/6 = 1166$ lb/ft for Group II

For Group IIa the results without added mass were not obtained. The maximum support reaction with added mass was 1015 lbs. or about one half that for Group II. Therefore an average reaction force of 600 lb/ft is considered appropriate for walls in this sub-Group

A further factor in using the weighted average reaction is the presence of well reinforced 2'-0" bond beams at all support points. These beams would be effective in distributing concentrated reactions from the local equipment loads.

3.4.4 Wall Stability

The first requirement of the criteria for wall stability is that there be no permanent offset in the wall response after the period of strong shaking. Satisfaction of this criterion may be judged from the time history plots presented in Figures 3.6 to 3.32. For all analyses the response remained stable and in no case was there a permanent deformation after the strong motion duration.

The second criterion for stability is that whenever the wall deflection exceeds the wall thickness there be a restoring force equal to or greater than and opposite in magnitude to the inertial force tending to overturn the wall. The post-processor used for the DRAIN-2D program produces the value of this ratio for all time steps at which the deflection exceeds the wall thickness.

For the analyses carried out on the Turbine Building the factored deflections exceeded the wall thickness during the El Centro and Olympia records for the Group I walls and for the El Centro record only for the Group II and Group III walls. For these analyses the value of the ratio of the restoring force to the overturning force at all time steps at which the central deflection exceeded the thickness of the wall, 7.625 inches, was obtained.

As discussed in the Commentary on the Criteria in Volume 1 engineering judgement must be exercised in interpreting the likely effect on the wall when the value of the restoring inertia force is not greater than and opposite in sign to the overturning secondary forces. In general this is of concern only when the wall deflections are increasing. For both the Group II and Group III walls all timesteps at which the stability was required to be checked had inertia forces greater in magnitude than the incremental overturning force and opposite in sign. For Group I walls a limited number of timesteps during which the deflection was greater than the thickness had inertia forces less than the overturning moment However these periods were of very short duration (maximum 0.045 secs) and therefore it is not considered that they would effect the wall stability. Thus the stability of all walls analyzed in the Turbine Building was considered satisfactory.

3.4.5 Wall Openings

Walls TB-1, TB-7, TB-8 and TB-11 have significant openings. Of these the most severe are those in wall TB-11, a Group I wall with a 12'-0" opening for a roll up door and the portion of TB-1 in Group II which

has a 12'-10" bus opening just above the wall mid-height. Openings in the remaining walls are relatively small, less than 4'-0" and the trimmer bars supplied for such openings are generally twice the area of the bars curtailed because of the opening.

The 12'-0" opening occurs in a portion of wall 40'-0" long between control joints which also has a 3'-4" door opening. Together these two openings account for 38.3% of the total wall length. A total of 7 #5 bars are curtailed because of the openings and the trimmer bars added around the openings add 8 #5 bars. Therefore the first criteria for openings is met, i.e. no area of steel is lost. For the masonry stress an ultimate strength procedure was followed assuming the effective area of face shell in compression to be reduced proportionately to the ratio of openings to solid wall. Using this reduced width the depth to the neutral axis was computed to be 0.57", less than one half the face shell thickness of 1.25". Therefore the effect of openings in the Group I walls is considered such as not to change the rating of the walls which were all satisfactory in terms of the other acceptance criteria.

The portion of TB-1 which is 15'-8" high and thus falls into Group II is 28'-0" long between control joints and contains an opening 12'-10" wide, or 45.8% of the wall width. The standard detail for such openings requires that 2-#5 bars each side are used. Therefore the number of bars curtailed, 5-#5 bars, is greater than the number of added bars from the trimmers. On this basis the effect of the openings violates the criteria. The opening is in a portion of wall with significant equipment loads as the wall beneath the opening carries the support reactions from 2 24" and 2 12" cable trays. For this wall portion an elastic plate analysis was carried out to determine an accurate distribution of loads from both vertical and horizontal spanning. The results of this analysis are described in the following section.

3.5 Elastic Analysis of Wall TB-1

The portion of wall TB-1 which is 15'-8" high contains a bus opening which extends over 12'-10" of the 28'-0" long wall portion. The wall elevation is shown in Figure 3.33. Because of the extent of the opening the simplified criteria used for assessing the effects of openings were not considered valid. An elastic plate analysis considering both horizontal and vertical spanning was carried out to enable an evaluation of the impact that the opening would have on the inelastic analysis results. To provide a basis on which to derive the effect of the opening the elastic analysis was run both with and without the opening. In the following sections the method of carrying out this elastic analysis is briefly described and the results are discussed.

3.5.1 SAP Model

The wall was modelled for analysis using the SAP program based on a mesh layout as shown in Figure 3.34. The wall was formed of plate

elements with beam elements to model the W8 girder at the wall top and boundary elements to obtain support reactions around the wall boundaries. Simple support conditions were assumed at the wall base and a pinned support was assumed at each end of the top W8 girder.

Plate properties were based on an orthotropic formulation to take account of the differing vertical and horizontal reinforcement. The modulus of elasticity and plate thickness were adjusted so as to give an effective stiffness equivalent to 1.5EIcr, where E is the masonry modulus of 1200 f'm and Icr is the transformed moment of inertia based on the area of reinforcement. The factor of 1.5 was the same as that determined in the inelastic analyses to give deflections equivalent to those obtained from test results (see Volume 2).

3.5.2 Method of Analysis

An eigenanalysis was carried out on the model as described above and the first five modes obtained. Using the results obtained the model was then analyzed using the response spectrum method with the 7% damped Housner spectrum as input. The analysis was then repeated with the wall modelled as before but with the opening removed, i.e. the wall solid over its entire area.

The individual modal displacements and stresses were combined using the square root of the sum of the squares procedure to obtain the final results. The Computech post-processor program GENOUT was used to scan the output file and provide tables of maximum response quantities. In the following section the results are discussed.

3.5.3 Results of Analysis

The results from the elastic analyses of wall TB-1 are summarized in Table 3.6 and an example plot of the first mode from the eigenanalysis is reproduced as Figure 3.35 The mode shape plotted is for the wall with openings

The wall with openings had a fundamental frequency of 2.59 hz which increased to 2.84 hz for the stiffer wall without openings. The elastic displacements and moments were much larger for the wall with openings than for the wall without openings However in the former case the maximum values listed in Table 3.6 are generally the result of local effects around the opening

For the typical wall reinforcing yielding of the reinforcement would occur at a moment of 1268 lb.in/in for vertical spanning and 860 lb.in/in for horizontal spanning These moments were exceeded in the vertical direction for both analyses and the horizontal moment for the wall with openings also exceeded the ultimate value However in the latter case only one element had a moment greater than the computed ultimate horizontal moment and this was in the region directly below the opening

where the extra trimmer bars effectively provide additional strength. For this same analysis the maximum vertical moments also occurred immediately adjacent to the opening where the strength of the wall is augmented by the trimmer bars. Excluding these moments in the strip governed by trimmer bar strength the highest vertical moment obtained from the elastic analysis for the wall with openings was 2181 lb.in/in which was 26% greater than the equivalent value for the wall with no openings.

The maximum elastic displacement occurred immediately beneath the opening and was due to the horizontal deflection of the unsupported wall at this position. In Figure 3.36 the wall deflections on a wall strip immediately adjacent to the opening position have been plotted for each analysis. The presence of the opening increased the deflection by 27%.

Generally then the opening increased both the elastic moments and the deflections by less than 30% when compared to the elastic results of the wall without openings. It is likely that the order of increase in the inelastic response of the wall with the openings inelastic response would be of the same order of magnitude as the increase in elastic response and Table 3.4 shows that the inelastic analysis of the wall without the opening had a maximum steel ductility of only 8.5, or 20% of the allowable value. It is clear from these elastic analyses that the effect of the opening would not cause the 500% increase in ductility needed to exceed the specified limit.

WALL I.D. NUMBER	HEIGHT (Feet)	REINFORCING		OPENINGS	GROUP
		Vertical	Horizontal		
TB-1	21'-4" *	#5 at 32"	#5 at 48"	YES	I.IIa
TB-2	14'-8"	#5 at 32"	#5 at 48"	NO	II
TB-3	14'-8"	#5 at 32"	#5 at 48"	NO	II
TB-4	14'-8"	#5 at 32"	#5 at 48"	NO	II
TB-5	21'-4"	#5 at 32"	#5 at 48"	NO	I
TB-6	21'-4"	#5 at 32"	#5 at 48"	NO	I
TB-7	21'-4" *	#5 at 32"	#5 at 48"	YES	I.IIa
TB-8	14'-8"	#5 at 32"	#5 at 48"	YES	II
TB-9	20'-8"	#5 at 32"	#5 at 48"	NO	I
TB-10	20'-8"	#5 at 32"	#5 at 48"	NO	I
TB-11	20'-8"	#5 at 32"	#5 at 48"	YES	I
TB-12	10'-0"	#5 at 24"	#5 at 48"	NO	III

* Also has portions 15'-8" high

TABLE 3-1 : TURBINE BUILDING MASONRY WALLS

	EARTHQUAKE RECORD		
	EL CENTRO 1940	TAFT 1952	OLYMPIA 1949
DISPLACEMENTS (inches)			
Mid-Span Maximum	8.69	6.99	9.96
Mid-Span Minimum	- 9.68	- 7.21	-10.23
Support Maximum	1.87	1.49	1.81
Support Minimum	- 1.59	- 1.61	- 2.57
STEEL STRAIN RATIO (Eu/Ey)	21.4	15.6	24.5
SUPPORT REACTION (lbs)	-928.	-552.	-992.
MASONRY COMPRESSIVE STRESS (psi)	311.	306.	314.

TABLE 3.2 : SUMMARY OF RESULTS — GROUP I WALLS

	EARTHQUAKE RECORD		
	EL CENTRO 1940	TAFT 1952	OLYMPIA 1949
DISPLACEMENTS (inches)			
Mid-Span Maximum	8.18	4.74	4.77
Mid-Span Minimum	- 8.60	- 5.13	- 5.10
Support Maximum	1.52	1.52	1.64
Support Minimum	- 1.62	- 1.44	- 1.84
STEEL STRAIN RATIO (Eu/Ey)	25.0	14.8	14.7
SUPPORT REACTION (lbs)	2649.	1916.	2829.
MASONRY COMPRESSIVE STRESS (psi)	314.	305.	305.

TABLE 3.3 : SUMMARY OF RESULTS — GROUP II WALLS

	EARTHQUAKE RECORD		
	EL CENTRO 1940	TAFT 1952	OLYMPIA 1949
DISPLACEMENTS (inches)			
Mid-Span Maximum	4.84	-	-
Mid-Span Minimum	-4.36	-	-
Support Maximum	2.90	-	-
Support Minimum	-3.73	-	-
STEEL STRAIN RATIO (Eu/Ey)	8.5	-	-
SUPPORT REACTION (lbs)	1015	-	-
MASONRY COMPRESSIVE STRESS (psi)	300	-	-

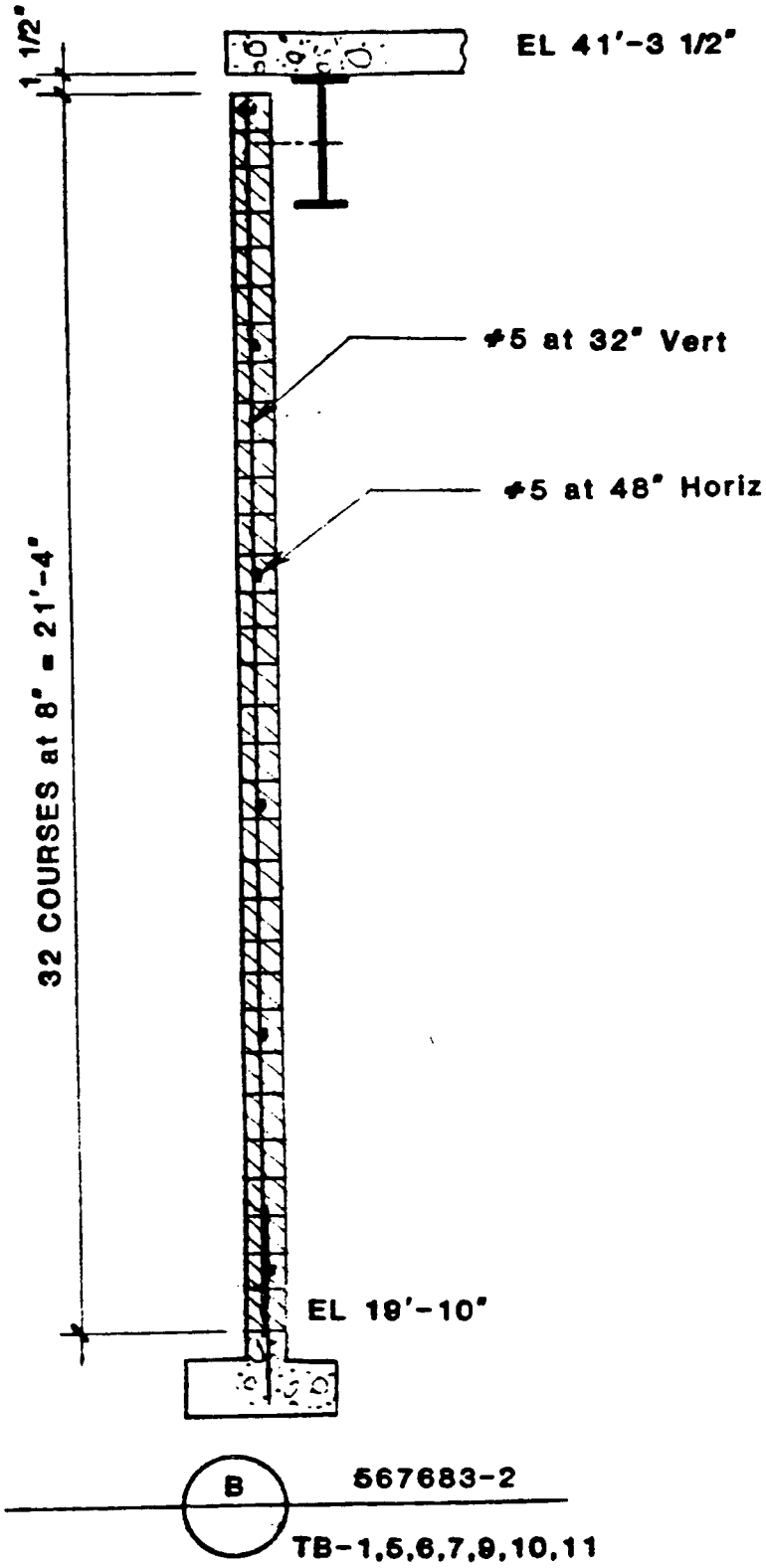
TABLE 3.4 : SUMMARY OF RESULTS — GROUP IIa WALLS

	EARTHQUAKE RECORD		
	EL CENTRO 1940	TAFT 1952	OLYMPIA 1949
DISPLACEMENTS (inches)			
Top Maximum	7.93	3.24	6.28
Top Minimum	-7.49	-3.19	-5.93
STEEL STRAIN RATIO (Eu/Ey)	16.8	5.0	11.7
MASONRY COMPRESSIVE STRESS (psi)	362	350	356

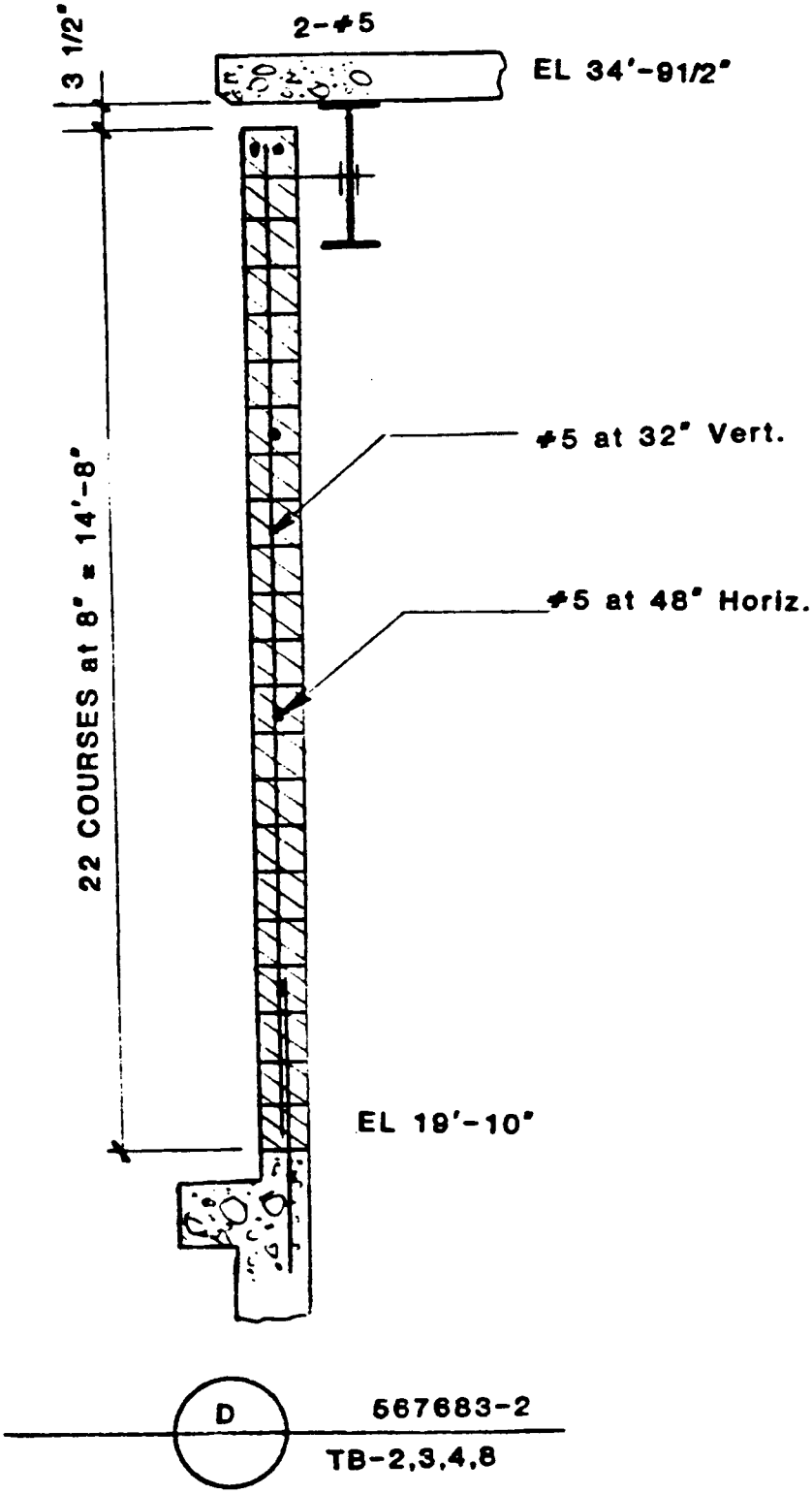
TABLE 3.5 : SUMMARY OF RESULTS — GROUP III WALLS

	WITH OPENING	WITHOUT OPENING
FREQUENCIES		
Mode 1	2.59 hz	2.84 hz
Mode 2	3.84 hz	4.15 hz
Mode 3	4.68 hz	4.64 hz
Mode 4	5.70 hz	6.55 hz
Mode 5	8.15 hz	8.73 hz
MAXIMUM DISPLACEMENT (inches)	2.51	1.61
MAXIMUM MOMENTS (lb-in/in)		
Vertical Spanning	2963	1728
Horizontal Spanning	939	759

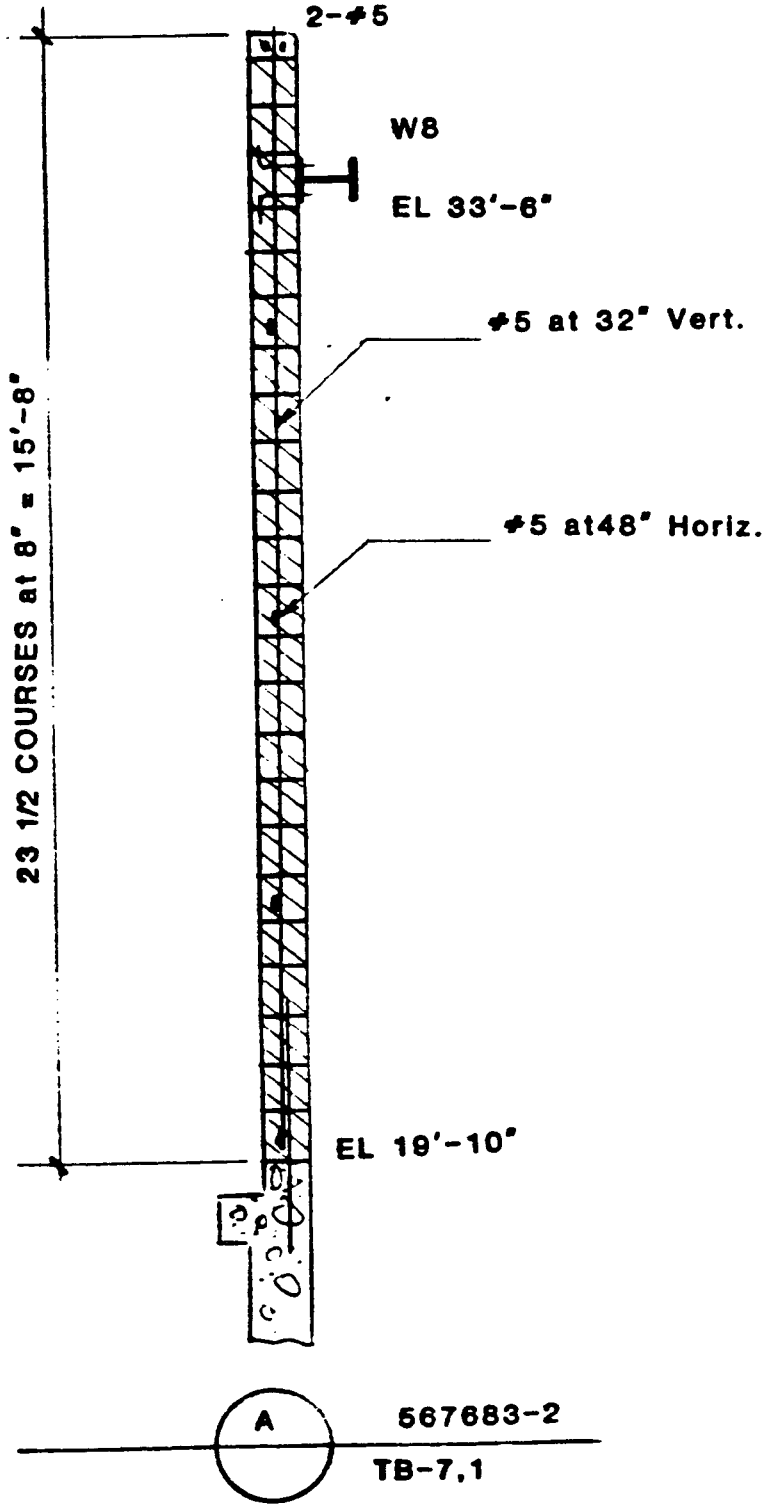
TABLE 3.6 : ELASTIC ANALYSIS OF WALL TB-1



PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
DRAWN	TURBINE BUILDING - GROUP I WALLS	3.1
CHECKED		



PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
DRAWN	TURBINE BUILDING - GROUP II WALLS	3.2
CHECKED		



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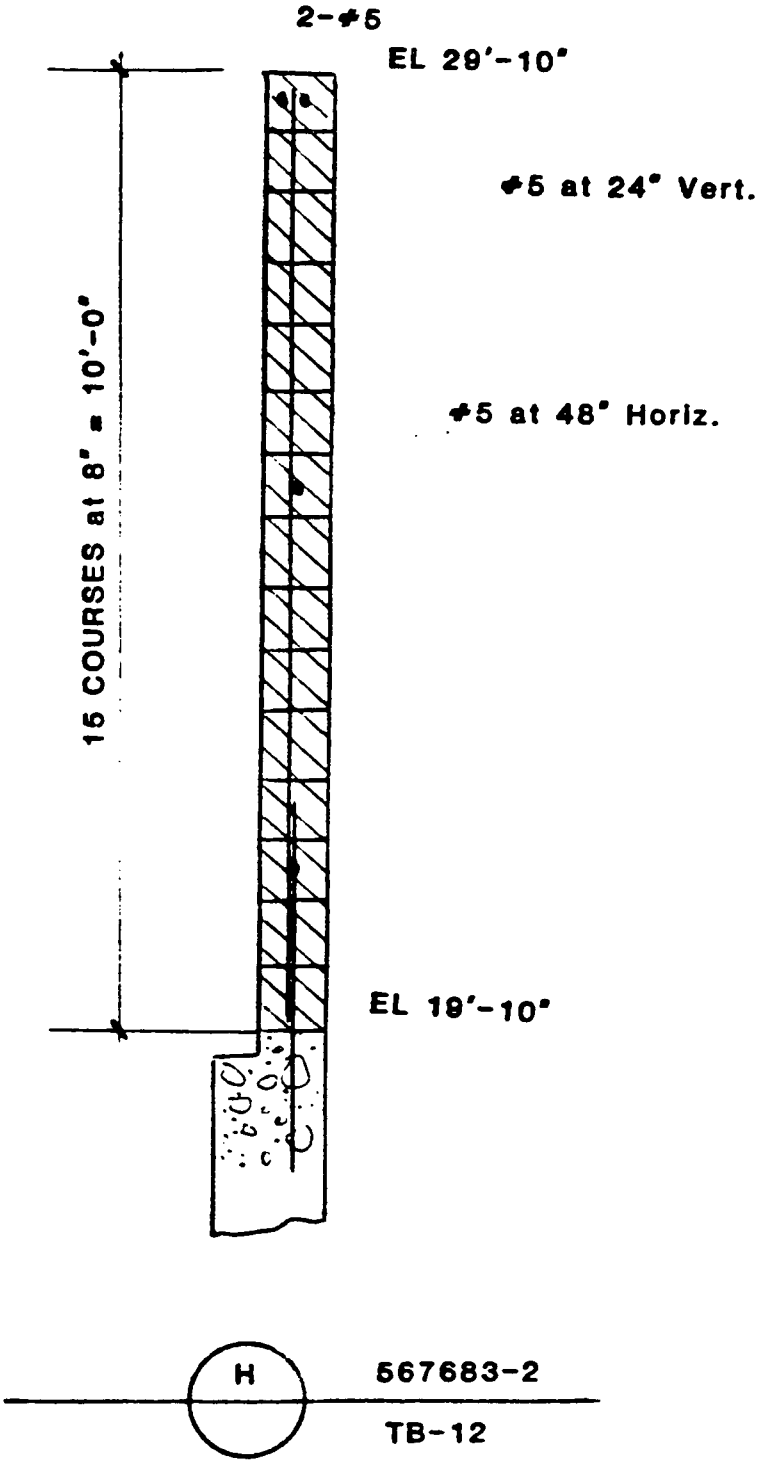
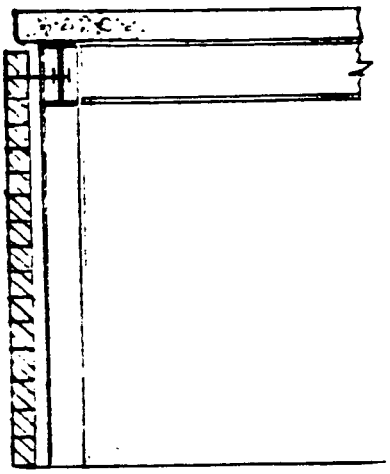
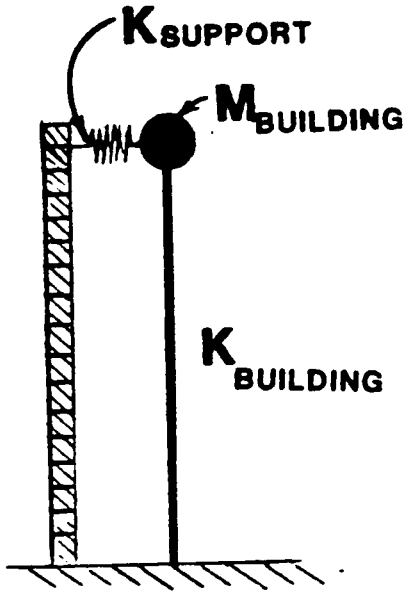


PROJECT NO. 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO.
DRAWN	TURBINE BUILDING - GROUP III WALLS	3.4
CHECKED		

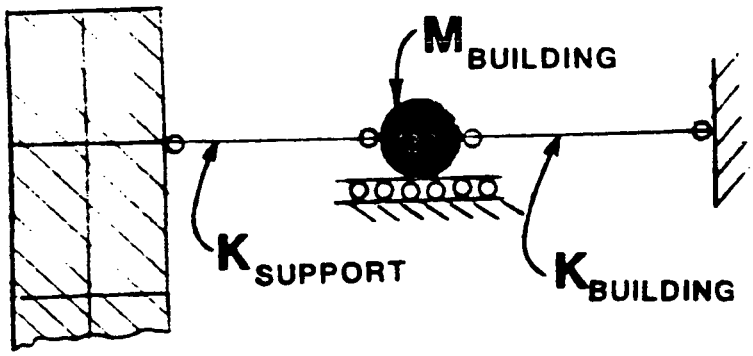
COMPUTECH



SECTION THROUGH BUILDING



BUILDING AS 1 D.O.F SYSTEM



DRAIN 2-D MODEL

PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO. 3.5
DRAWN	TURBINE BUILDING - SUPPORT MODEL	
CHECKED		

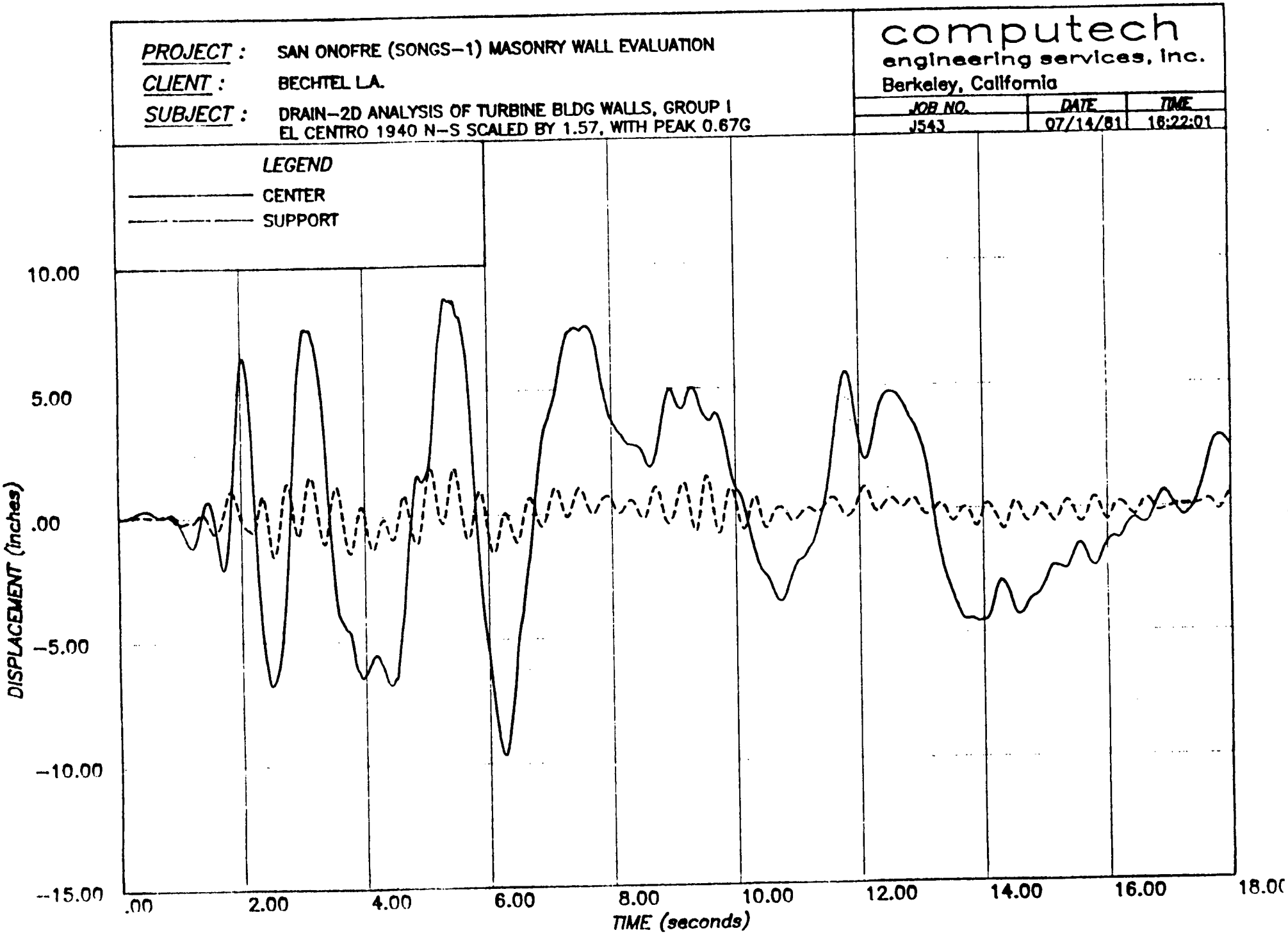


FIGURE 3.6 TURBINE BUILDING - GROUP I :EL CENTRO

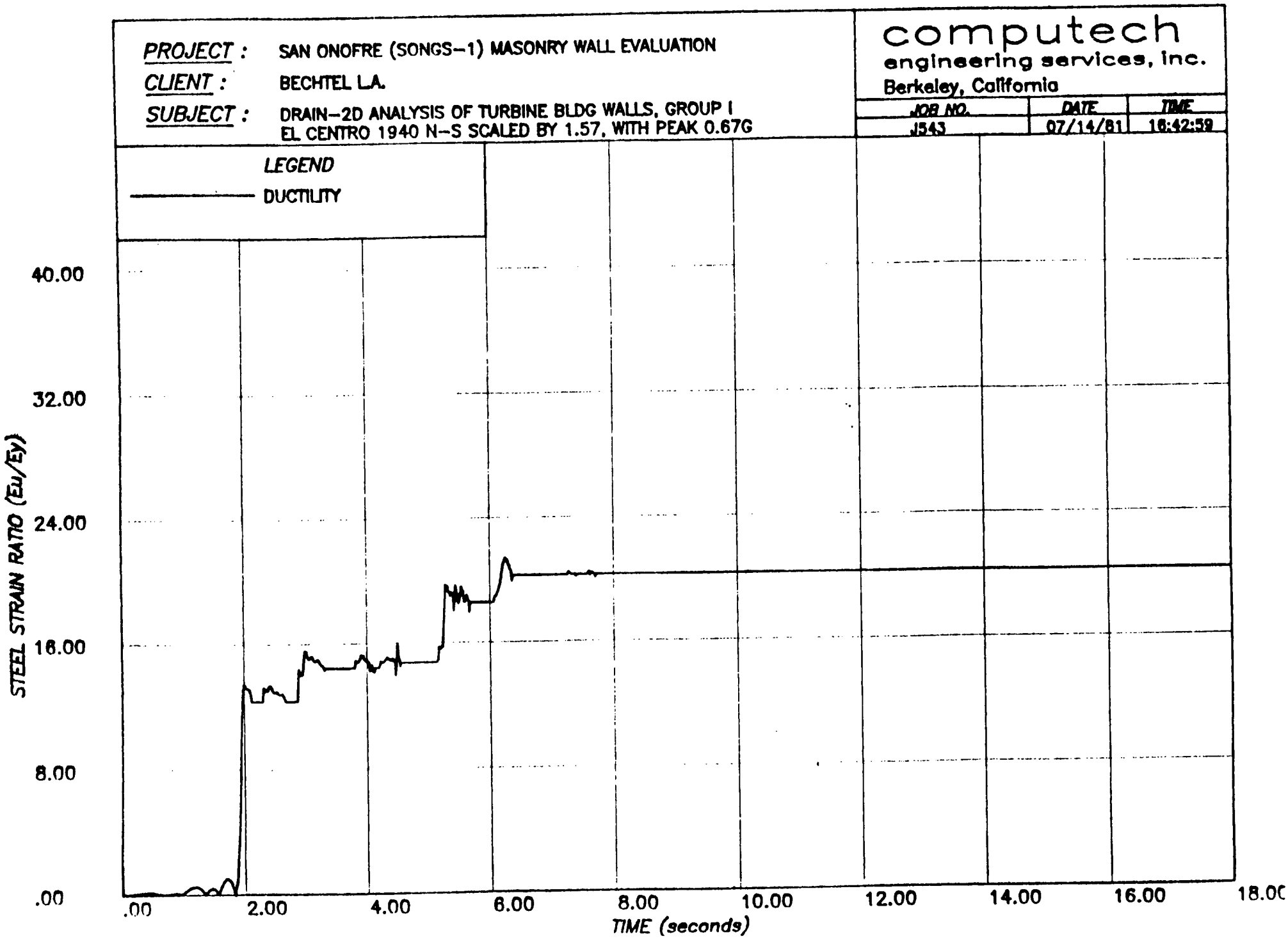
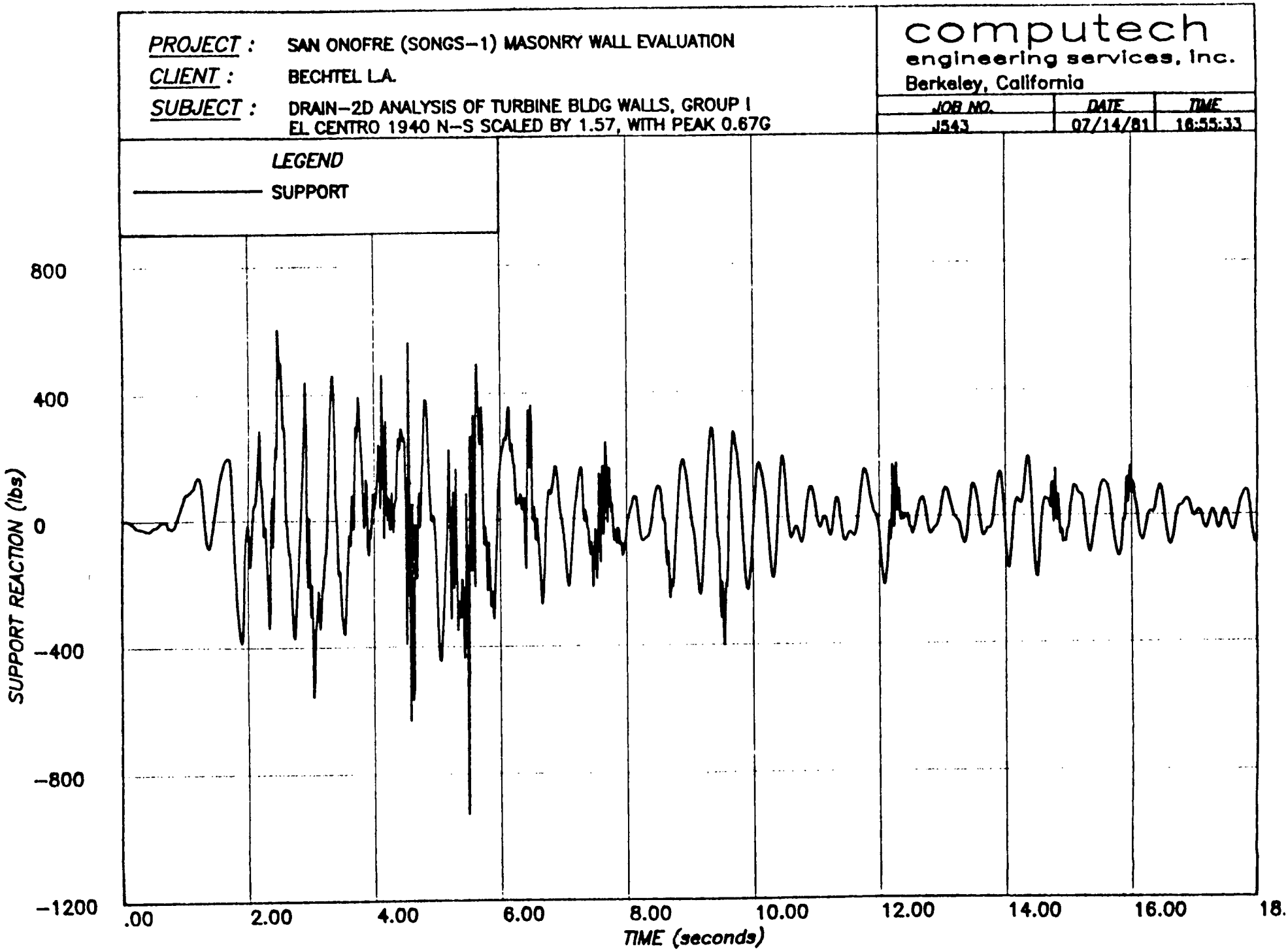


FIGURE 3.7 TURBINE BUILDING - GROUP I EL CENTRO


PROJECT : SAN ONOFRE (SONGS-1) MASONRY WALL EVALUATION
CLIENT : BECHTEL L.A.
SUBJECT : DRAIN-2D ANALYSIS OF TURBINE BLDG WALLS, GROUP I
EL CENTRO 1940 N-S SCALED BY 1.57, WITH PEAK 0.67G
computech
engineering services, inc.
Berkeley, California
JOB NO. J543
DATE 07/14/81
TIME 16:55:33
LEGEND
SUPPORT
800
400
0
-400
-800
-1200
SUPPORT REACTION (lbs)
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2.00
4.00
6.00
8.00
10.00
12.00
14.00
16.00
18.
TIME (seconds)

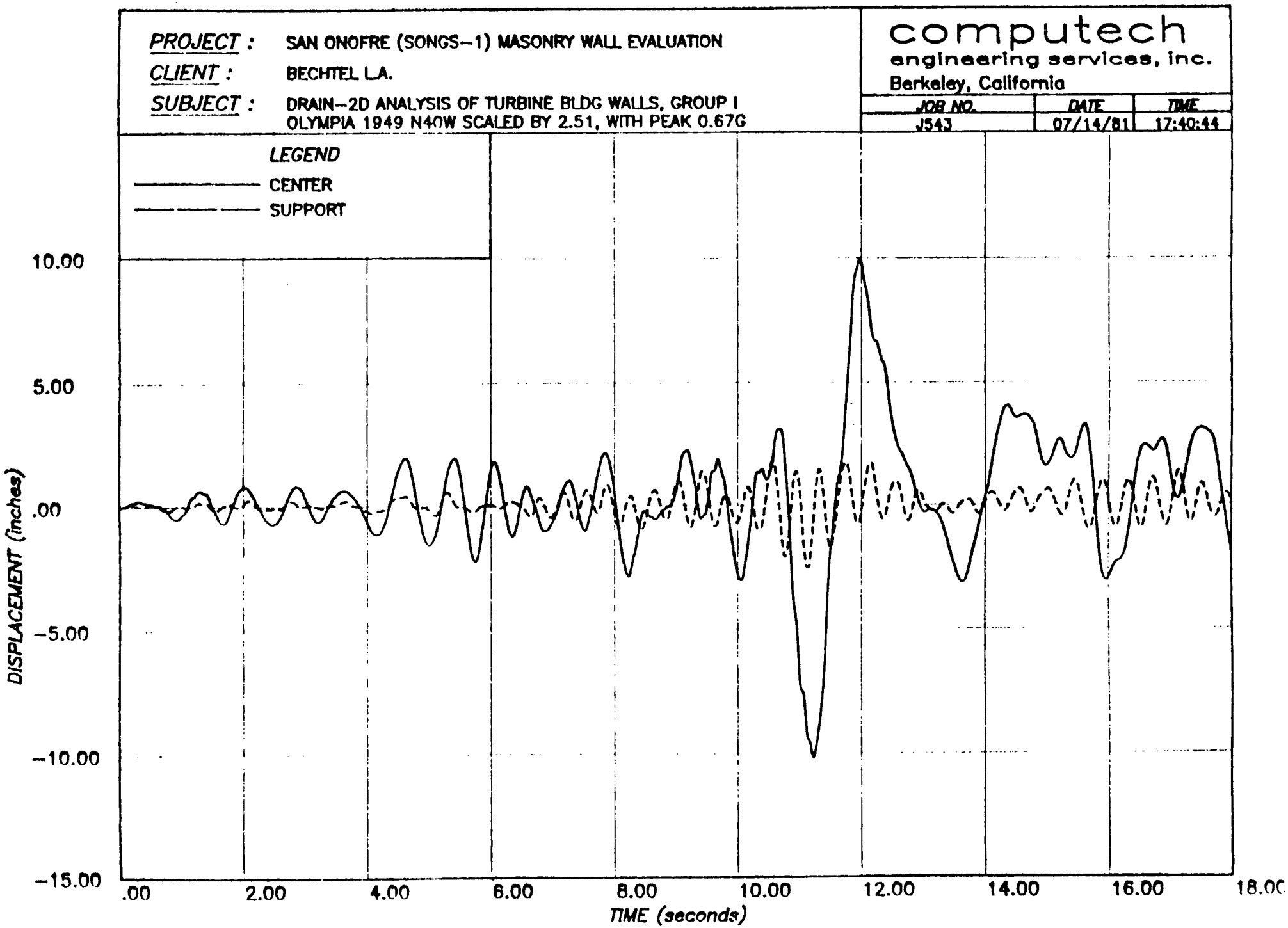


FIGURE 3.9 TURBINE BUILDING - GROUP I :OLYMPIA

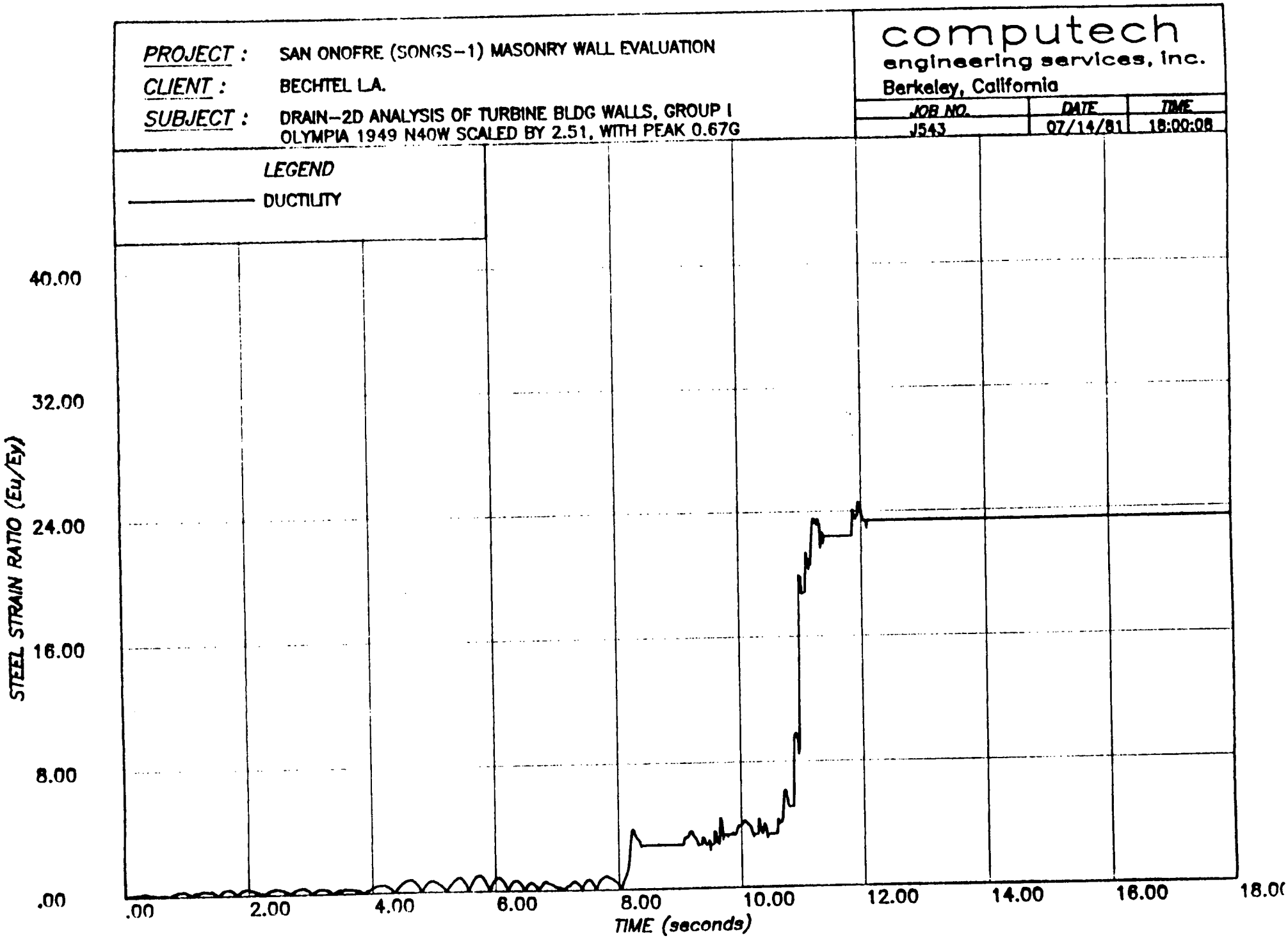


FIGURE 3.10 TURBINE BUILDING - GROUP I :OLYMPIA

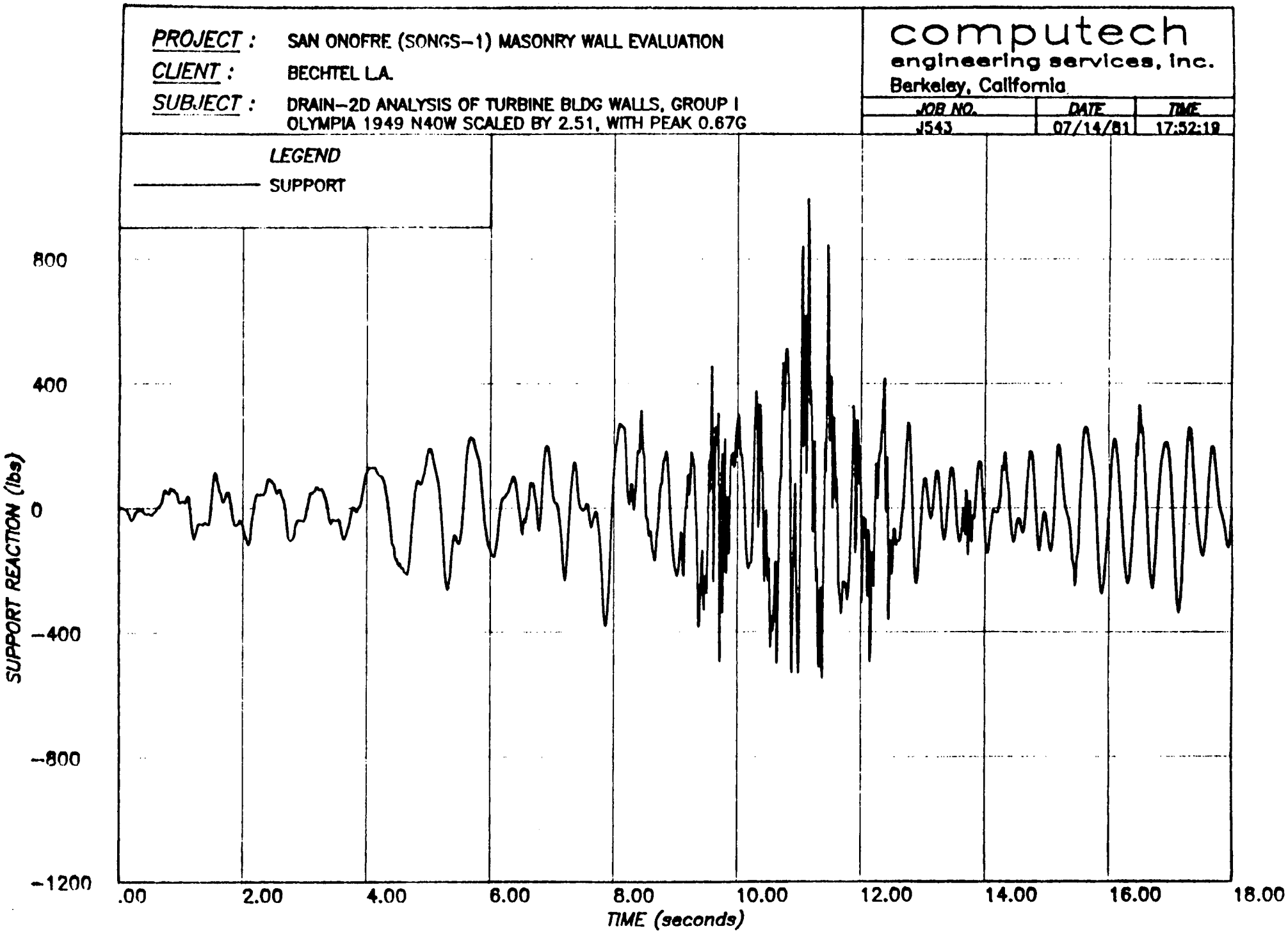


FIGURE 3.11 TURBINE BUILDING - GROUP I :OLYMPIA

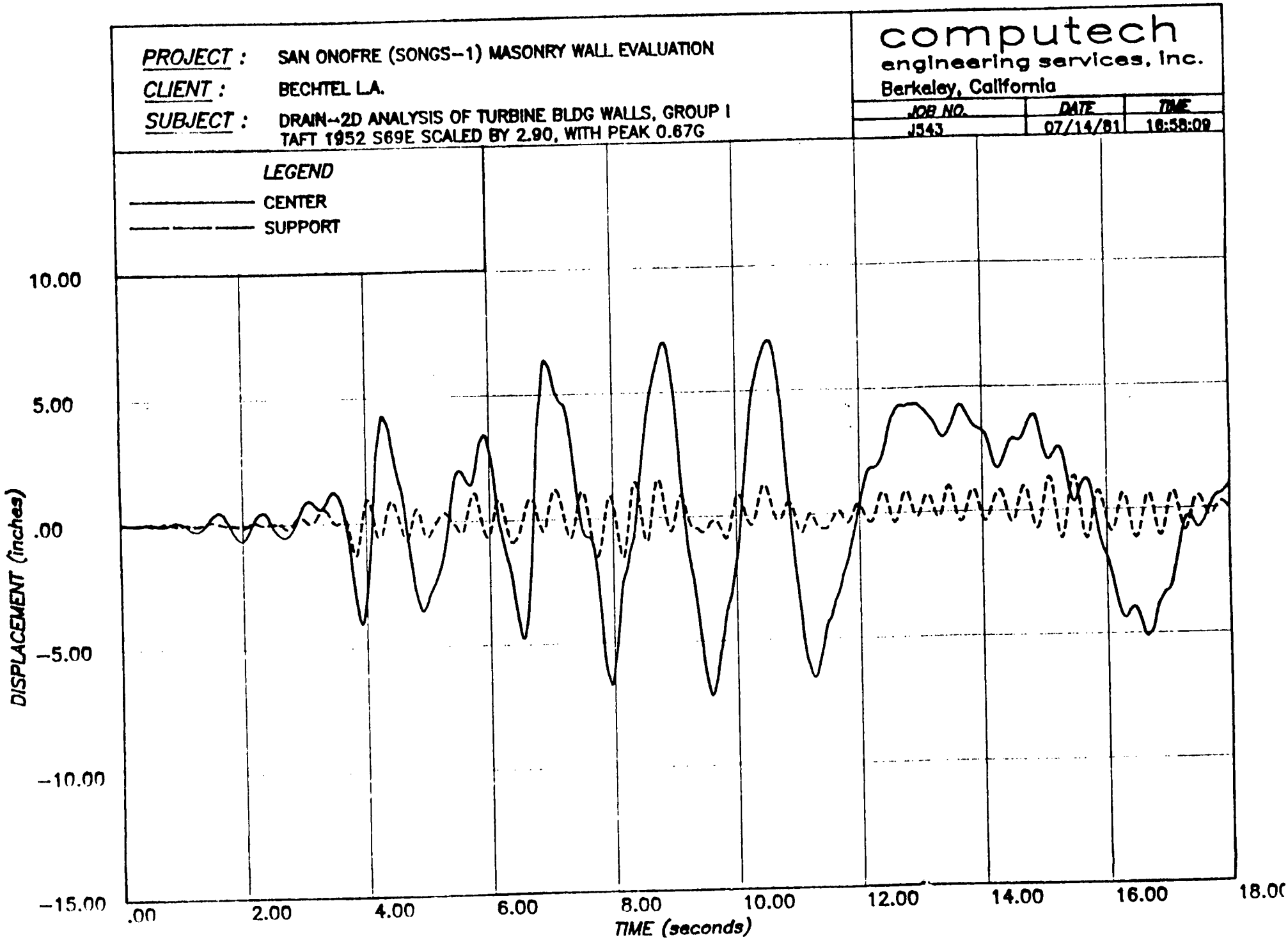


FIGURE 3.12 TURBINE BUILDING - GROUP I :TAFT

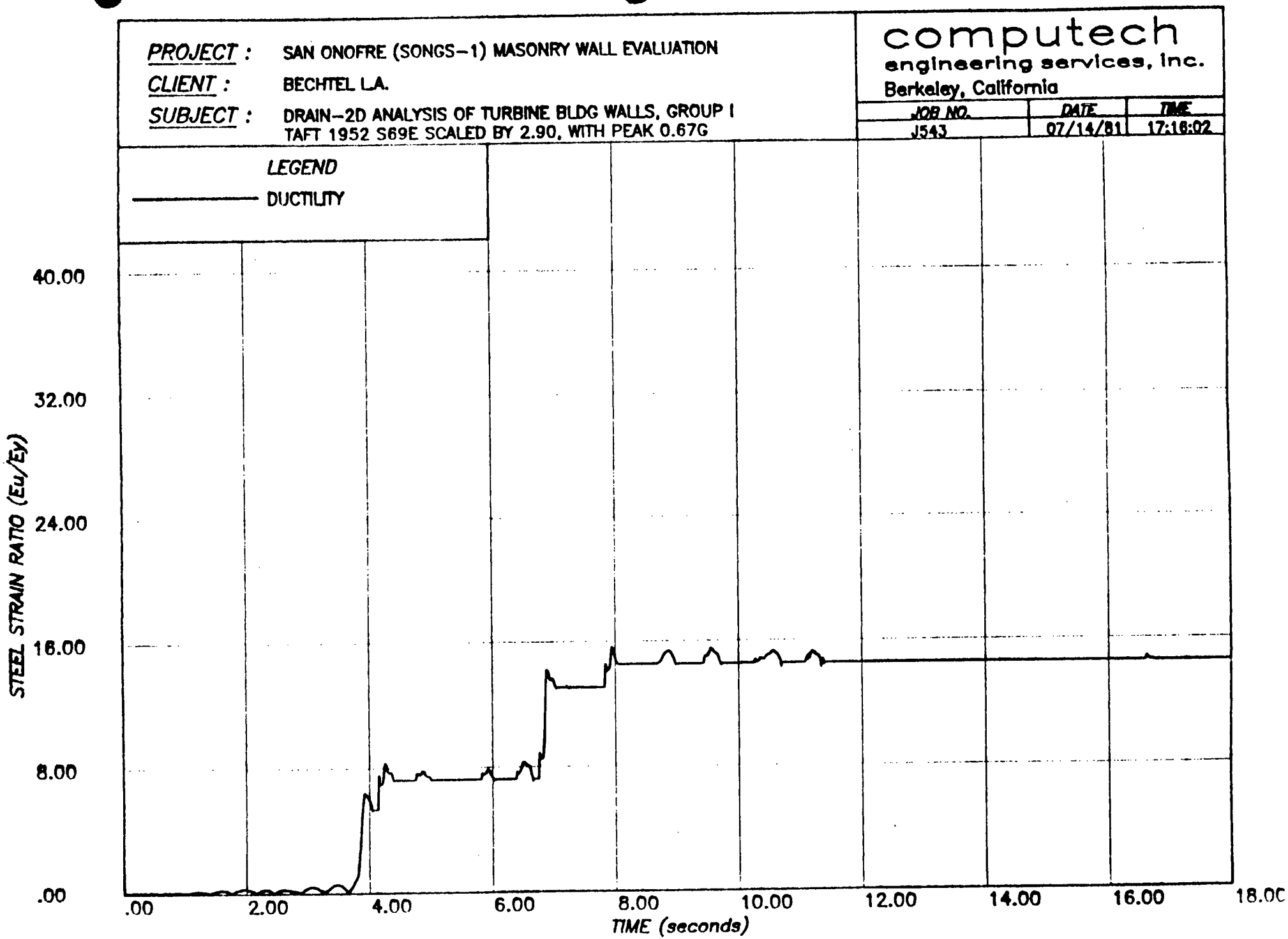


FIGURE 3.13 TURBINE BUILDING - GROUP I :TAFT

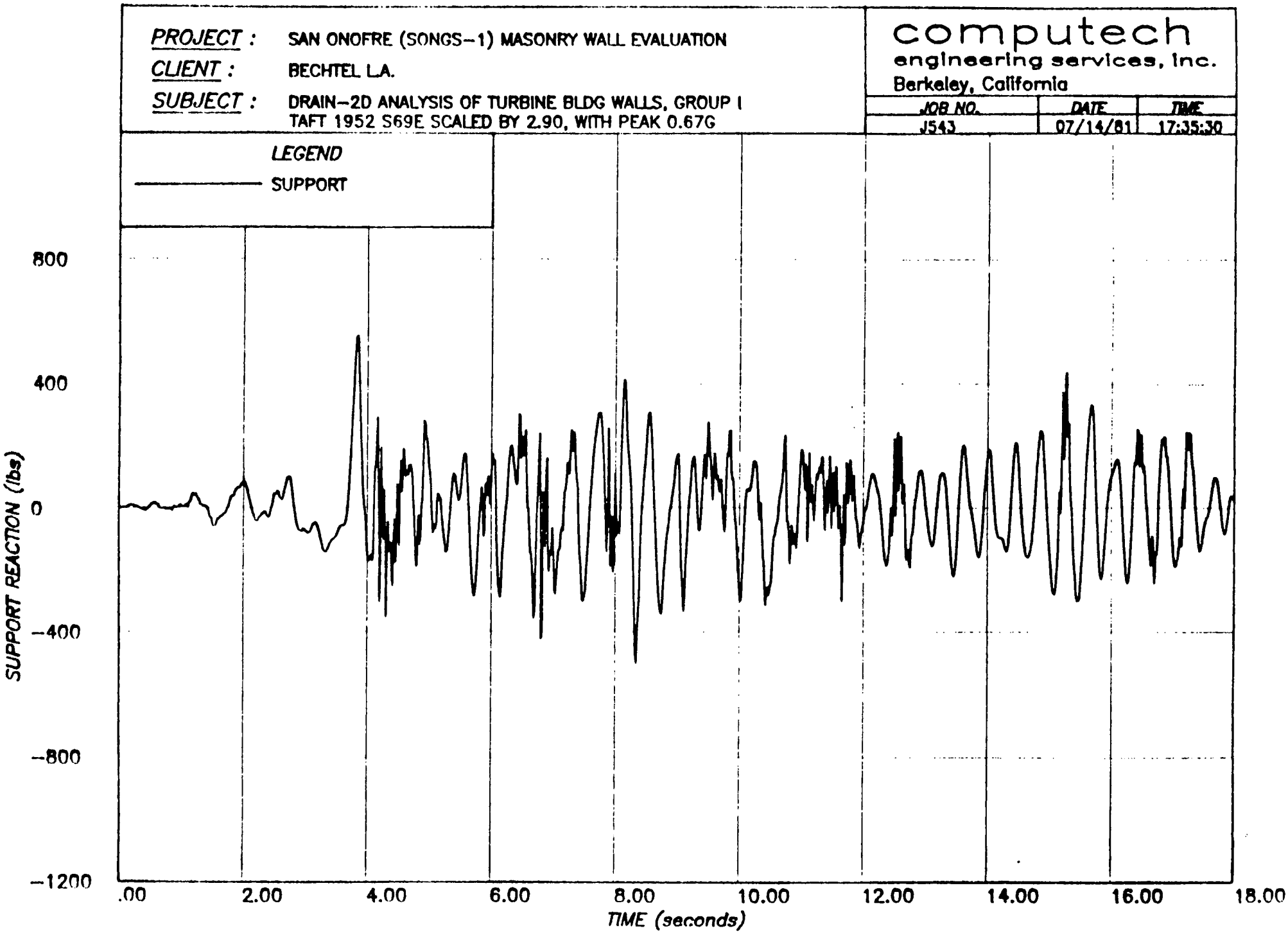


FIGURE 3.14 TURBINE BUILDING - GROUP I :TAFT

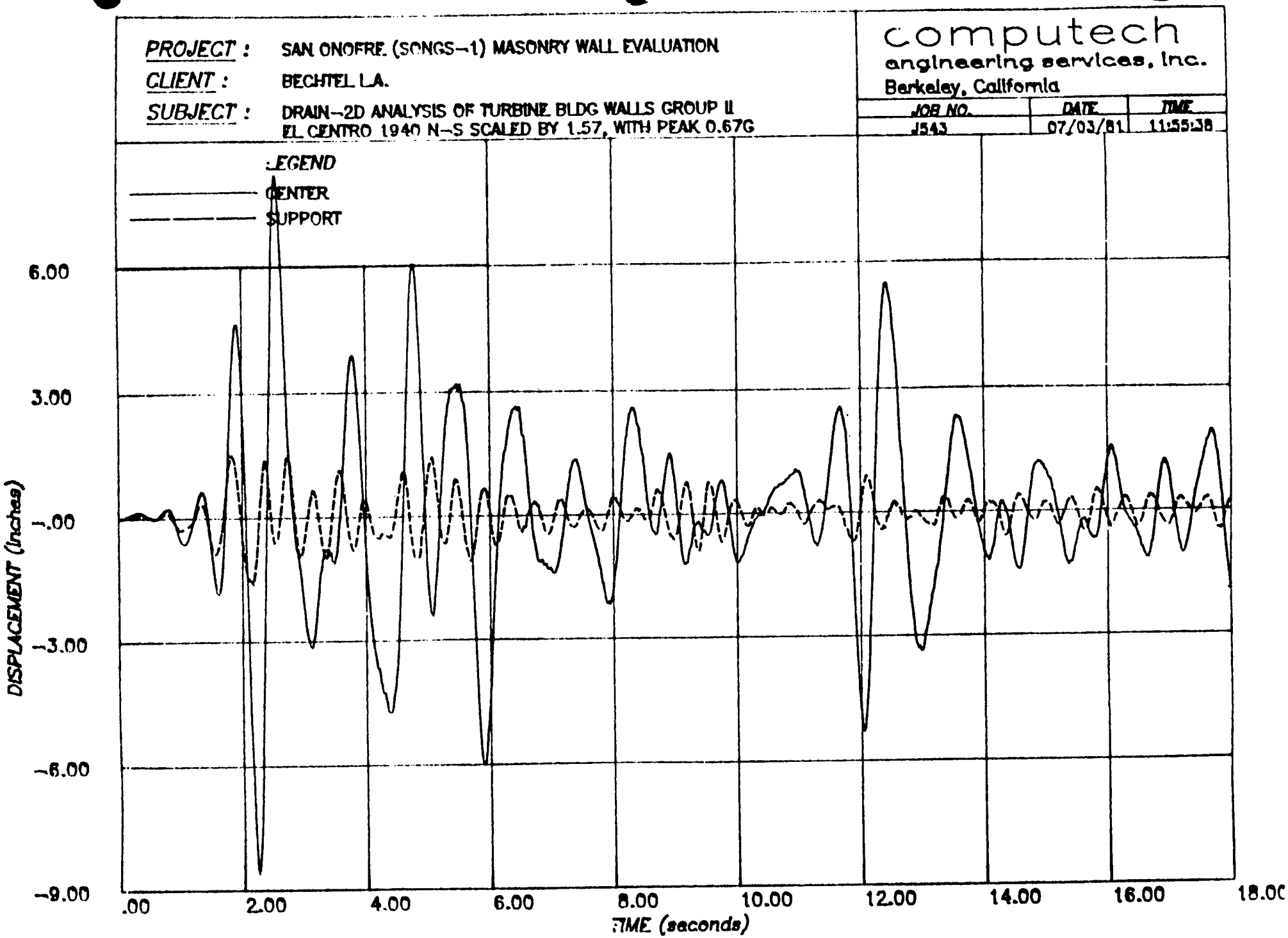


FIGURE 3.15 TURBINE BUILDING - GROUP II :EL CENTRO

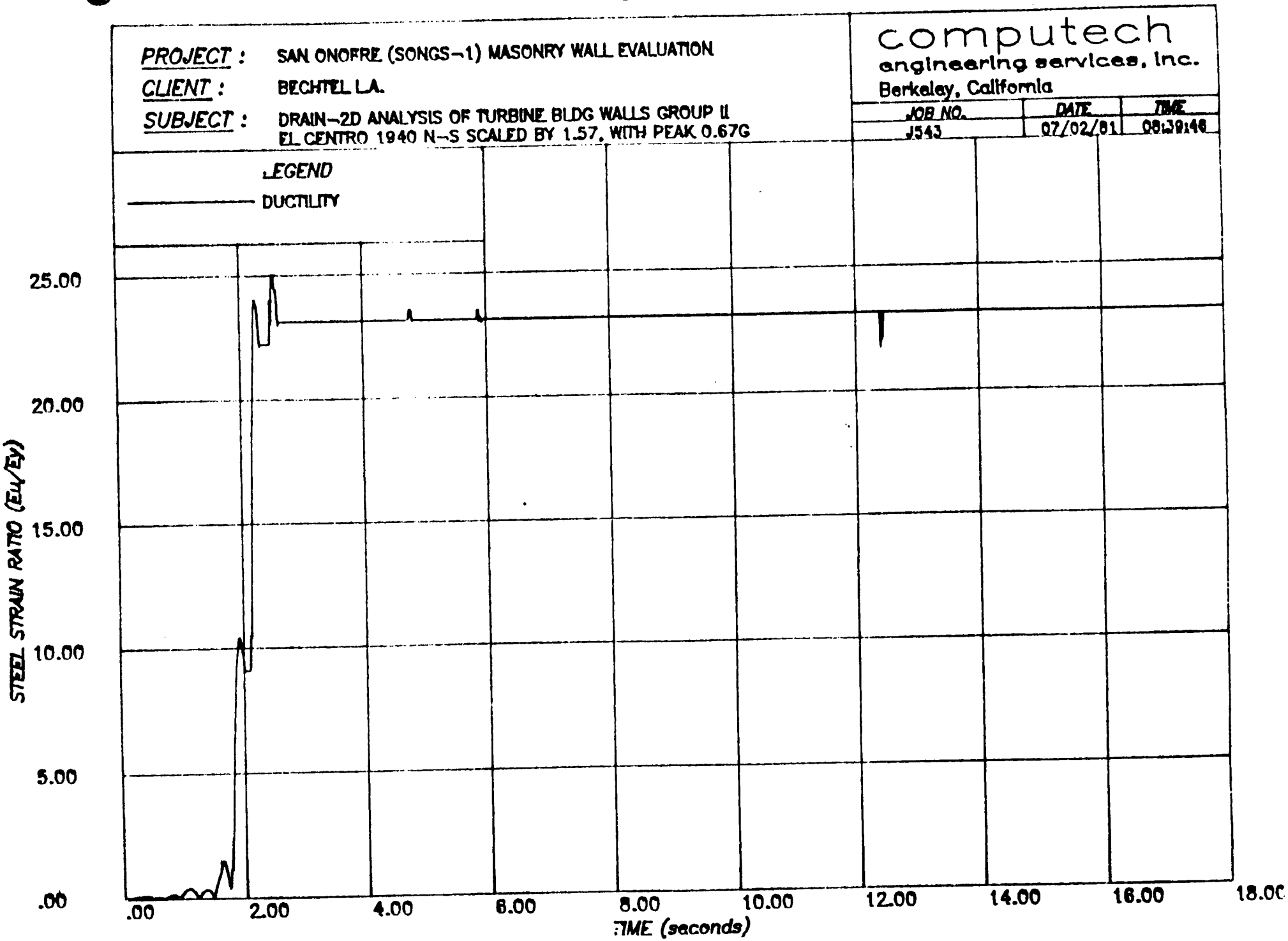


FIGURE 3.16 TURBINE BUILDING - GROUP II :EL CENTRO

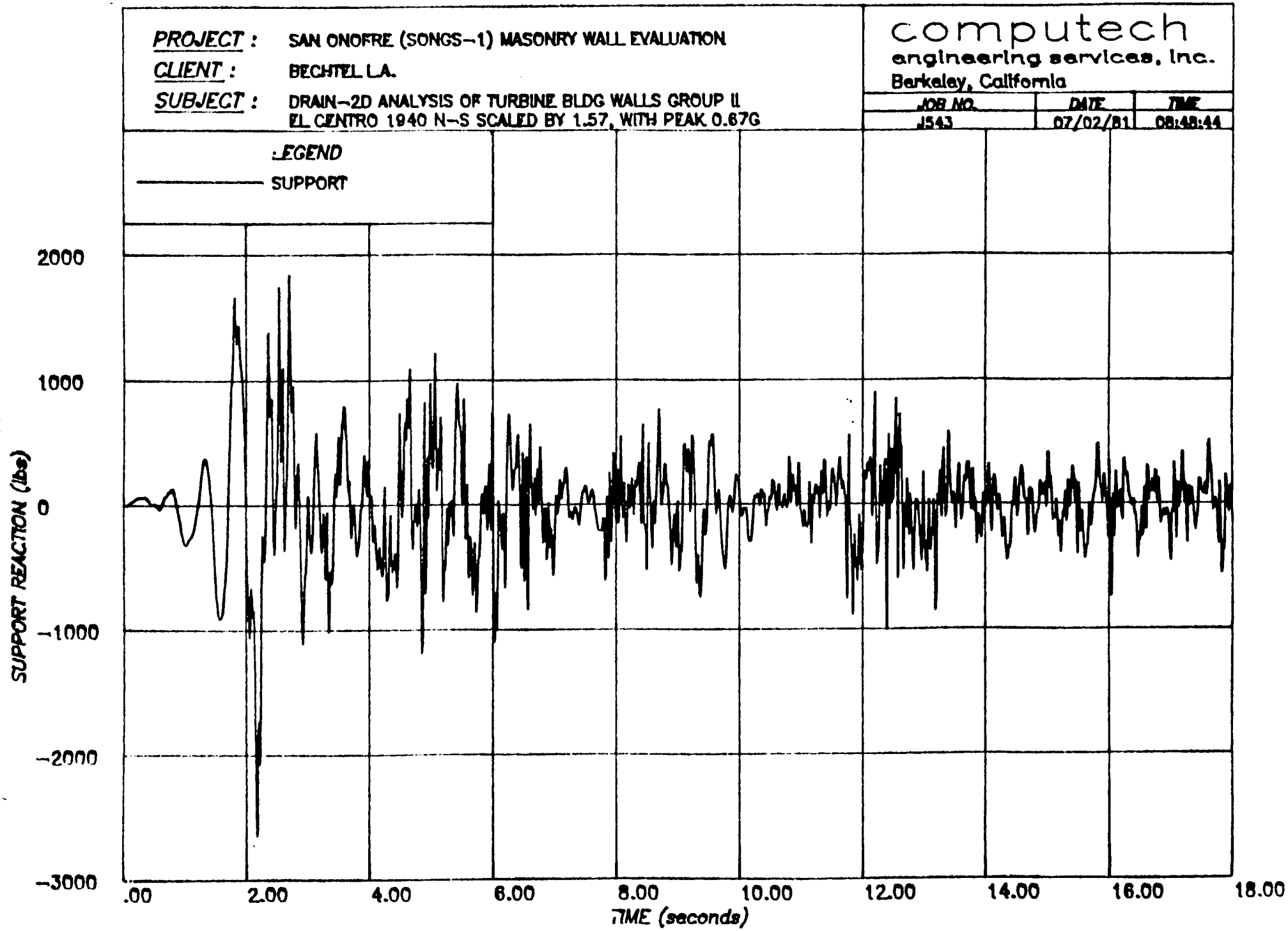


FIGURE 3.17 TURBINE BUILDING - GROUP II :EL CENTRO

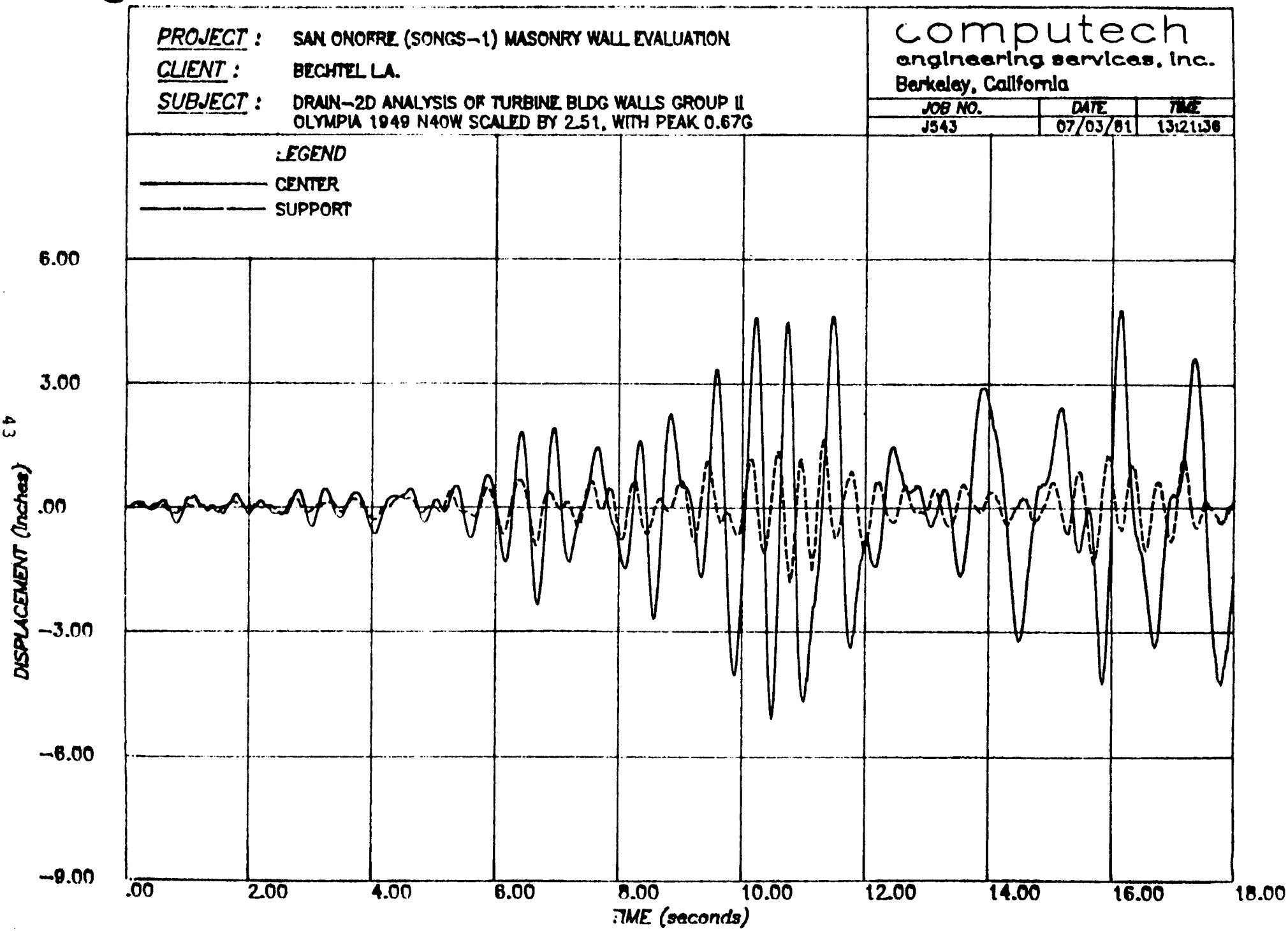


FIGURE 3.18 TURBINE BUILDING - GROUP II :OLYMPIA

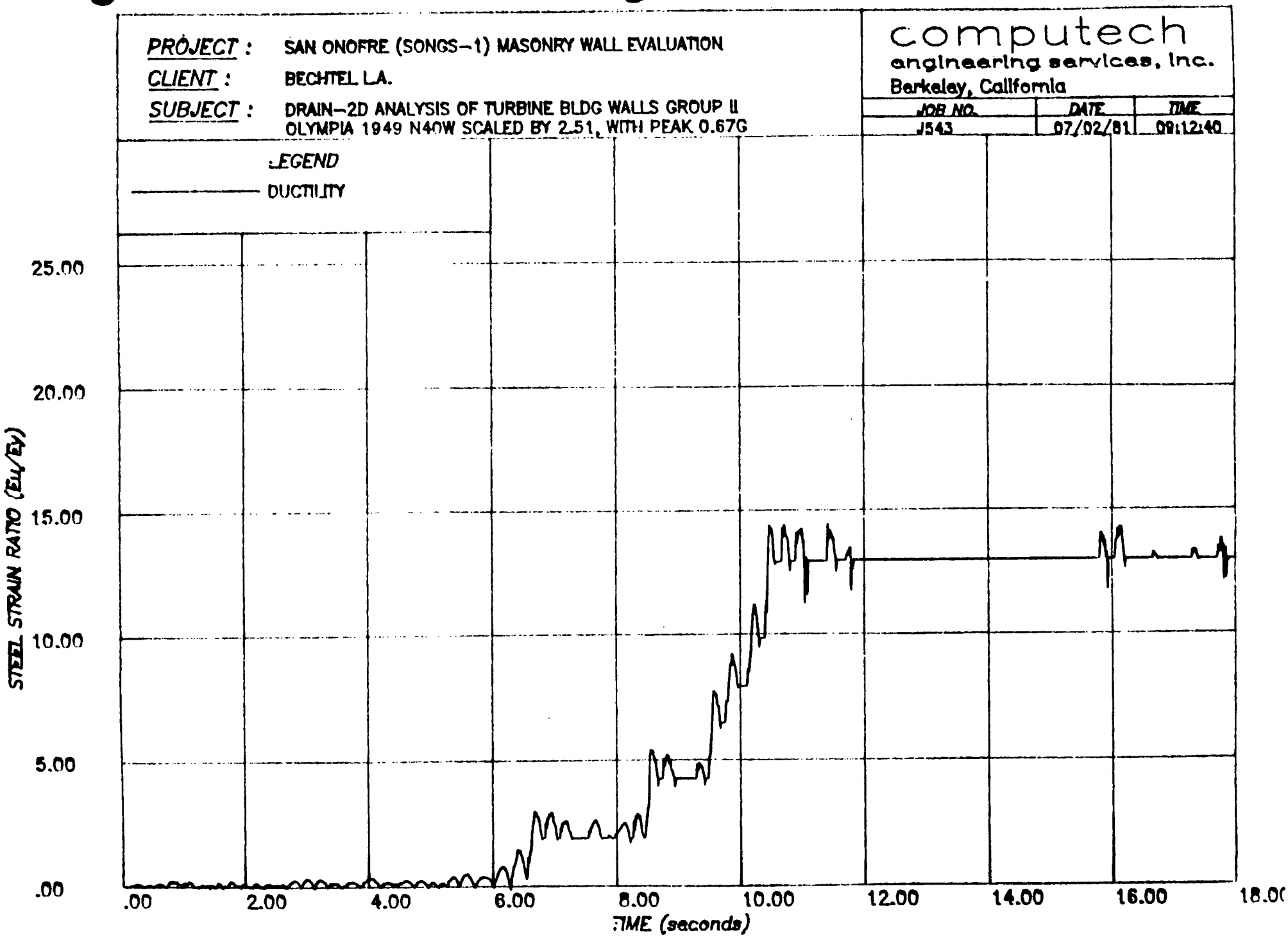


FIGURE 3.19 TURBINE BUILDING - GROUP II :OLYMPIA

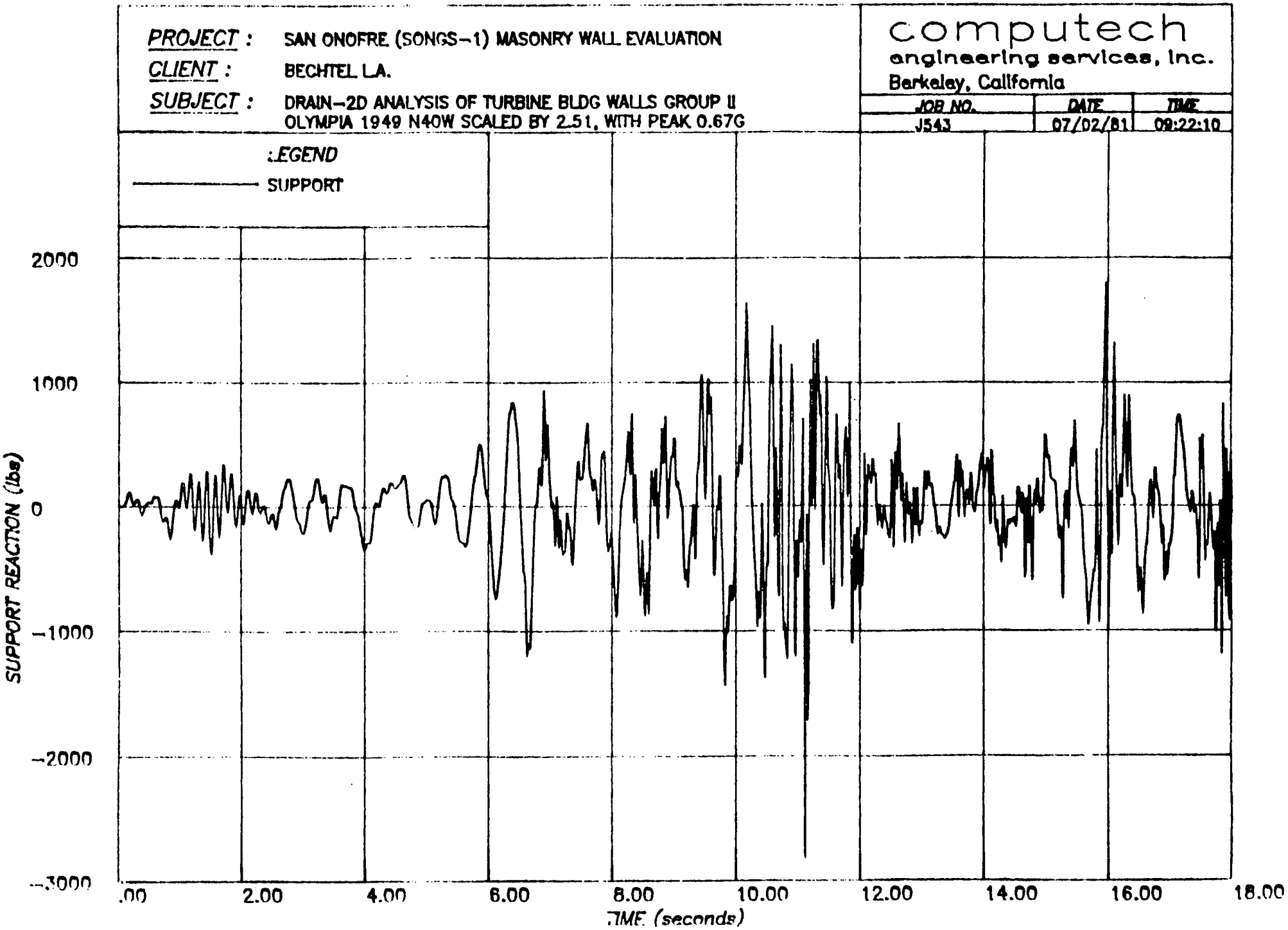


FIGURE 3.20 TURBINE BUILDING - GROUP II :OLYMPIA

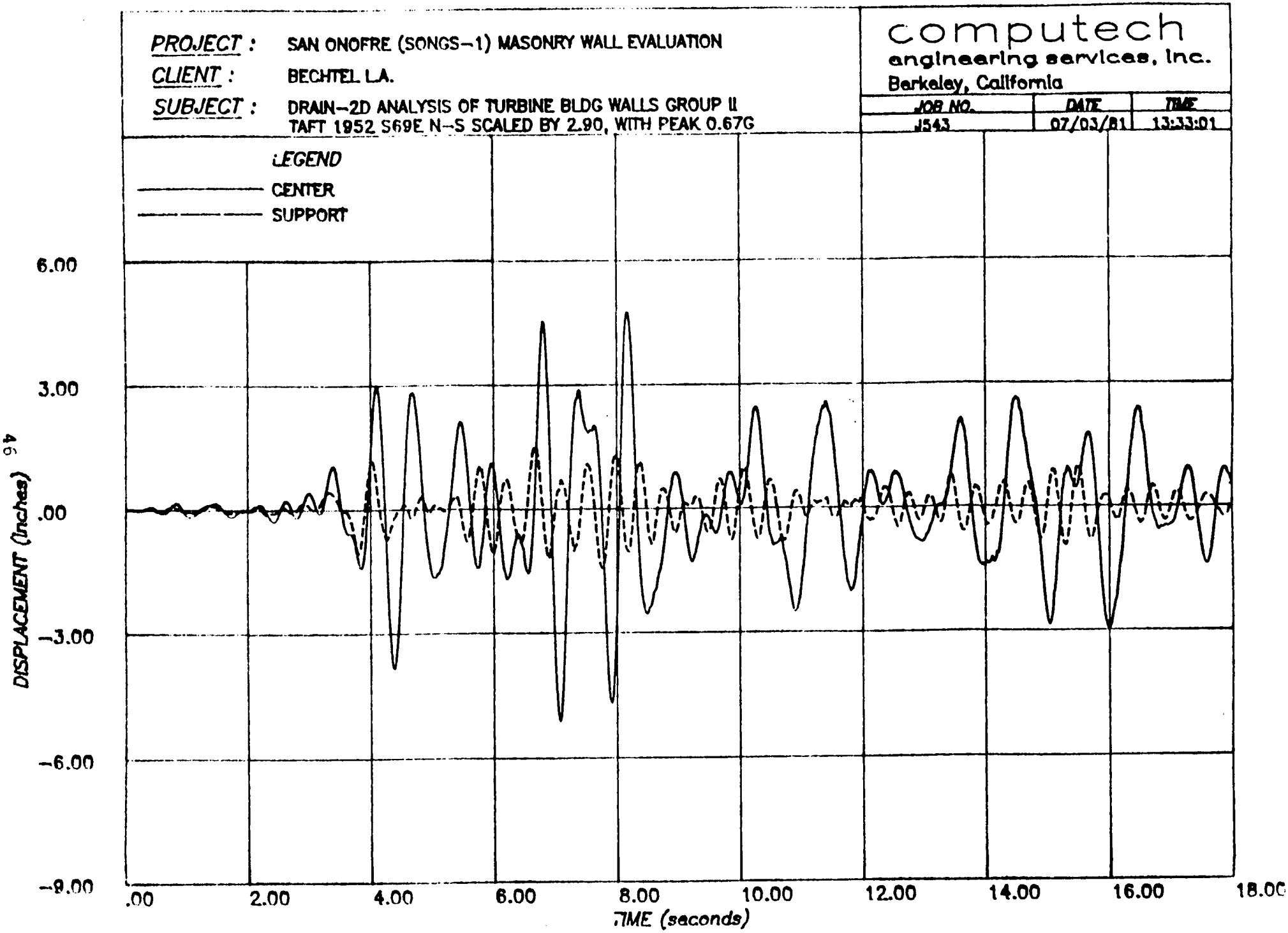


FIGURE 3.21 TURBINE BUILDING - GROUP II :TAFT

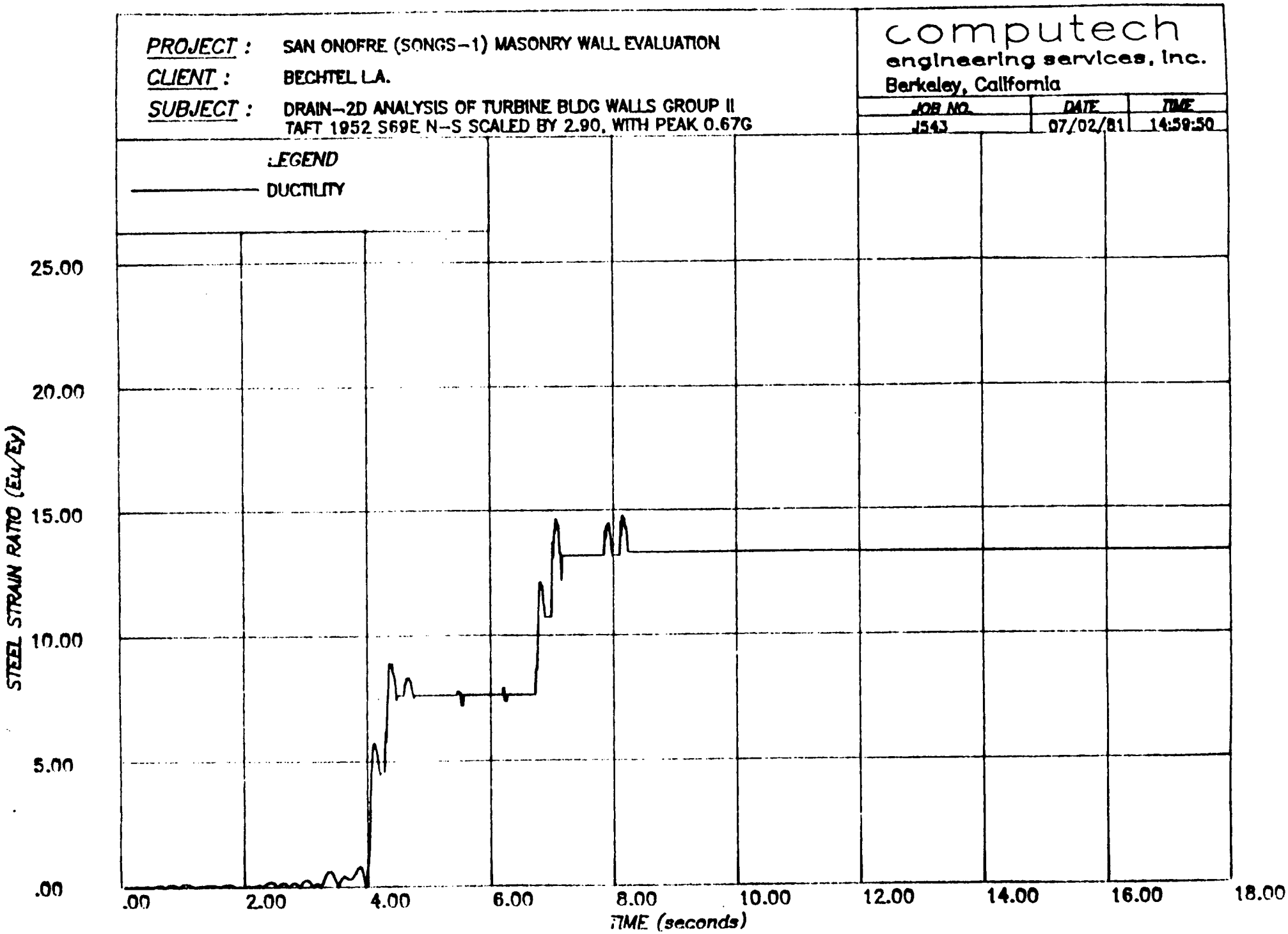


FIGURE 3.22 TURBINE BUILDING - GROUP II :TAFT

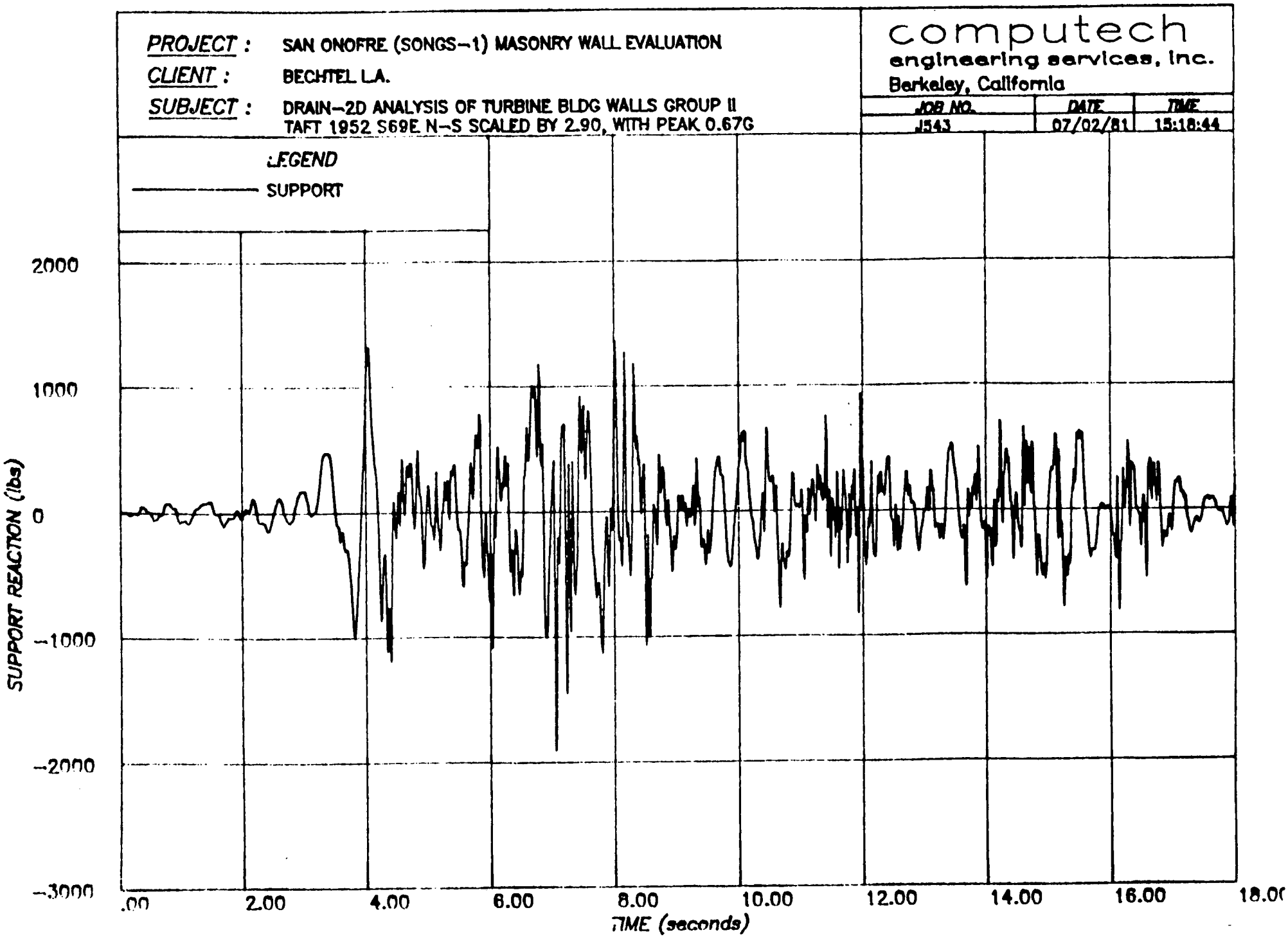


FIGURE 3.23 TURBINE BUILDING - GROUP II TAFT

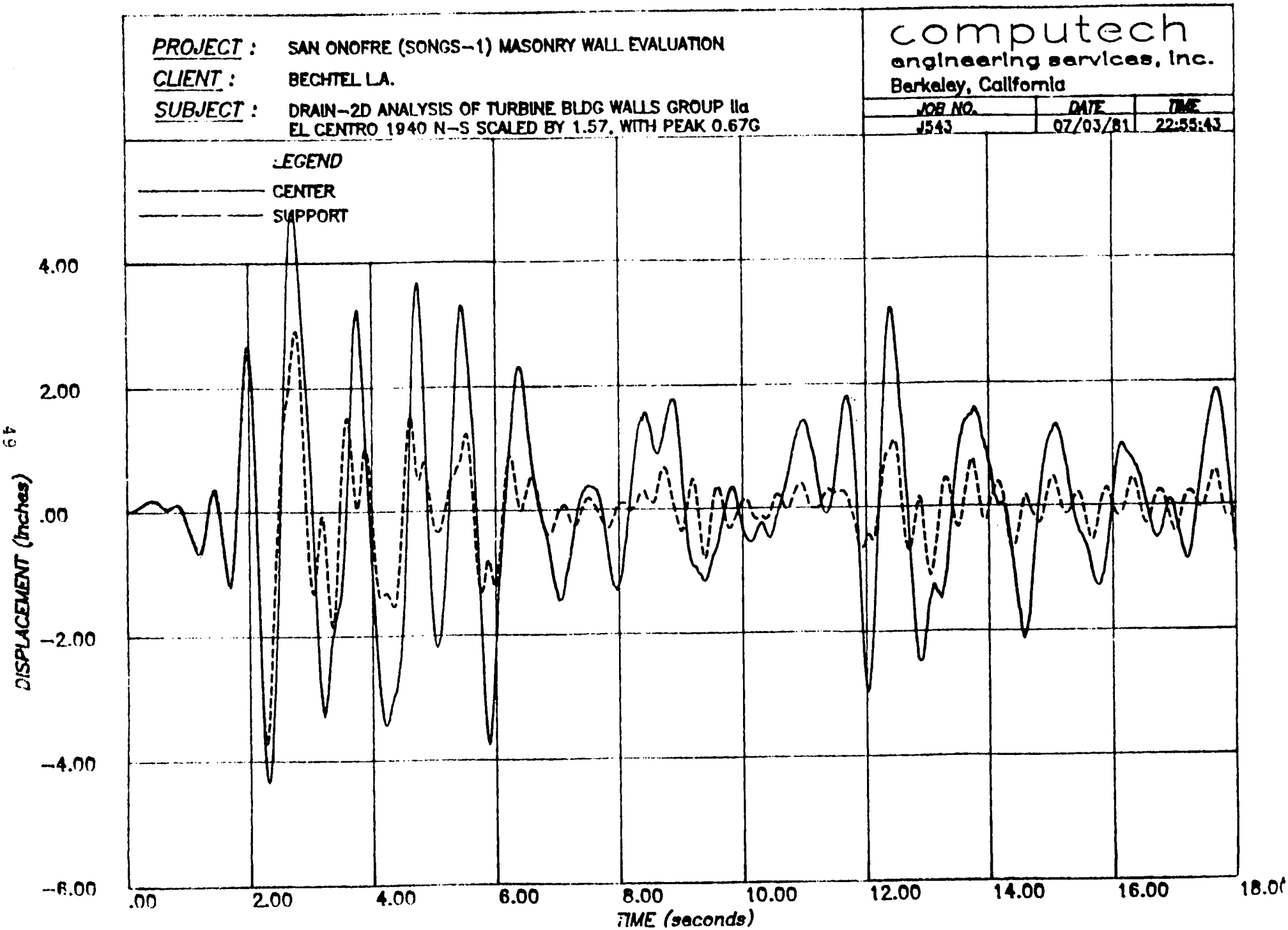


FIGURE 3.24 TURBINE BUILDING - GROUP IIa :EL CENTRO

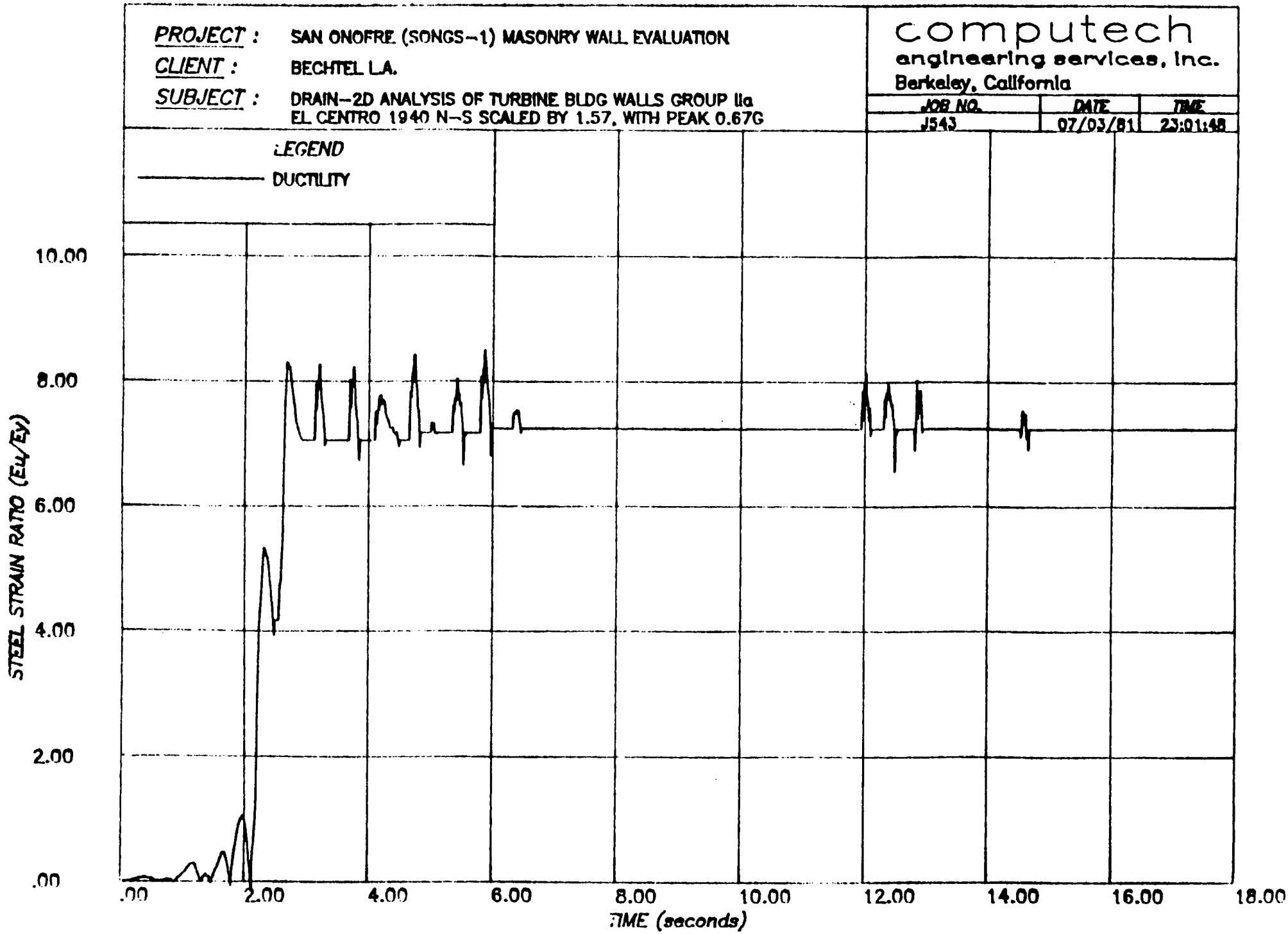


FIGURE 3.25 TURBINE BUILDING - GROUP IIa EL CENTRO

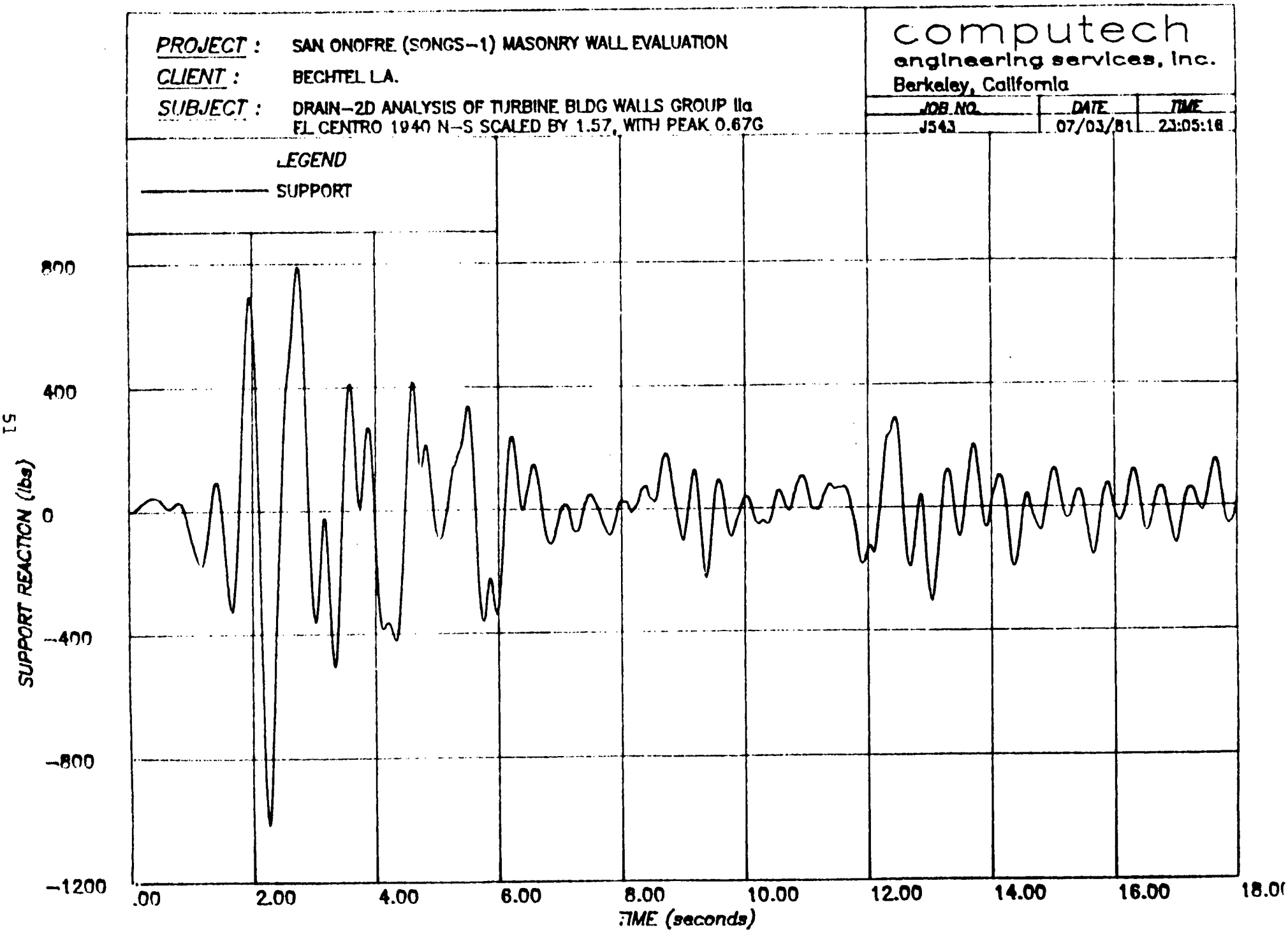


FIGURE 3.26 TURBINE BUILDING - GROUP IIa :EL CENTRO

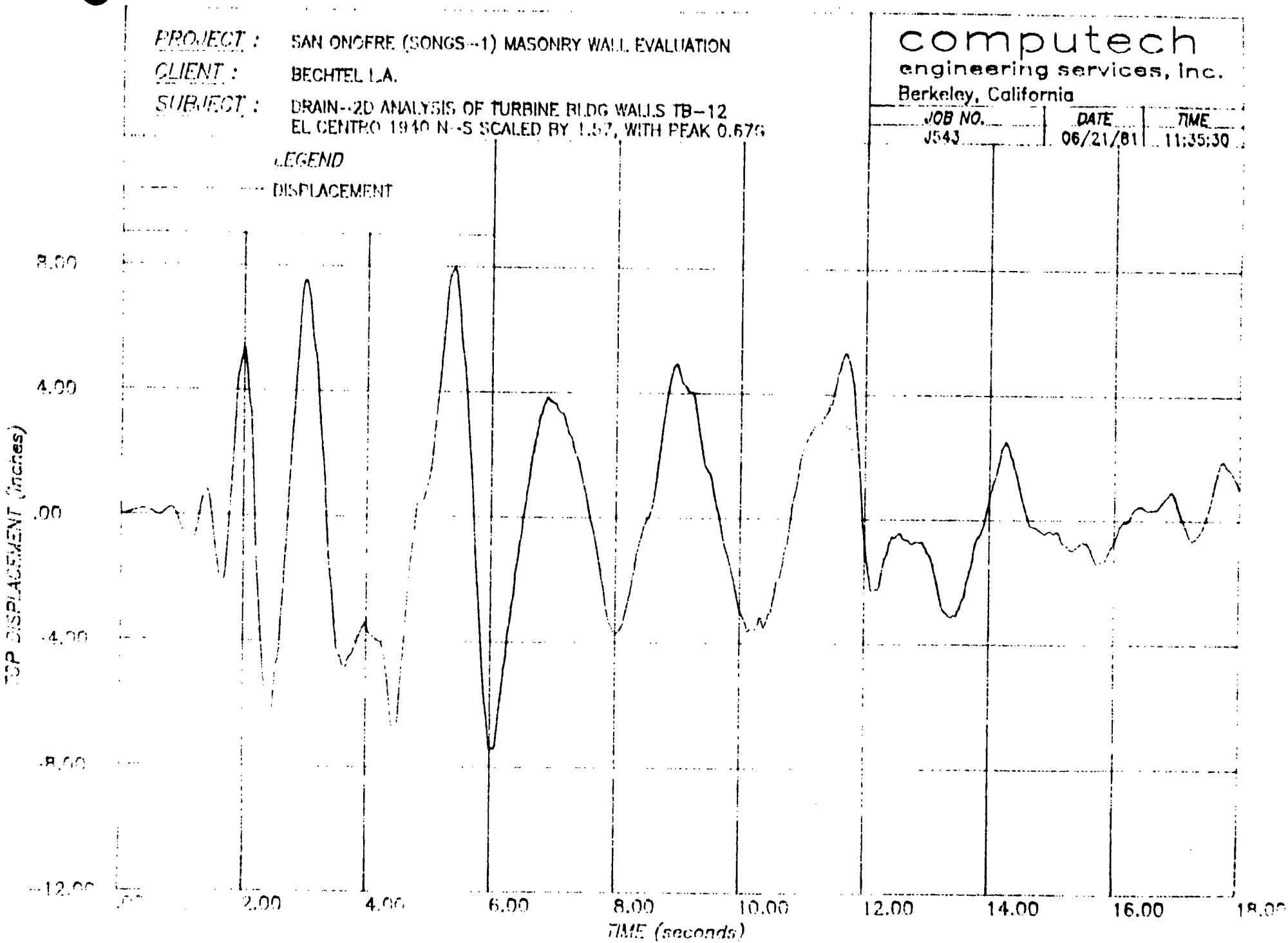


FIGURE 3.27 TURBINE BUILDING - GROUP III :EL CENTRO

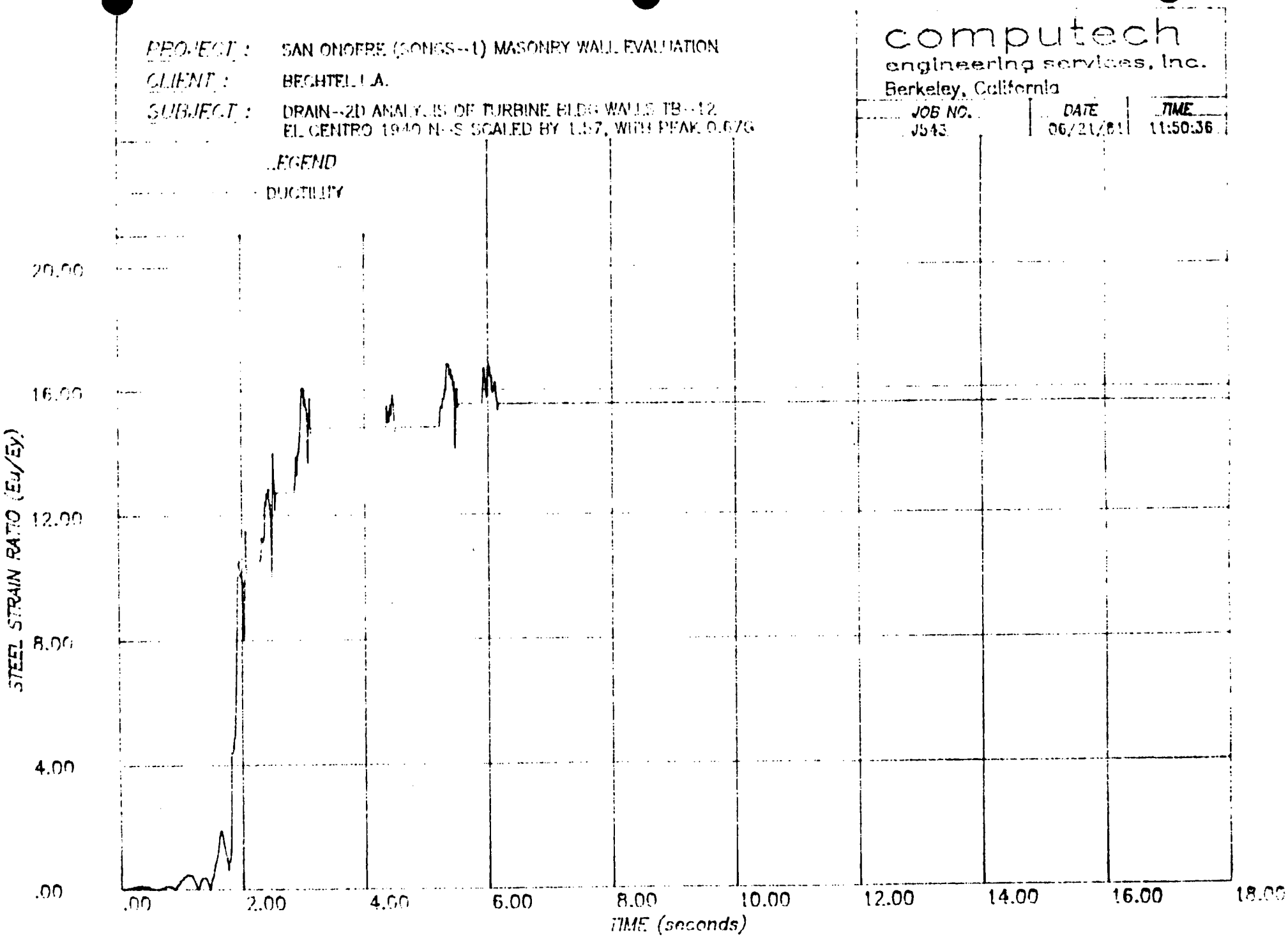


FIGURE 3.28 TURBINE BUILDING - GROUP III :EL CENTRO

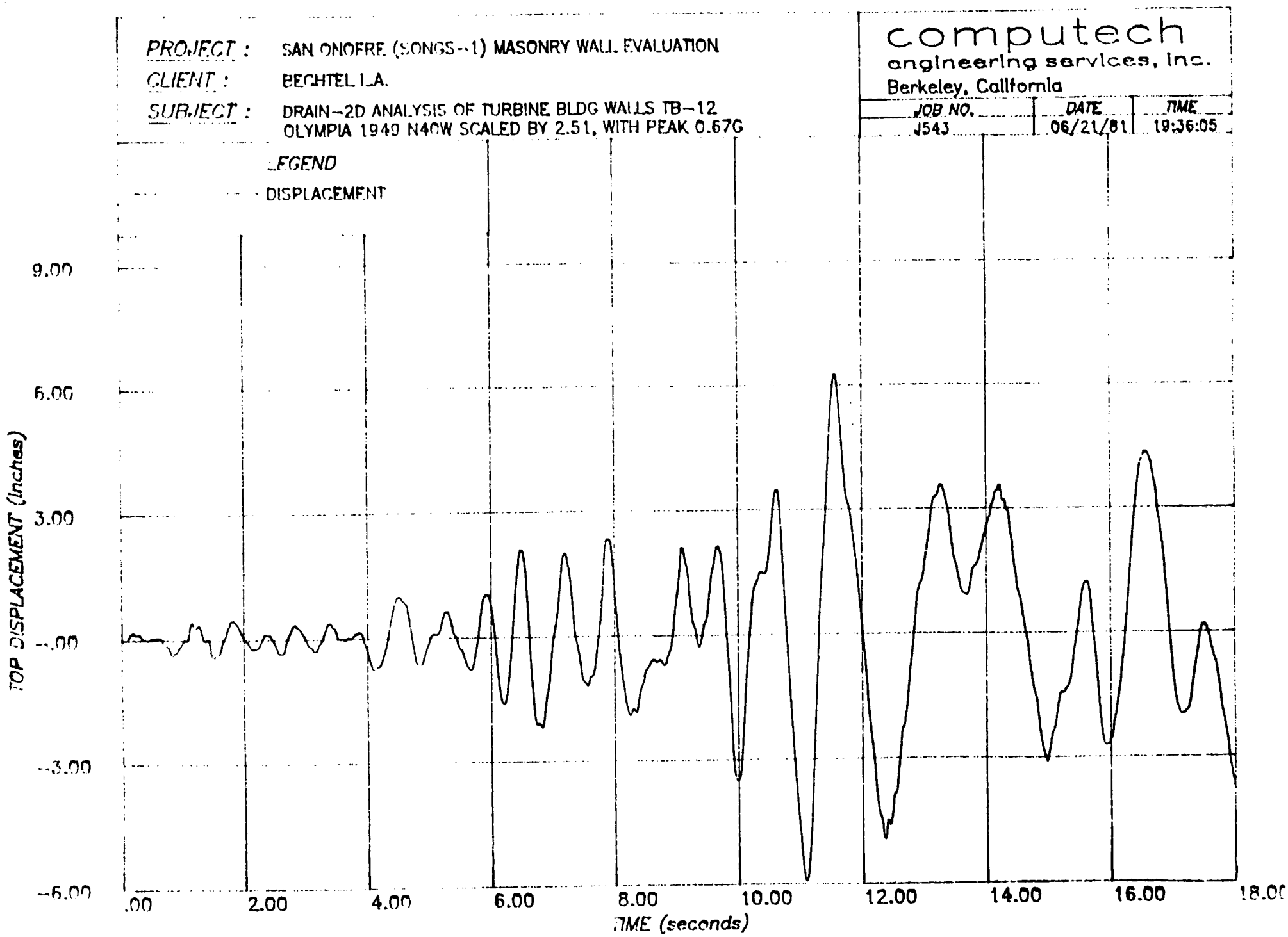


FIGURE 3.29 TURBINE BUILDING - GROUP III :OLYMPIA

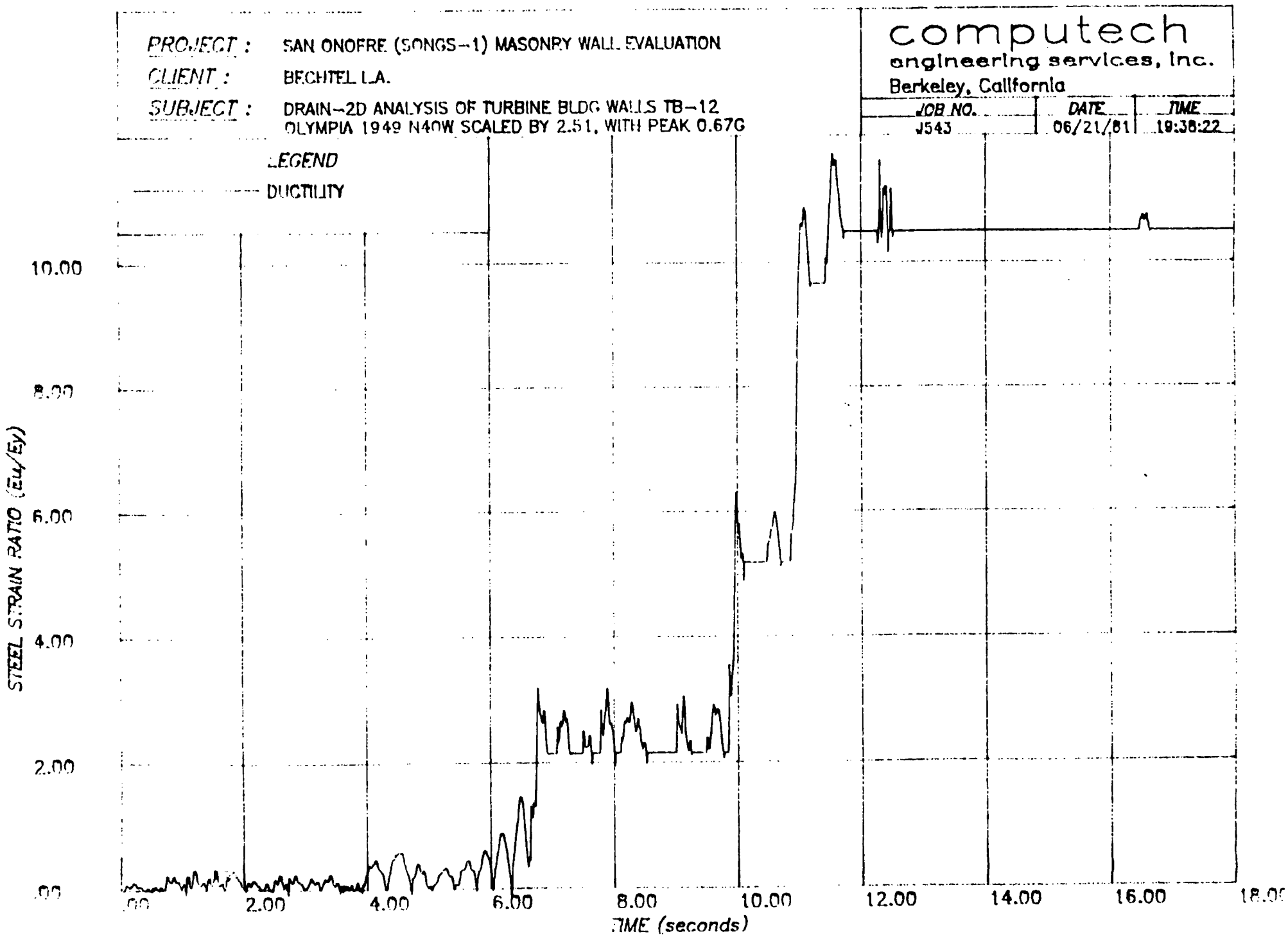


FIGURE 3.30 TURBINE BUILDING - GROUP III :OLYMPIA

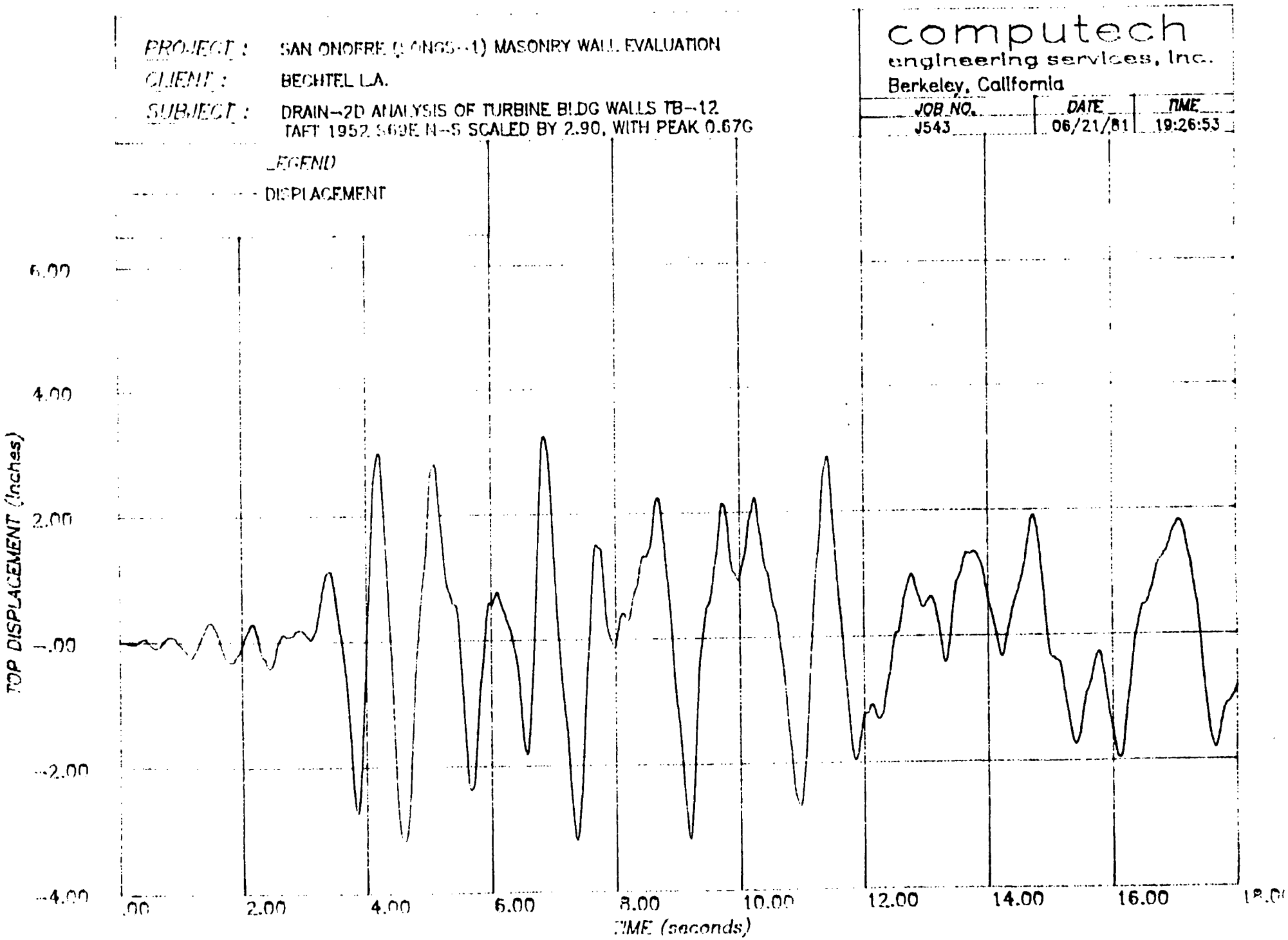


FIGURE 3.31 TURBINE BUILDING - GROUP III TAFT

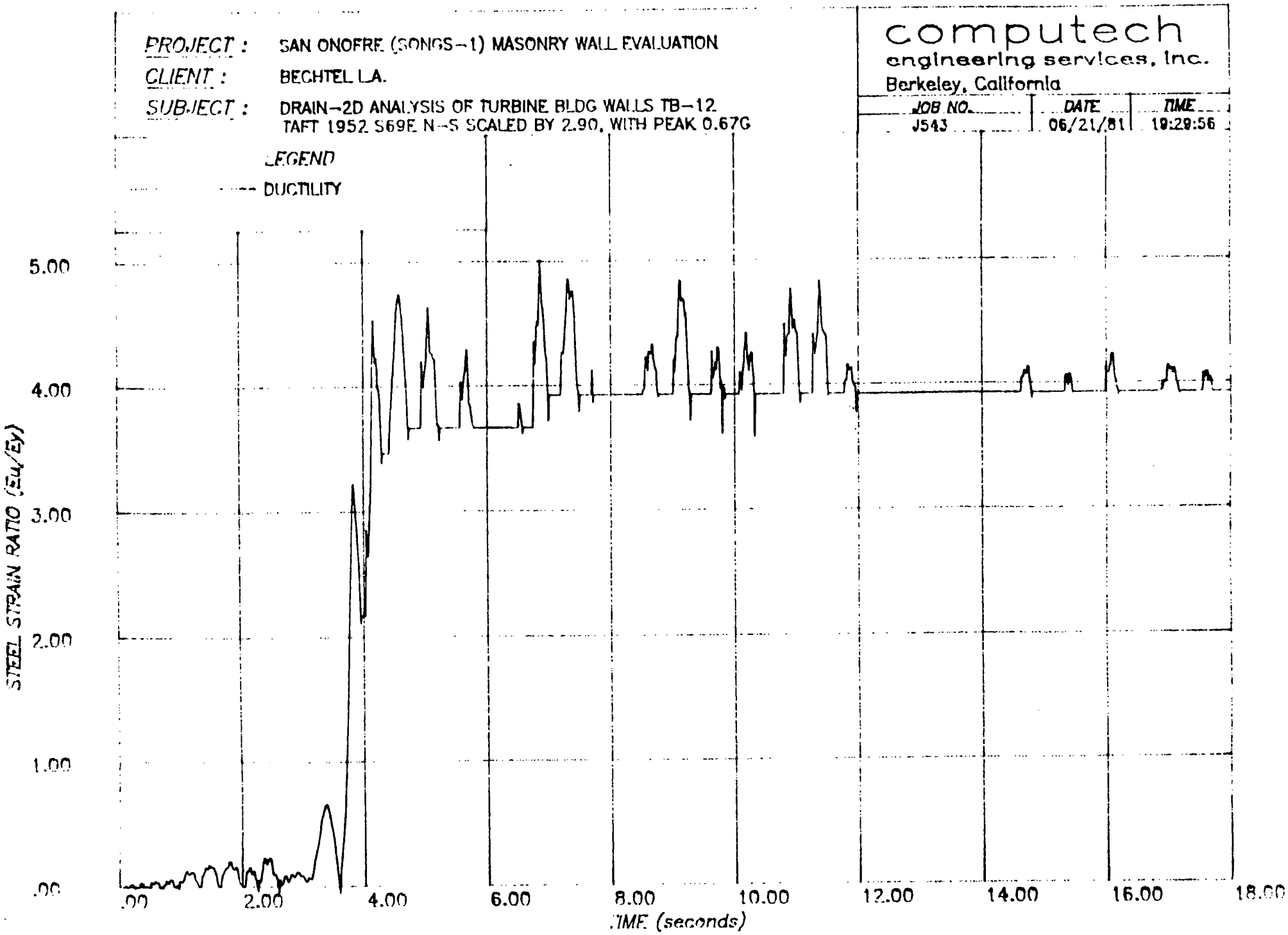
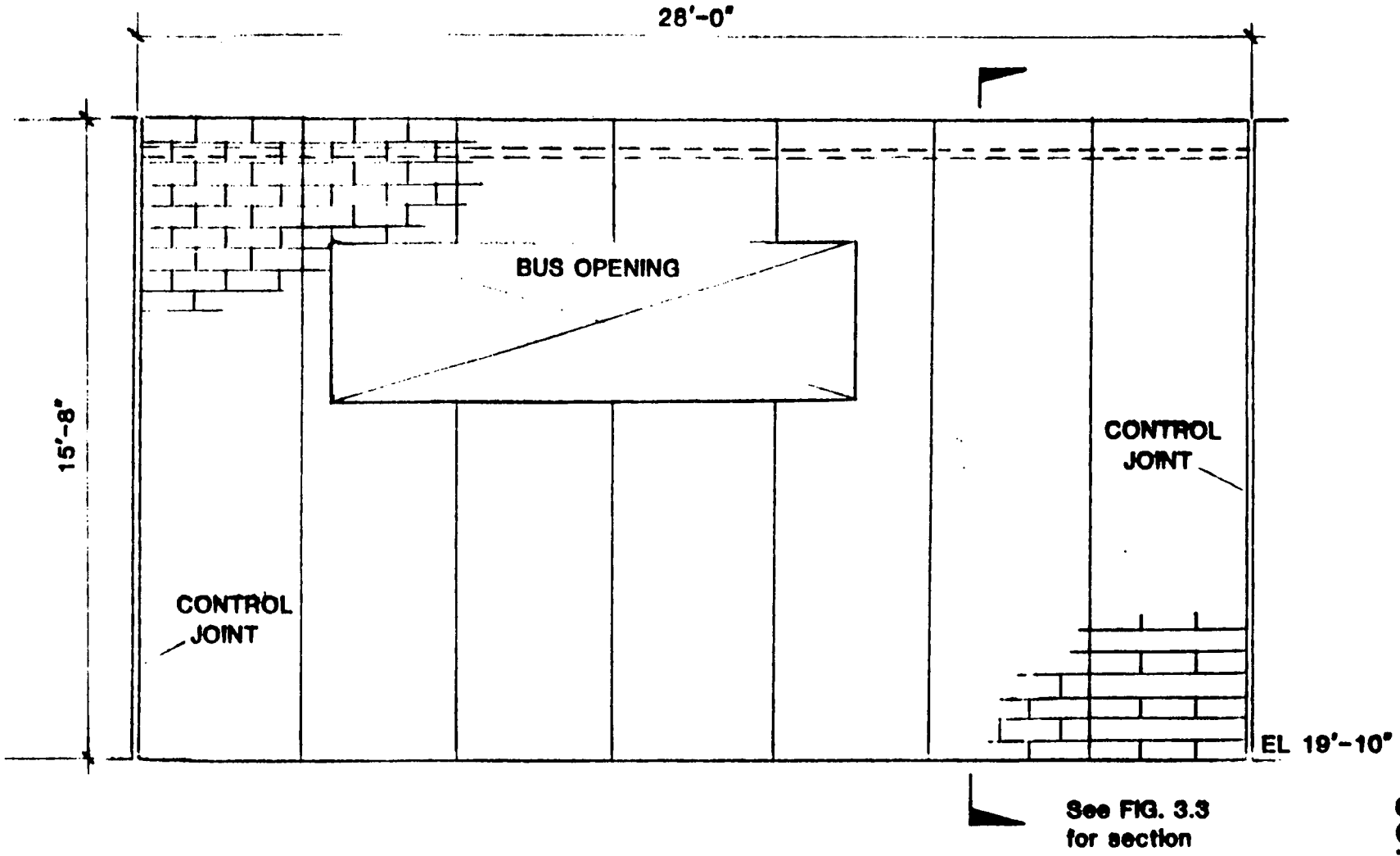


FIGURE 3.32 TURBINE BUILDING - GROUP III TAFT



WALL ELEVATION

COMPUTECH

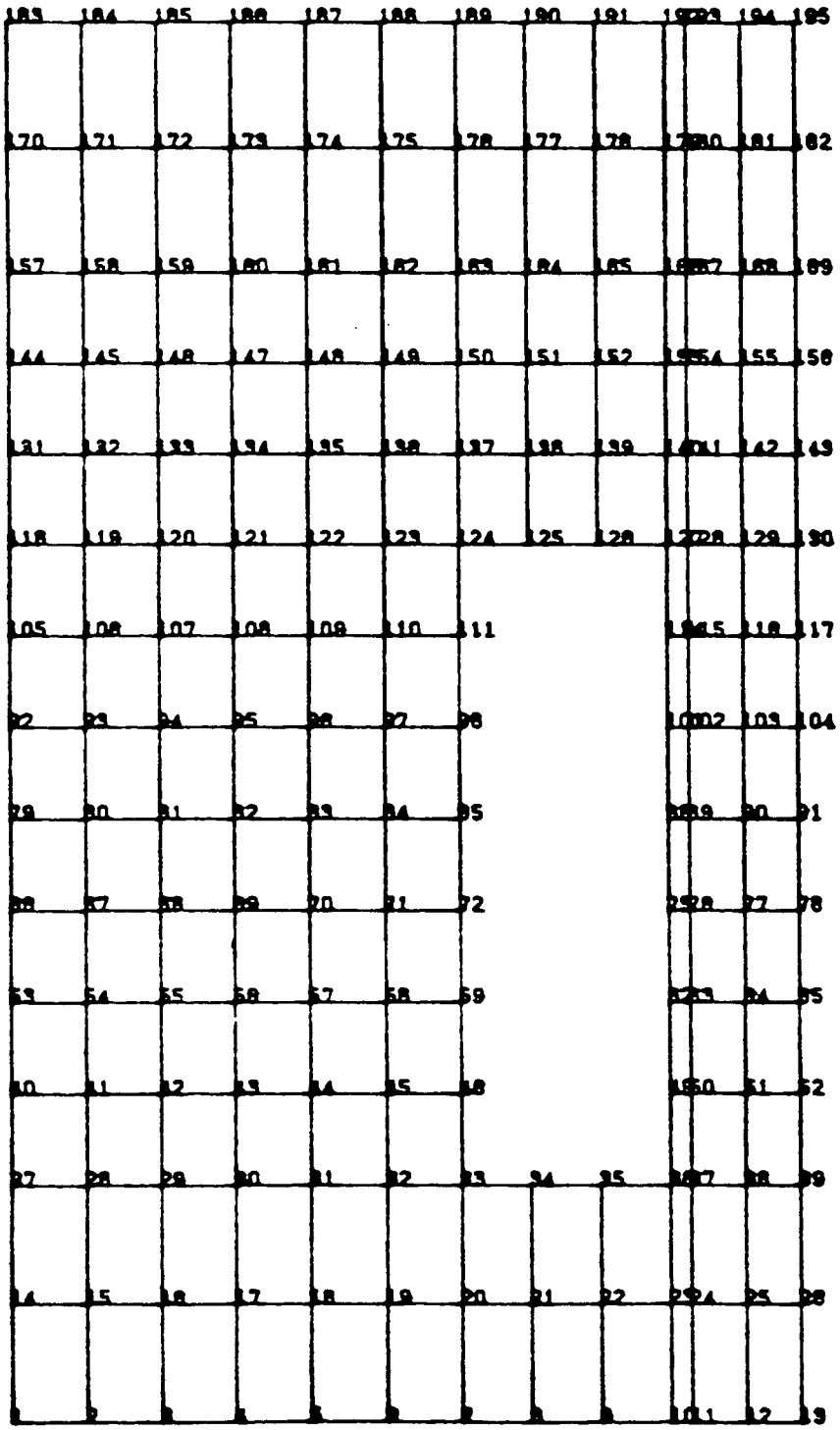
PROJECT NO 643
DRAWN
CHECKED

SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1

TURBINE BUILDING - WALL TB-1 ELEVATION

FIGURE NO 3.33

TURBINE BLDG., WALL TB-1
UNDEFORMED SHAPE

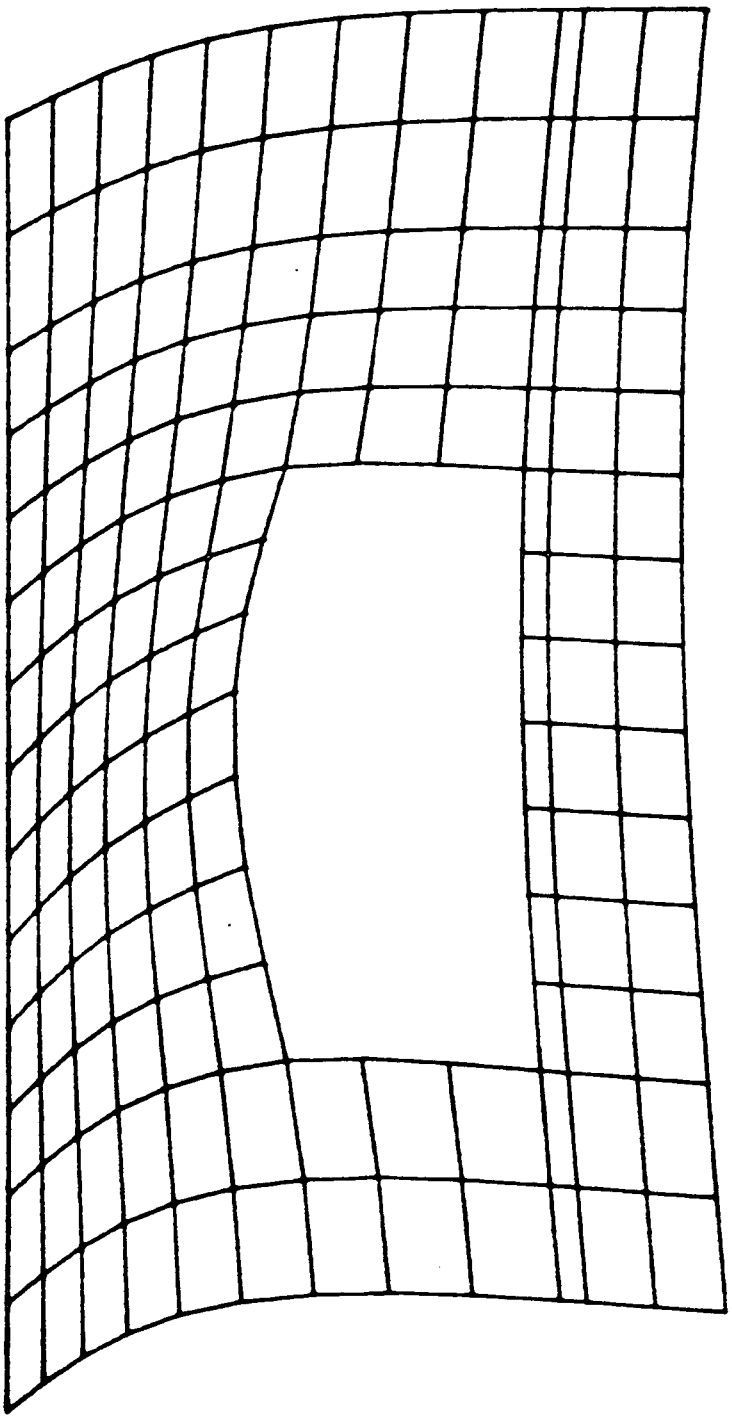


PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO	
DRAWN	TURBINE BUILDING - WALL TB-1 "SAP" MESH	3.34	
CHECKED			

COMPUTECH

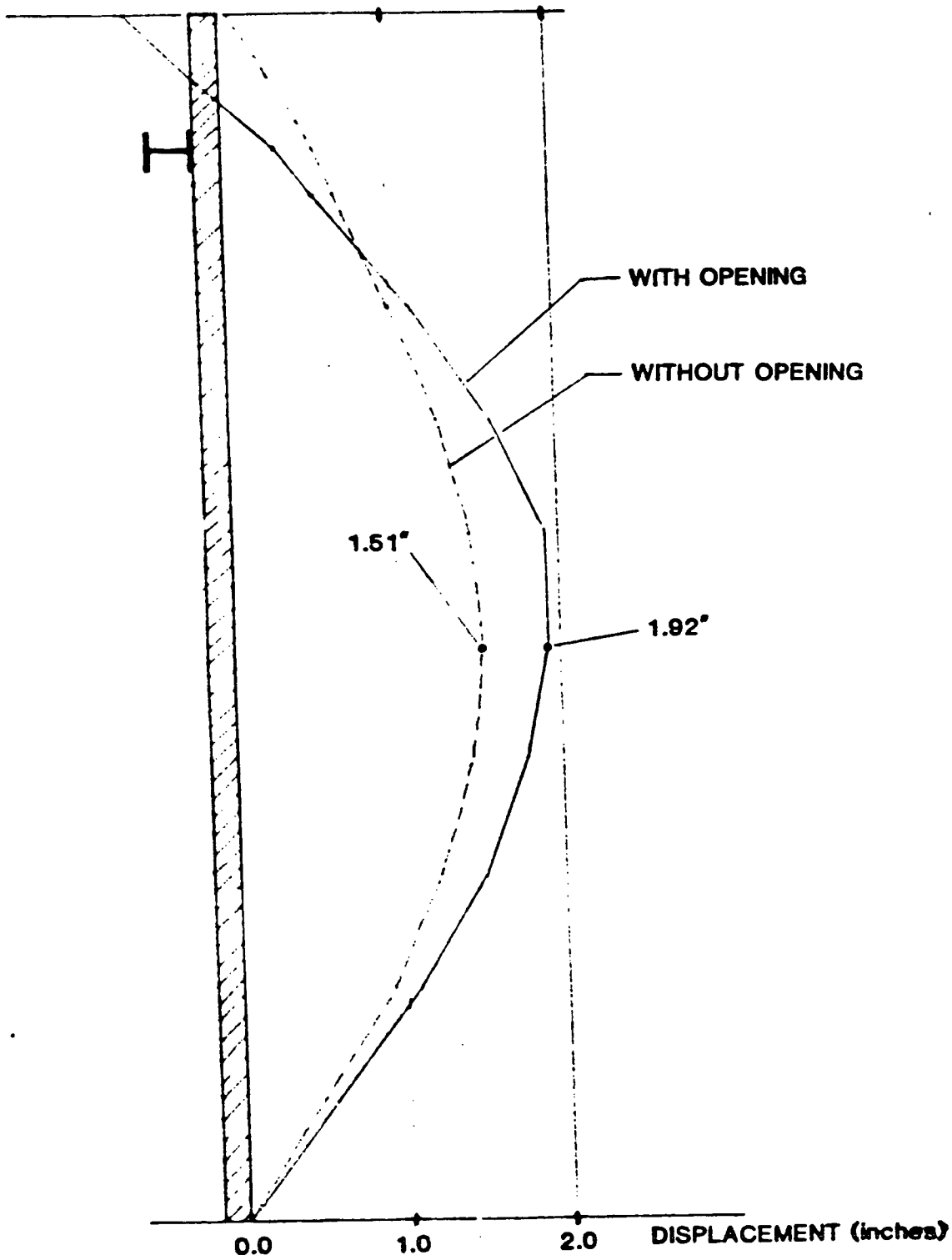
TURBINE BLDG., WALL TB-1

MODE 1 FREQ = 2.5862 Hz



PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
DRAWN	TURBINE BUILDING - WALL TB-1 MODE 1	3.35
CHECKED		

COMPUTECH



PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
DRAWN	TURBINE BUILDING - WALL TB-1 ELASTIC DEFLECTIONS	3.36
CHECKED		

4 VENTILATION EQUIPMENT BUILDING

4.1 Description of Walls

The Ventilation Equipment Building is a single story masonry building rectangular in plan with dimensions 44'-0" by 21'-4". The building is 20'-0" overall in height and has a roof formed of built up roofing and gravel. The four 8" masonry walls included in the evaluation of this building form the exterior walls of the building. A typical section through the walls is shown on Figure 4.1.

The walls are all centrally reinforced with #7 bars at 32" centers vertically and #5 bars at 48" centers horizontally. A 2'-0" bond beam reinforced with 2 #5 bars top and bottom exists at the top of all four walls. Because of the relatively small plan dimensions of the building, the presence of the bond beam and the wall to roof connections into the bond beam it was assumed that diaphragm action of the built up roofing was effective. Therefore the wall top support was considered rigid because of the high stiffness of the masonry acting as shear walls.

All walls carry some added weight due to equipment supports and this was included in the analyses as added lumped masses. The four walls contained openings for doors and the passage of equipment and ducts. Some of these openings were relatively large and their effect on the wall evaluation is discussed in Section 4.4.5.

It should be noted that for the Ventilation Equipment Building in addition to forming the exterior cladding the masonry walls form the lateral load resisting system. Therefore the stability of the building as a whole is dependent on the integrity of the masonry walls.

4.2 Inelastic Analysis

The four walls were modelled as a single DRAIN-2D model of 120 nodes. The model boundary conditions assumed simple supports at the top and bottom with five cracked joints modelled at mid height. Although the wall geometry, and thus the model, is symmetrical about mid height the added masses due to equipment loads are not evenly distributed about the axis of symmetry and therefore the whole wall height rather than one half was included in the model. A short very stiff truss member was added at the top of the wall to enable support reaction forces to be extracted from the analysis results.

To determine the most severe additional mass due to equipment loads all appendages on the wall spaced closer than 4'-0" to one of the corners were disregarded. This was based on the assumption that the strips of masonry wall immediately adjacent to a wall at right angles would distribute the loads horizontally to the adjacent wall rather than by vertical spanning as assumed in the DRAIN-2D model. With these exclusions the total added

weight was 400 lbs. or about one third the weight of the masonry wall itself. This is a much lower proportion than was encountered in the Turbine Building.

As for the other buildings the wall model was analyzed for the effects of the three scaled earthquake components selected for the San Onofre. Unit 1 wall evaluation. A summary of the results obtained is given in the following section.

4.3 Results of Analysis

Table 4.1 gives a summary of the maximum displacements, steel strain ratio, support reactions and masonry stress for each of the three time histories used in the analysis. In Figures 4.2 through 4.10 time histories for these response parameters are plotted for the El Centro, Olympia and Taft scaled time histories.

From the steel strain ratios in Table 4.1 it is apparent that significant yield occurred only under the El Centro time history. For Olympia only very slight hinging occurred and for the Taft analysis the wall remained elastic. The displacements were relatively small reaching a maximum of 5.21 inches for El Centro and 3.28 inches and 2.24 inches for the Olympia and Taft time histories respectively. The magnitude of the support reaction varied with the earthquake time history in the same manner as the displacements and steel strains with El Centro producing the greatest response. The masonry compressive stress was approximately equal for the El Centro and Olympia records where yield occurred and placed an upper limit on the masonry stress and was much less for the Taft record where response was elastic.

r4.4 Evaluation of Results

In the following sections each of the relevant response parameters are evaluated in terms of the criteria given in Volume 1 of this report.

4.4.1 Reinforcement Ductility

The maximum steel ductility, as measured by the strain ratio, was 3.65, 1.16 and 0.71 for the El Centro, Olympia and Taft records respectively. All these values are substantially below the criteria limit of 45.

4.4.2 Masonry Compressive Stress

Masonry compressive stresses were 552 psi, 548 psi and 390 psi for the El Centro, Olympia and Taft records respectively. For the former two values the total masonry force was limited once yield of the reinforcing occurred. The maximum value of 552 psi is less than the limit of 0.85 f'm set in the criteria which for San Onofre, Unit 1 gives a value of 1147 psi.

The vertical reinforcing in this building is relatively heavy compared with the other buildings because the walls function as lateral load resisting shear walls under in-plane loads. For this reason the yield force in the steel and the consequent equilibrating masonry force is relatively large.

4.4.3 Wall Supports

The maximum support reactions are given in Table 4.1 which shows that the El Centro record produced the most severe loading condition with a value of 824 lbs. The added mass due to equipment loads was relatively low with a total of 400 lbs at the most critical section, or about one third the weight of the bare wall. However this mass was concentrated in the top three feet of the wall and therefore would have maximum effect on the numerical value of the top reaction.

4.4.4 Wall Stability

The overall wall stability as shown by the displacement time histories in Figures 4.2, 4.5 and 4.8 for the El Centro, Olympia and Taft time histories respectively is shown to be satisfactory. The wall oscillates about the original undeflected baseline and no permanent set is apparent.

As the maximum deflections did not exceed the thickness of the masonry wall during any of the time history analyses the static check for secondary moment effects was not required

4.4.5 Wall Openings

Each of the four Ventilation Equipment Building masonry walls has openings. Walls VB-3 and VB-4 have relatively small openings and the trimmer bars provide an area of reinforcing greater than the area curtailed because of the opening. In walls VB-1 and VB-2 the openings are more substantial and because of the relatively high reinforcement in the walls and the importance of the walls as lateral load resisting elements an elastic plate analysis of these two walls was performed. Elevations for walls VB-1 and VB-2 are shown in Figures 4.11 and 4.12 respectively

In the following sections the plate analyses of these two walls are discussed.

4.5 Elastic Analysis of Walls VB-1 and VB-2

Each of the two walls was analyzed using the SAP computer program and as a basis for comparison a wall of the same overall dimensions and added equipment weights but without any openings was also analyzed. This latter

analysis provided a means of checking the influence of the openings on the elastic response and allowed engineering judgement to be exercised as to whether the inelastic response of walls with openings would be significantly different than the assumed inelastic model with no openings.

4.5.1 SAP Model

A similar procedure as was used for the elastic analyses in the Turbine Building was followed for VB-1 and VB-2. The SAP model used orthotropic plate elements with properties using 1.5EIcr based on the actual wall reinforcement.

The wall models were simply supported on all four sides, by adjacent walls on the vertical boundaries and by the roof diaphragm at the top boundary. Added masses as given on the masonry drawings were included in the model. The mesh layout is given in Figure 4.13 for wall VB-1 and in Figure 4.15 for wall VB-2. The model without openings used a similar mesh layout to both of these examples.

4.5.2 Method of Analysis

As for the other walls a response spectrum analysis using the 7% damped Housner spectrum was performed. The modal responses were combined for the first 5 modes using the square root of the sum of the squares procedure.

4.5.3 Results of Analysis

Figures 4.14 and 4.16 respectively show the first mode shapes for walls VB-1 and VB-2. In Table 4.2 a number of response paramters are listed for each analysis.

The frequencies were similar for each of the two models with openings and for the wall without openings. Elastic displacements were greatest for the wall with the greatest number of openings, VB-1, where the maximum value was about 27% higher than for the case with no openings. In general these higher displacements were due to the additional deflection from horizontal bending along an opening boundary.

The computed yield capacities of each of these walls were 860 lb.in/in for horizontal spanning and 2352 lb.in/in for vertical spanning. The horizontal value was exceeded only for wall VB-2 and in this analysis only one element had a moment greater than 860 lb.in/in, the element between the two wall openings. In this area there are a total of 4 trimmer bars and therefore the capacity is over twice the value calculated for the typical horizontal reinforcement of #5 at 48". Therefore in no wall would yield occur due to horizontal spanning.

The elastic vertical moments in all analyses exceeded the yield capacity.

This is expected as the inelastic analysis on the wall section without openings revealed yielding under two of the three earthquake components analyzed. For wall VB-1 a total of 6 elements had vertical moments greater than the maximum value of 2606 lb.in/in from the analysis of the wall without openings. However all 6 of these elements were immediately adjacent to openings where the wall strength is augmented by trimmer bars. Therefore it can be concluded that the effect of openings on wall VB-1 is to increase moment values locally but only in areas where trimmer bars provide additional strength, and so the wall inelastic response would be comparable to that of the wall without openings.

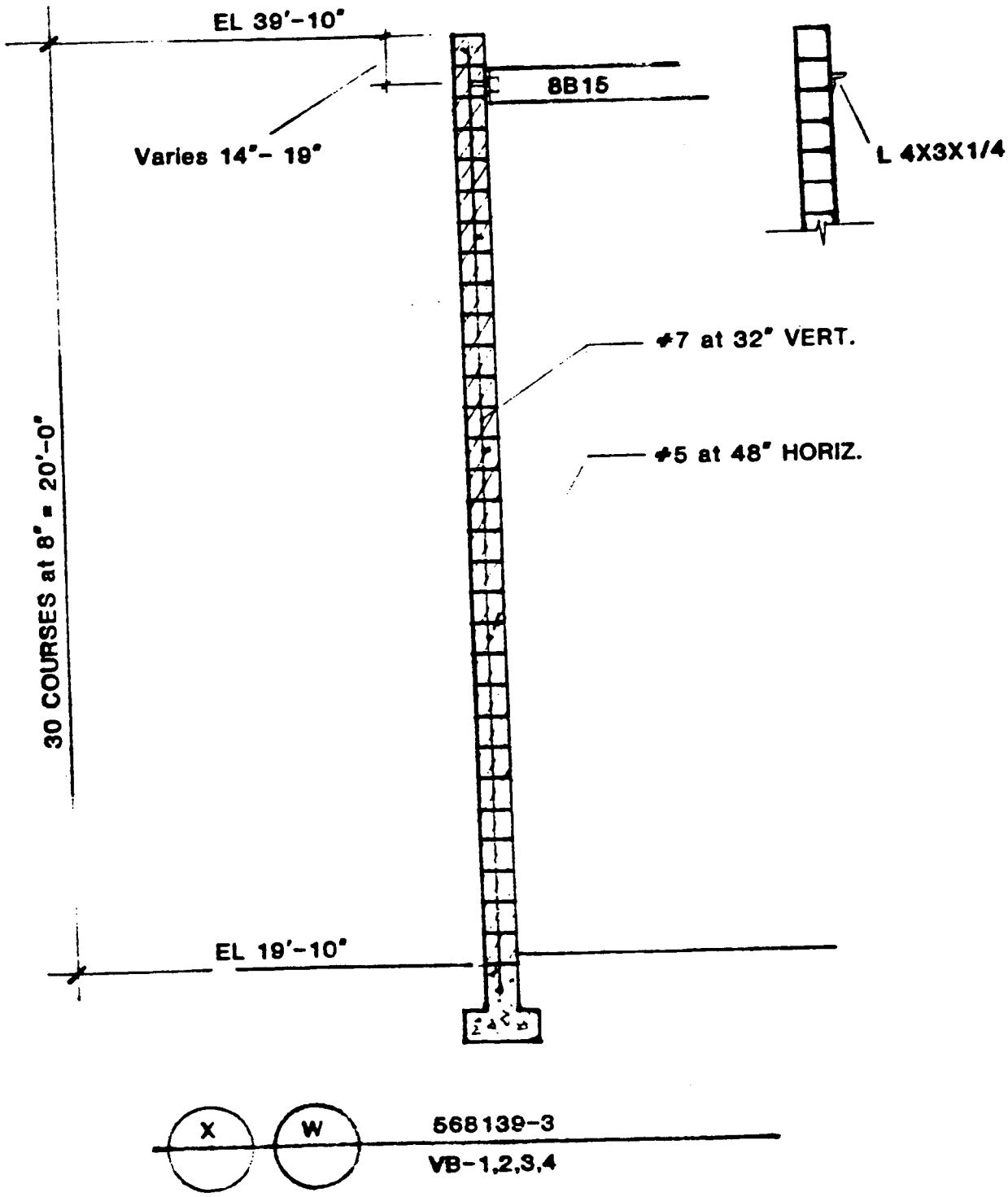
Wall VB-2 had a total of 14 elements with elastic vertical moments greater than the maximum in the wall without openings. Two of these were immediately adjacent to the openings and of the remainder the maximum value was 2959 lb.in/in, 13.5% higher than for the equivalent wall without openings. As the inelastic analysis revealed relatively low deflections and steel ductilities, as shown in Table 4.1, it is considered that the effect of the openings in wall VB-2 would not be such as to raise inelastic response to undesirable levels. On this basis both walls VB-1 and VB-2 are considered satisfactory.

	EARTHQUAKE RECORD		
	EL CENTRO 1940	TAFT 1952	OLYMPIA 1949
DISPLACEMENTS (inches)			
Mid-Span Maximum	4.24	2.16	3.28
Mid-Span Minimum	-5.21	-2.24	-2.98
STEEL STRAIN RATIO (Eu/Ey)	3.65	0.71	1.16
SUPPORT REACTION (lbs)	824	-479	-605
MASONRY COMPRESSIVE STRESS (psi)	552	390	548

TABLE 4.1 : SUMMARY OF RESULTS — VENTILATION BUILDING

	WITH OPENINGS		WITHOUT OPENINGS
	VB-1	VB-2	
FREQUENCIES (hz)			
Mode 1	2.47	2.43	2.59
Mode 2	3.65	3.63	3.73
Mode 3	5.16	5.11	5.46
Mode 4	7.58	6.81	7.74
Mode 5	8.25	8.44	9.25
MAXIMUM DISPLACEMENT (inches)	2.64	2.39	2.07
MAXIMUM MOMENTS (lb-in/in)			
Vertical Spanning	3631	3328	2606
Horizontal Spanning	618	998	426

TABLE 4.2 : ELASTIC ANALYSIS OF WALLS VB-1 AND VB-2



PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
DRAWN	VENTILATION BUILDING : TYPICAL SECTION	4.1
CHECKED		

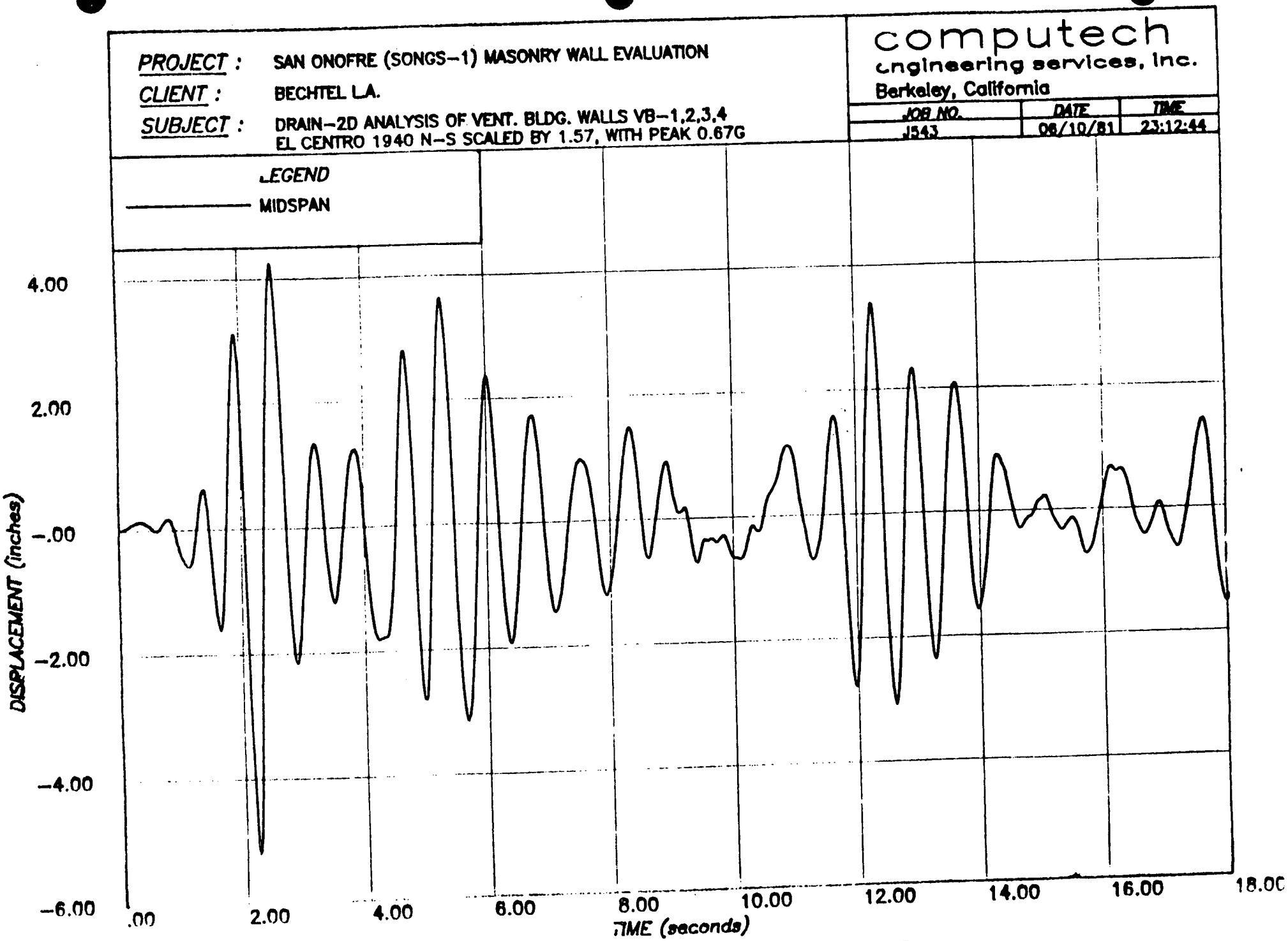


FIGURE 4.2 VENTILATION BUILDING : EL CENTRO

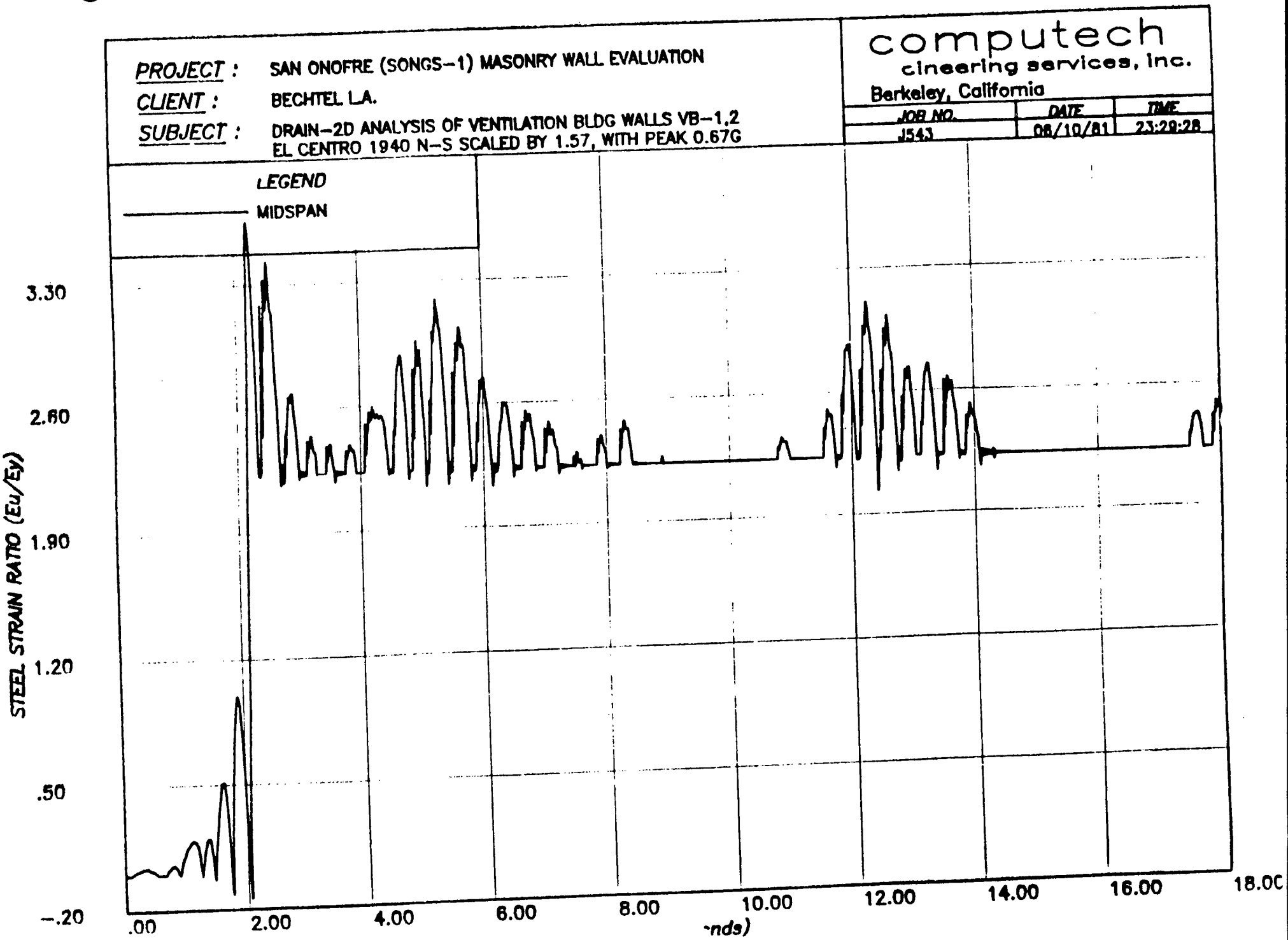


FIGURE 4.3 VENTILATION BUILDING : EL CENTRO

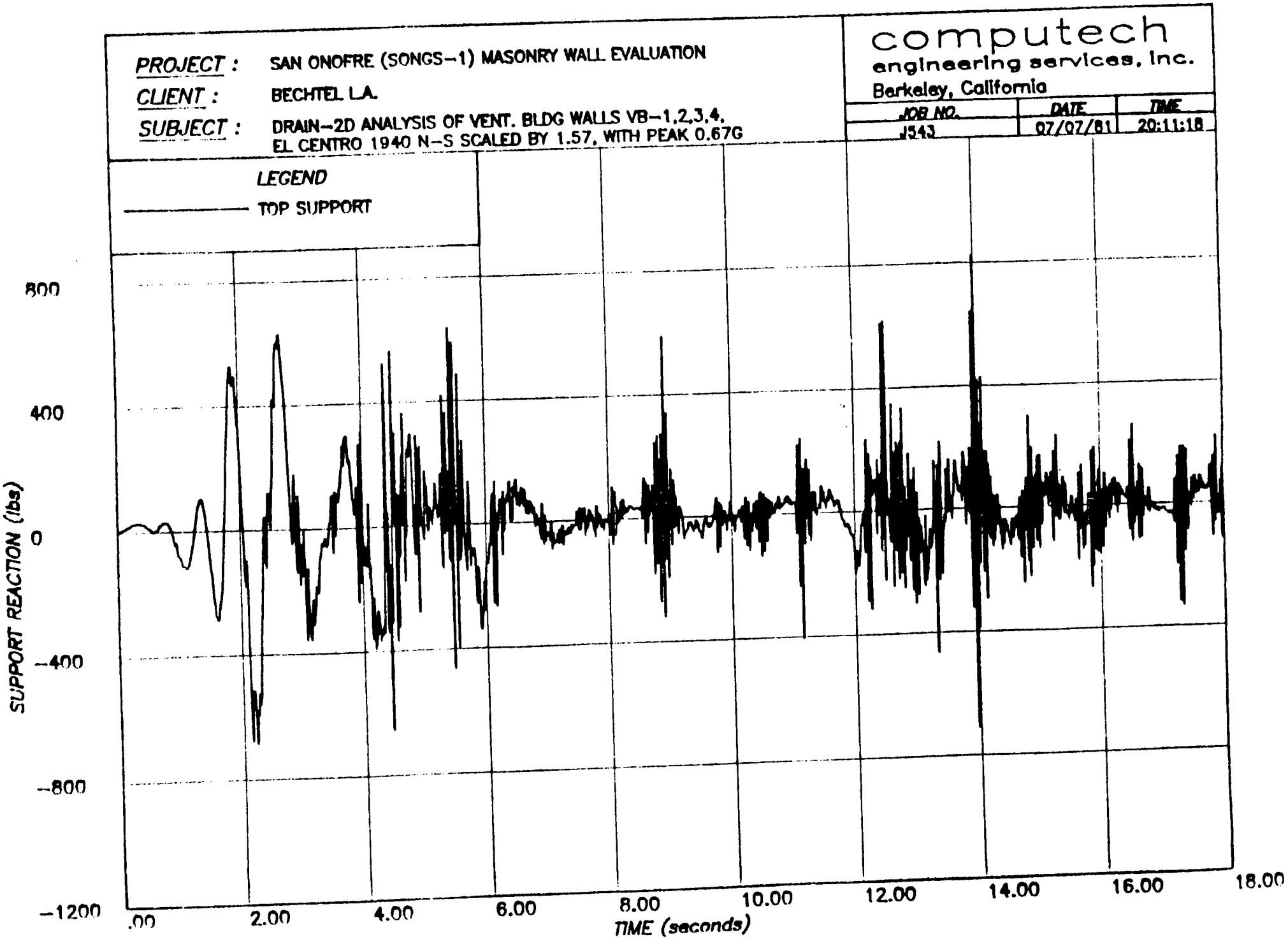


FIGURE 4.4 VENTILATION BUILDING : EL CENTRO

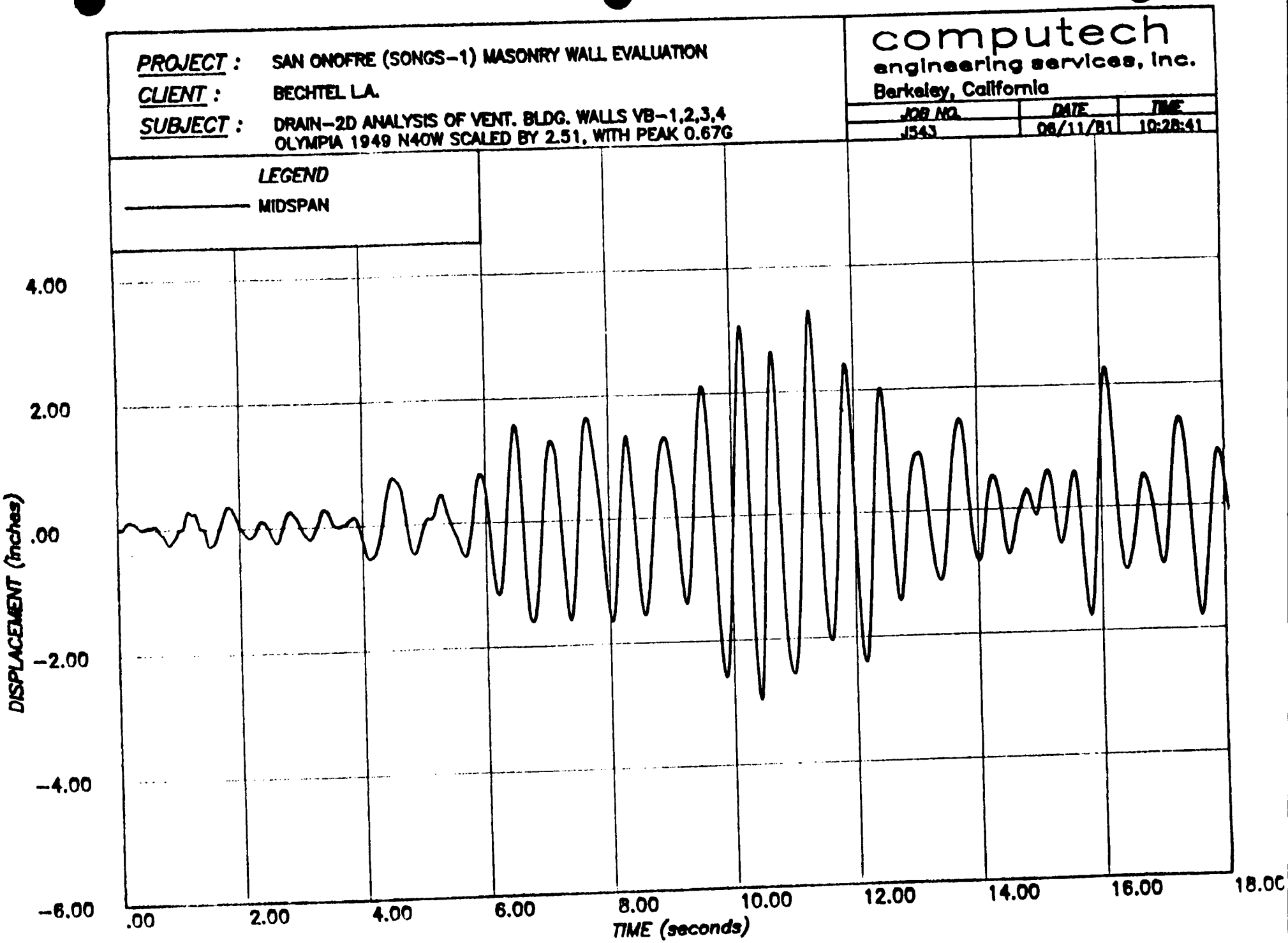


FIGURE 4.5 VENTILATION BUILDING : OLYMPIA

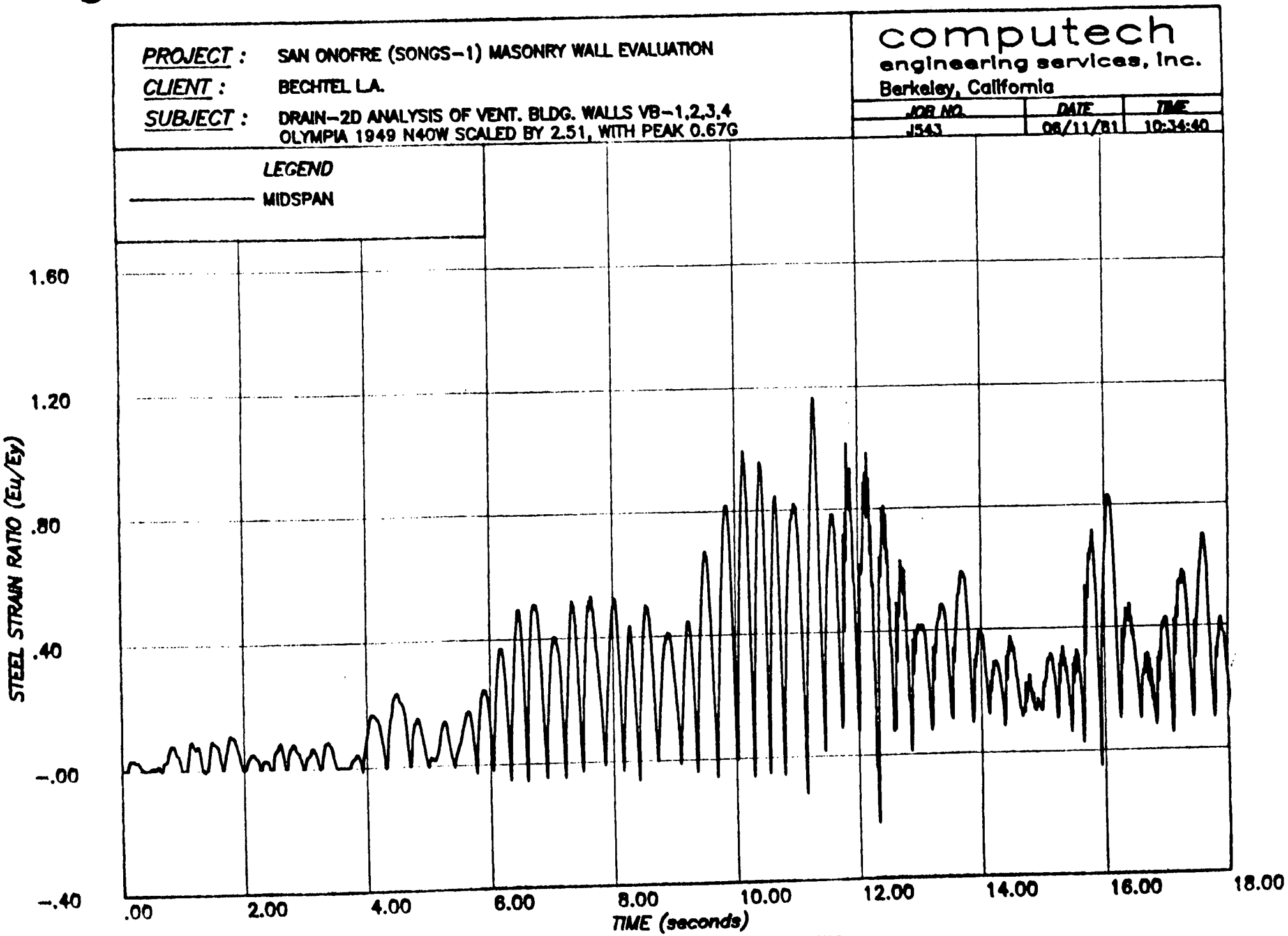


FIGURE 4.6 VENTILATION BUILDING : OLYMPIA

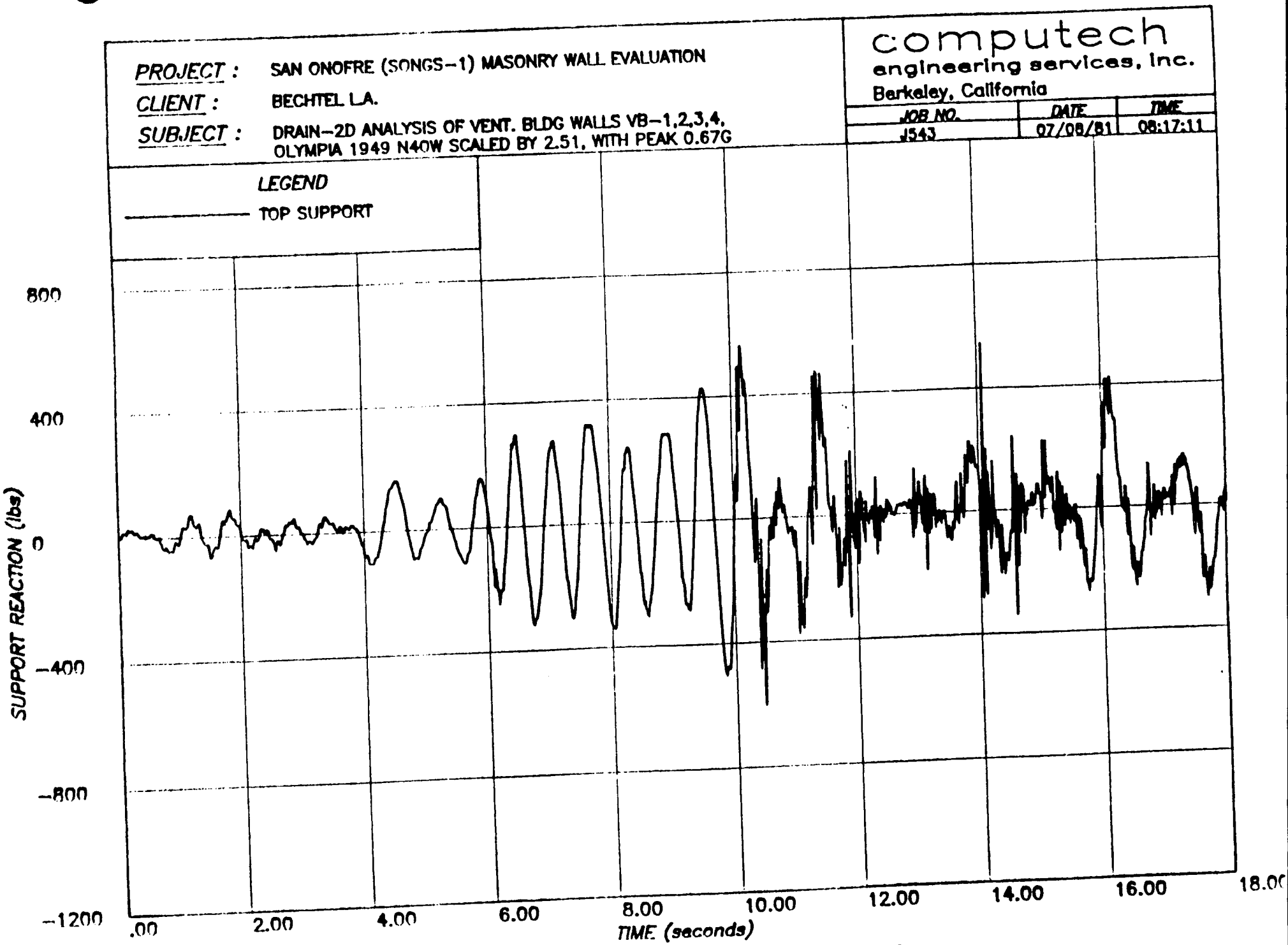


FIGURE 4.7 VENTILATION BUILDING : OLYMPIA

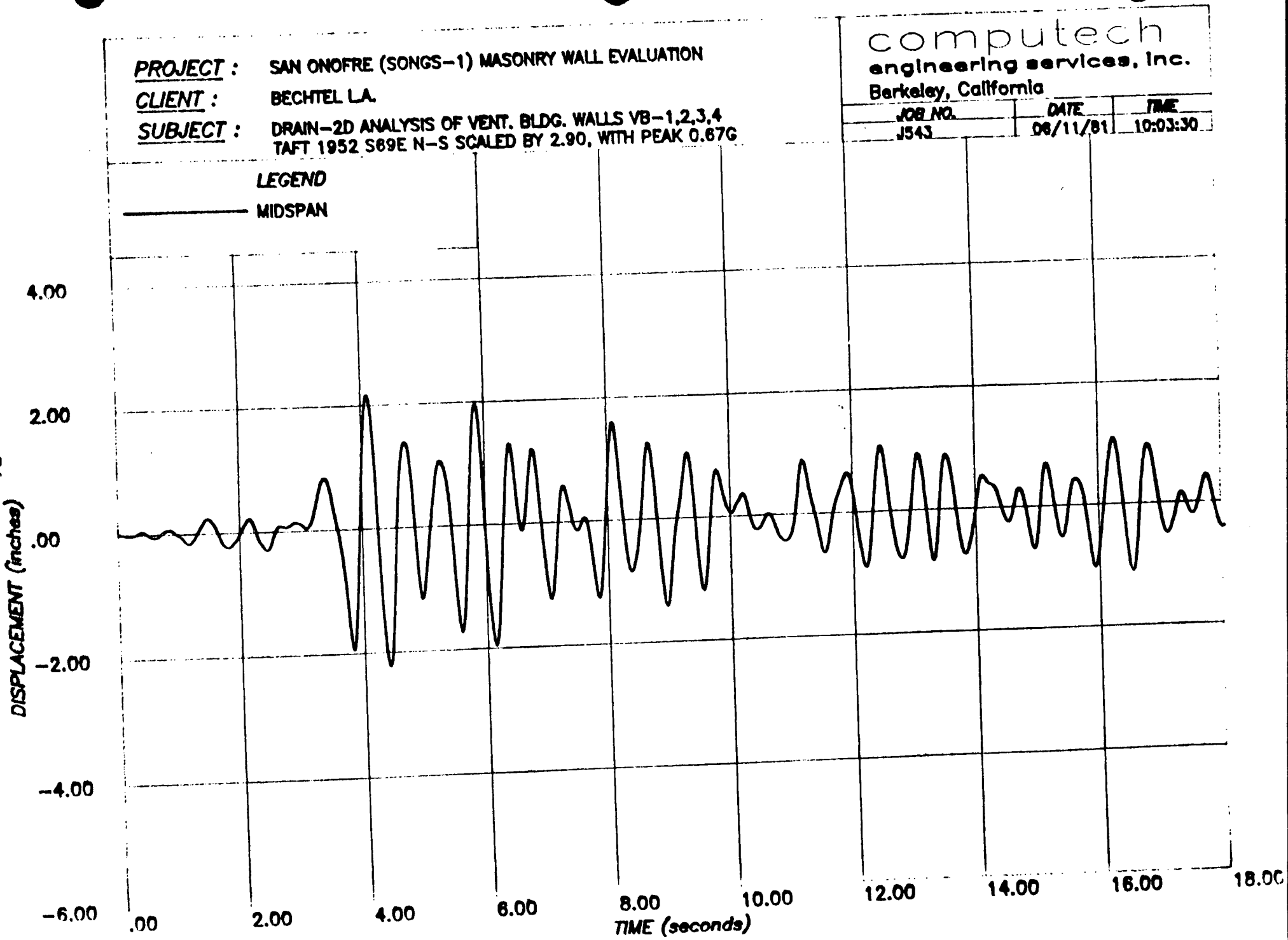


FIGURE 4.8 VENTILATION BUILDING : TAFT

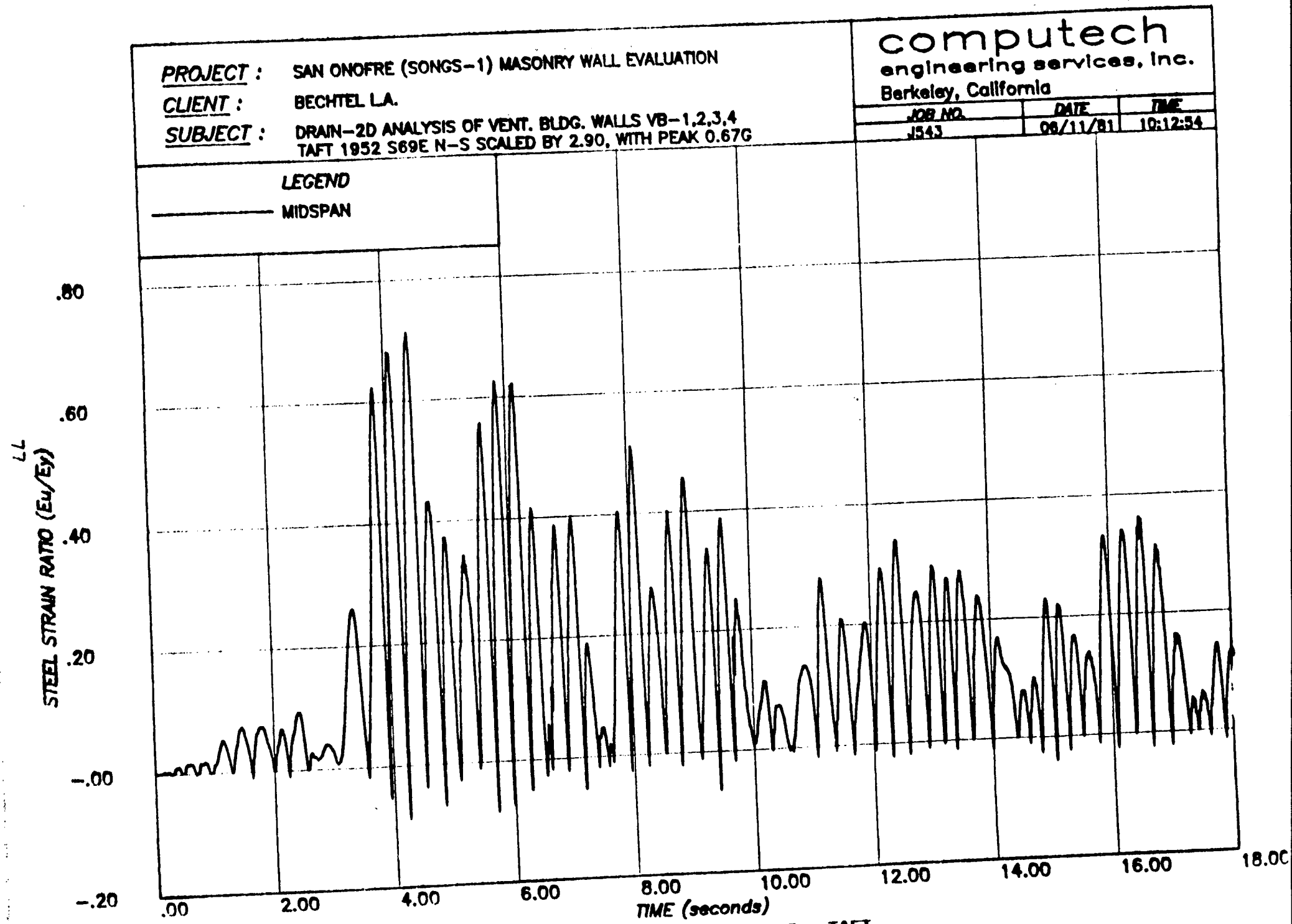


FIGURE 4.9 VENTILATION BUILDING : TAFT

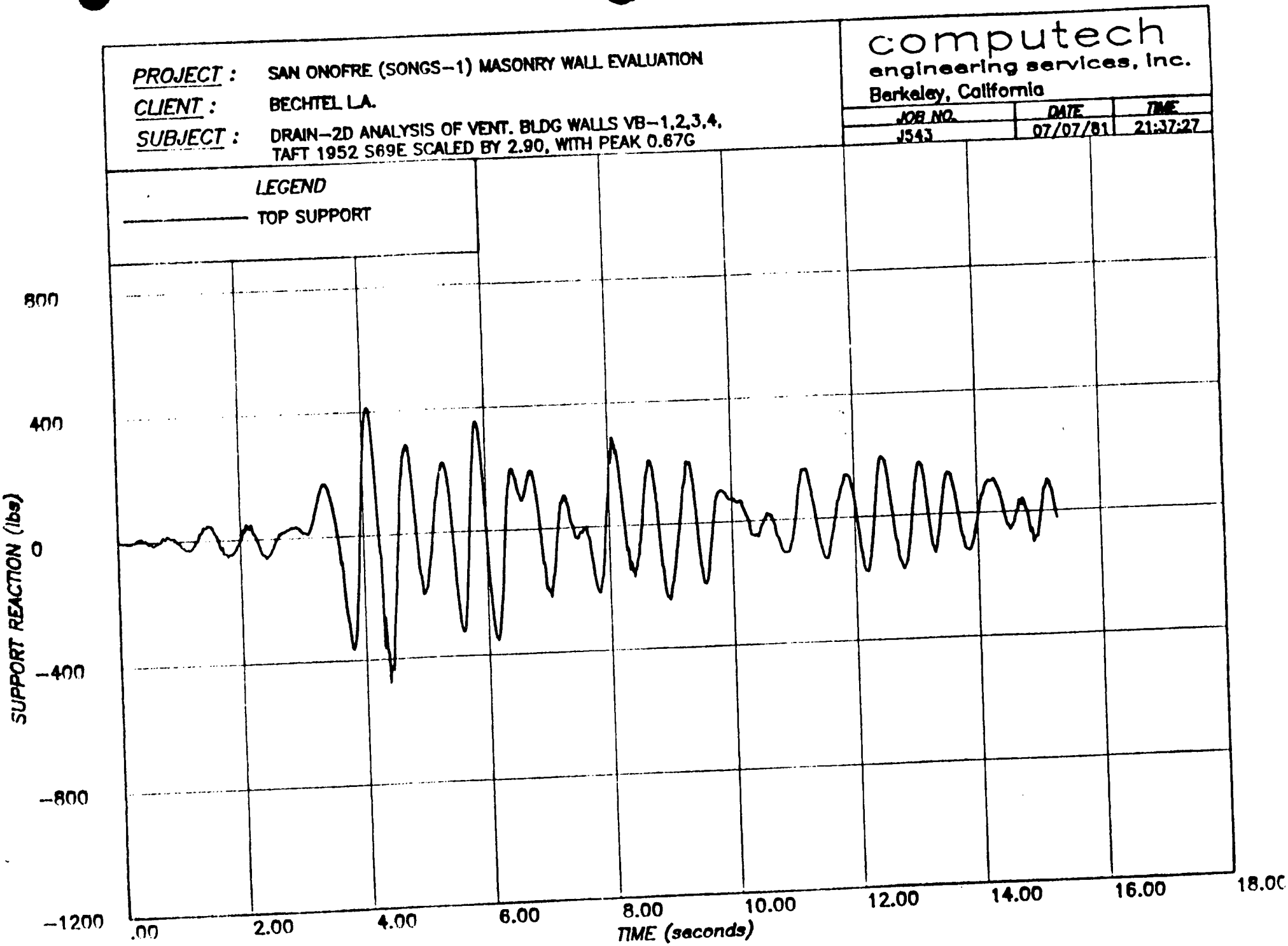
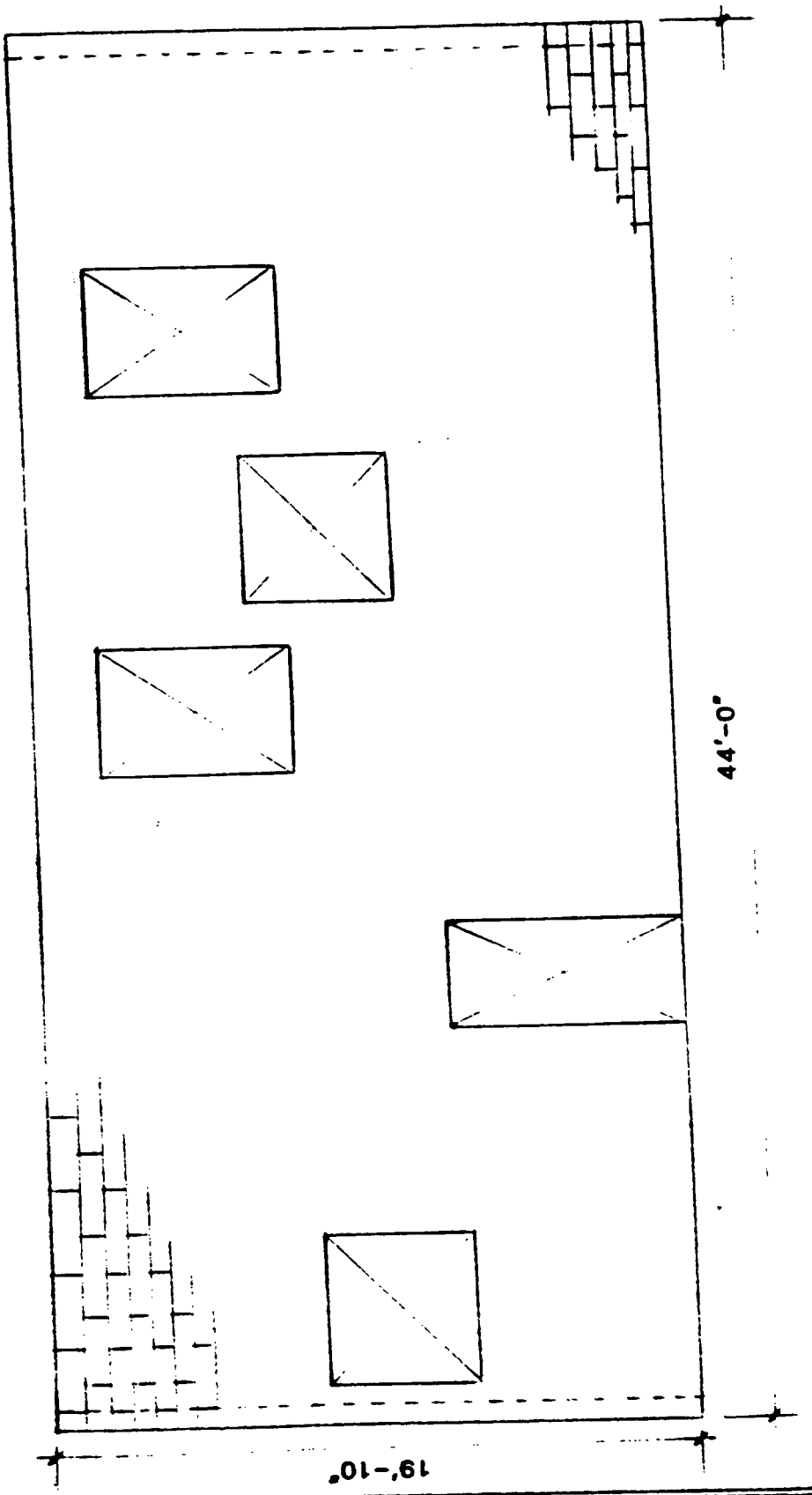


FIGURE 4.10 VENTILATION BUILDING : TAFT

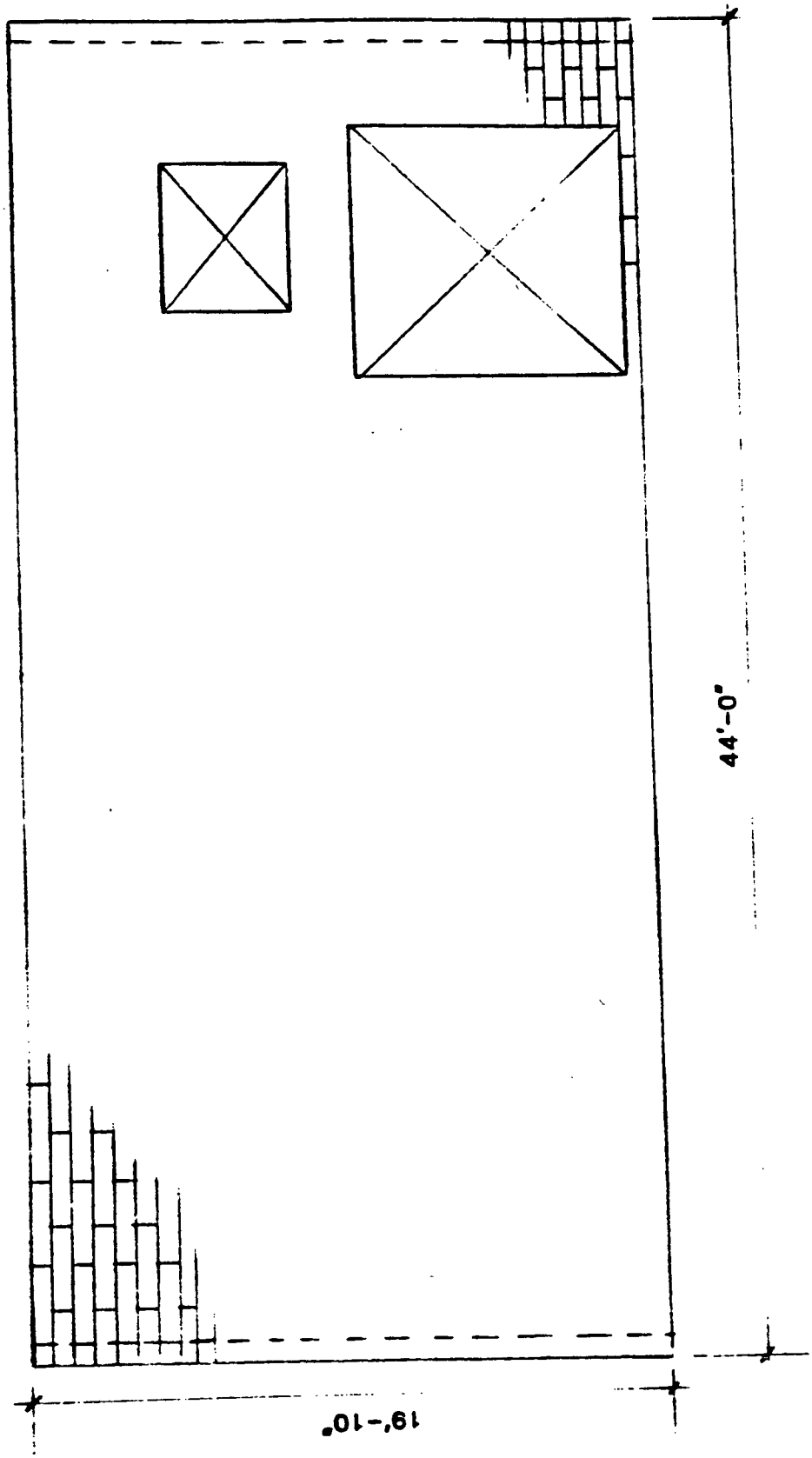
COMPUTECH



WALL ELEVATION

PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
DRAWN	VENTILATION BUILDING : WALL ELEVATION VB-1	4.11
CHECKED		

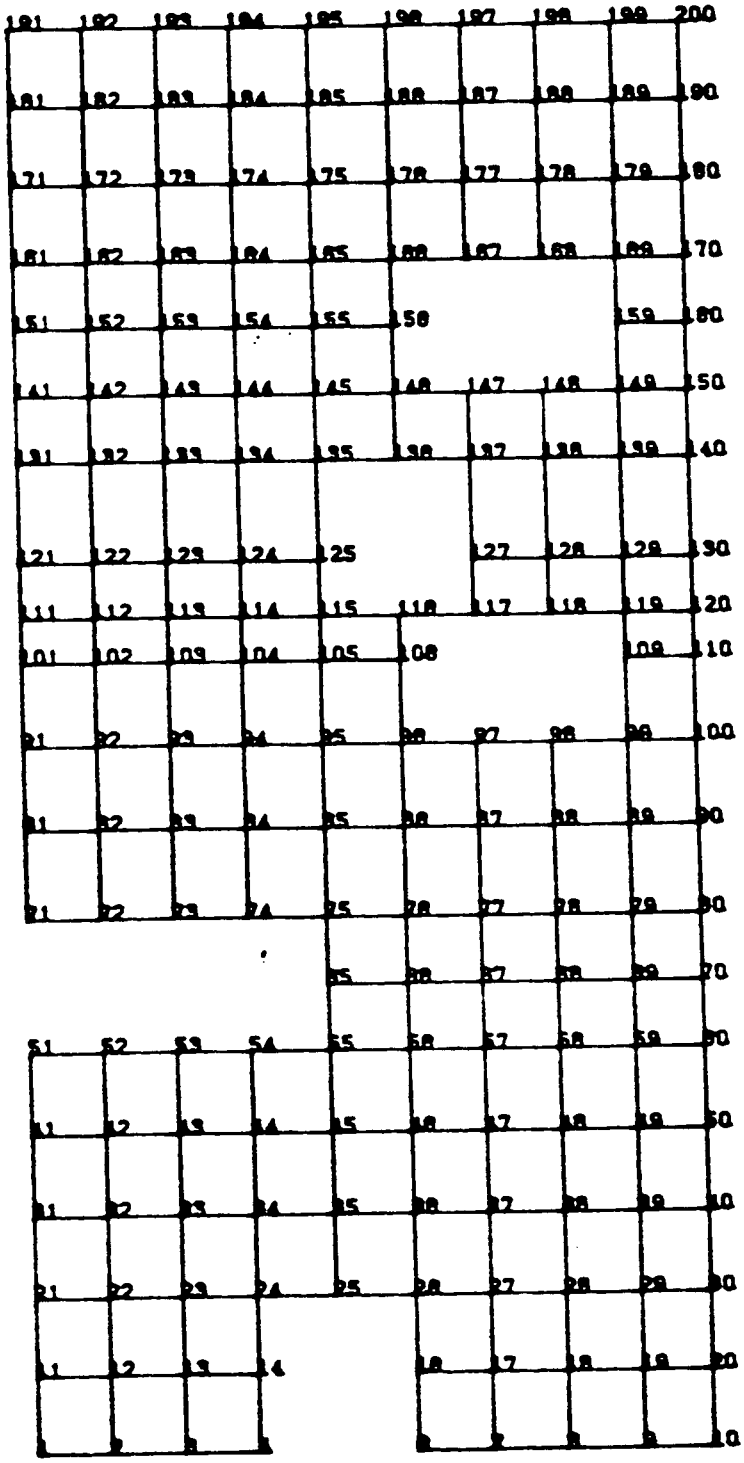
COMPUTECH



WALL ELEVATION

PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO.
DRAWN	VENTILATION BUILDING : WALL ELEVATION VB-2	4.12
CHECKED		

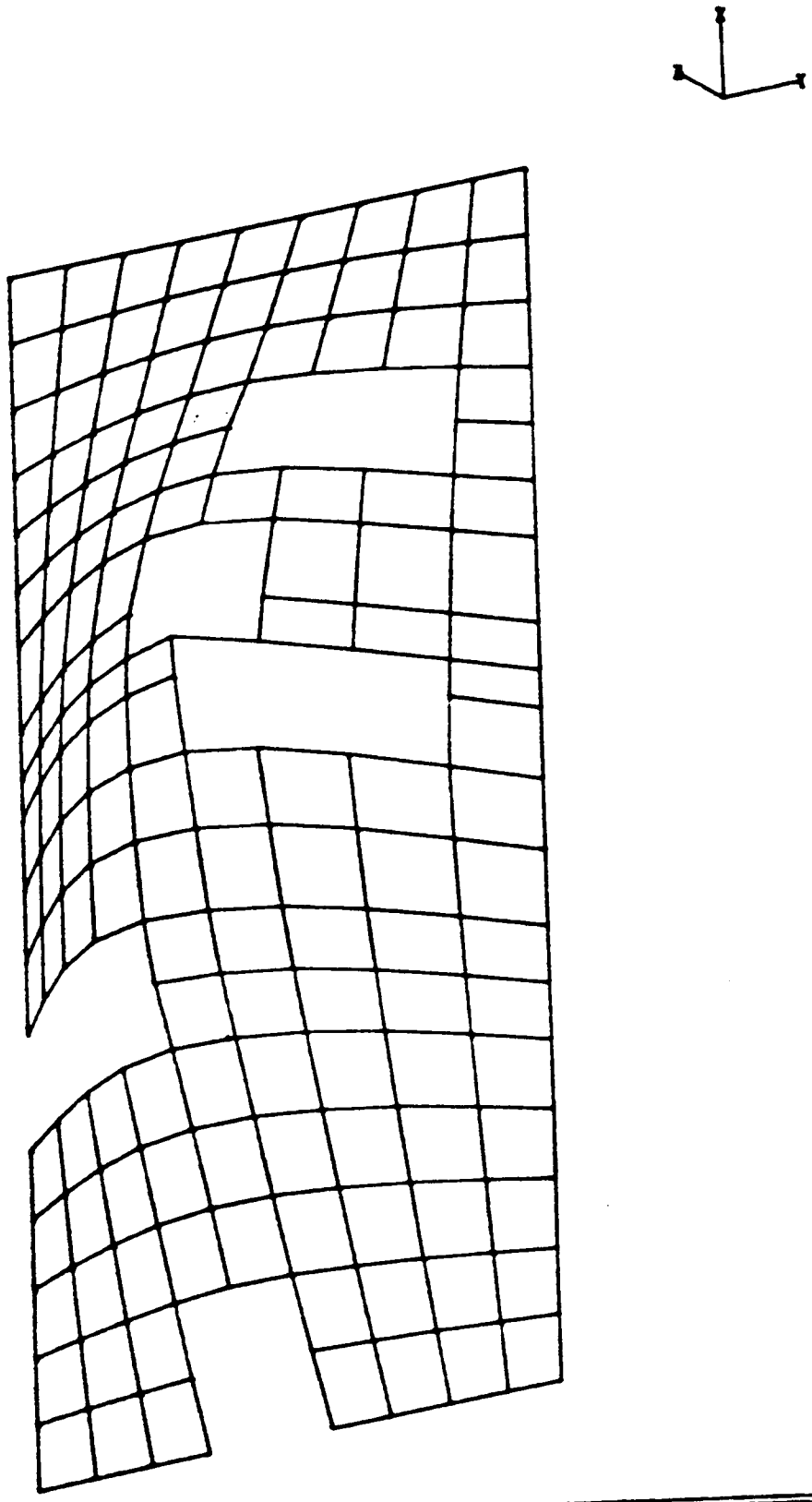
VENTILATION BLDG., WALL VB-1
UNDEFORMED SHAPE



PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
DRAWN	VENTILATION BUILDING : "SAP" MESH VB-1	4.13
CHECKED		

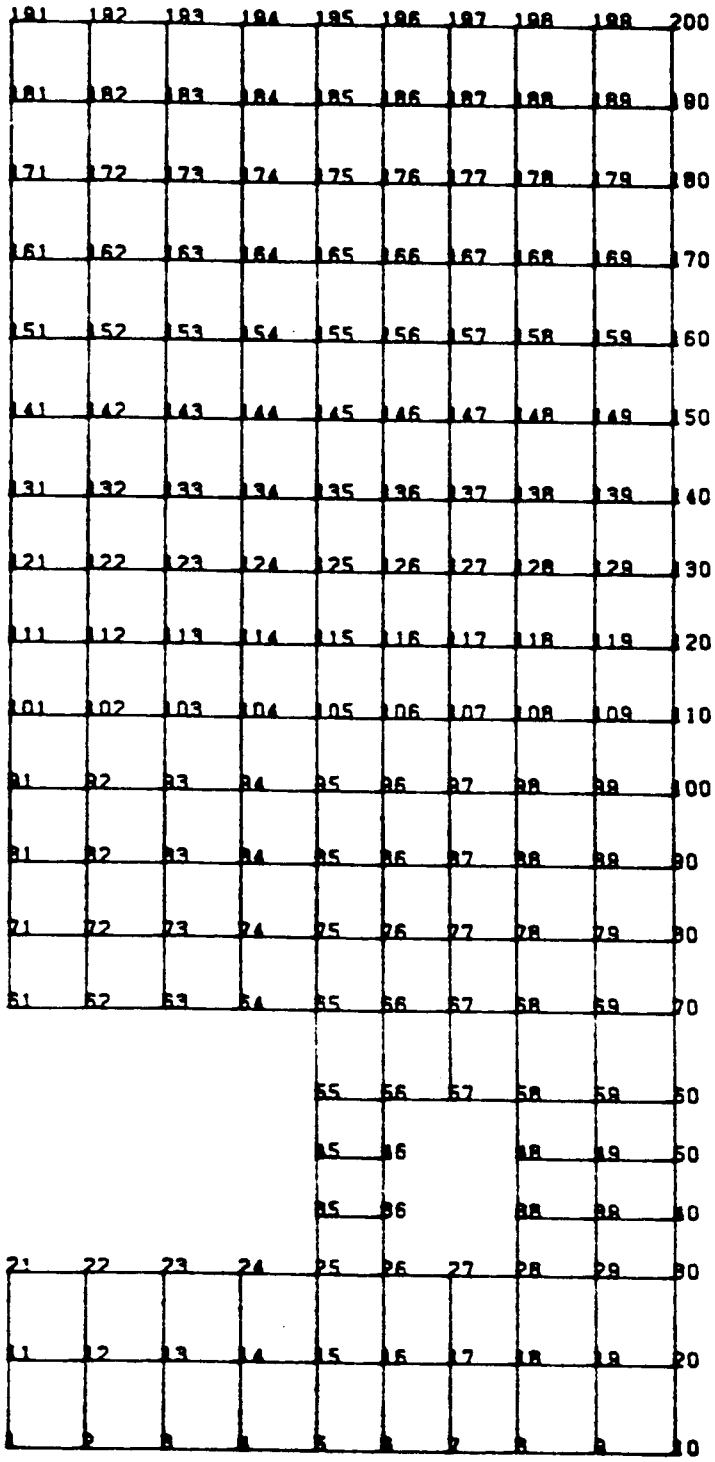
COMPUTECH

VENTILATION BLDG., WALL VB-1
MODE 1 FREQ = 2.4677 Hz



PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
DRAWN	VENTILATION BUILDING : MODE 1 VB-1	4.14
CHECKED		

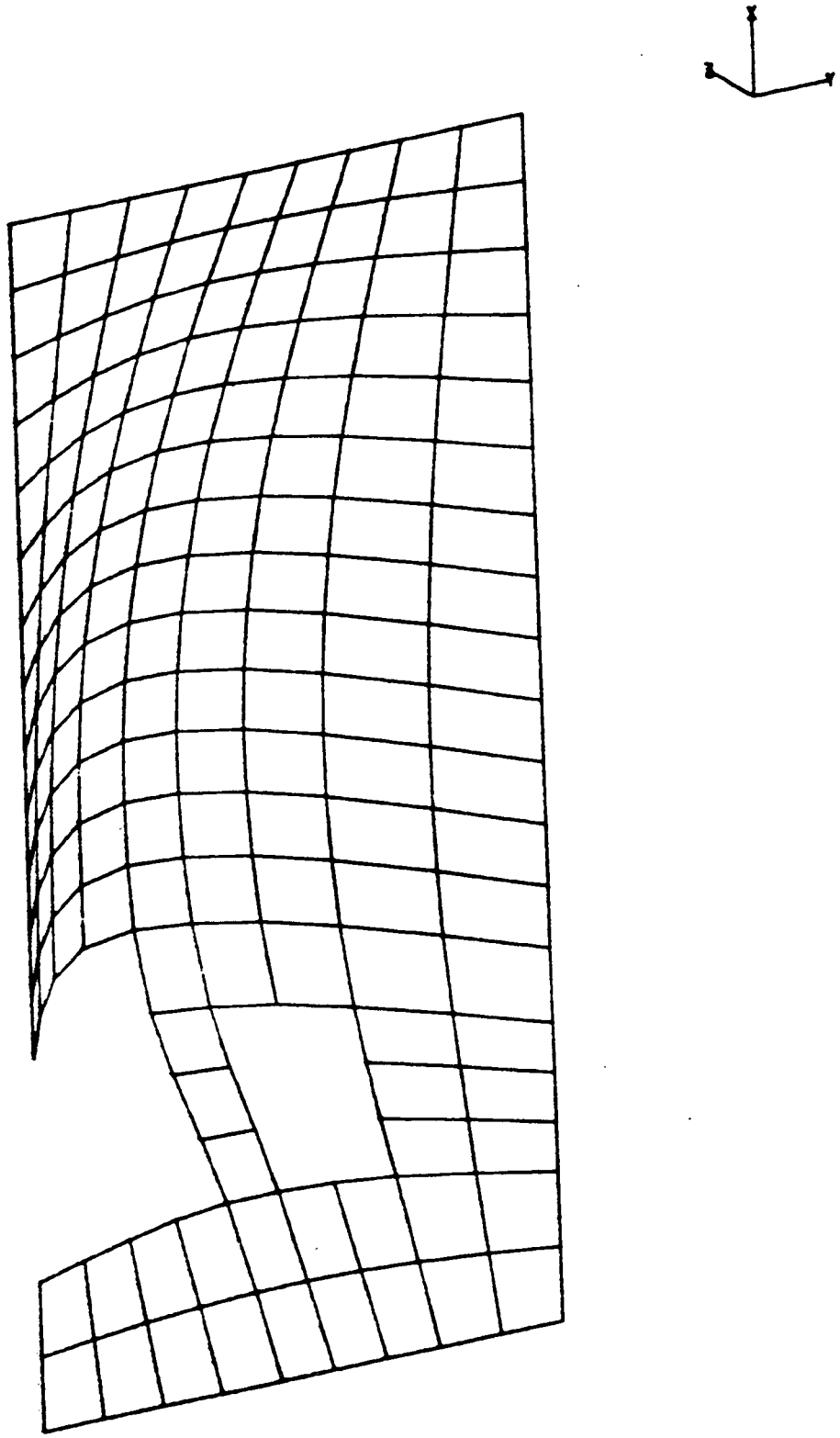
VENTILATION BLDG., WALL VB-2
UNDEFORMED SHAPE



PROJECT NO 643	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
DRAWN	VENTILATION BUILDING : "SAP" MESH VB-2	4.15
CHECKED		

COMPUTECH

VENTILATION BLDG., WALL VB-2
MODE 1 FREQ. = 2.4340 Hz



PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
DRAWN	VENTILATION BUILDING : MODE 1 VB-2	4.16
CHECKED		

5 REACTOR AUXILIARY BUILDING

5.1 Description of Walls

The Reactor Auxiliary Building has a total of 15 masonry walls of which 7 walls, numbered SB-1 through SB-7, were subject to evaluation. These walls are all in the Storage Building. The Storage Building is a masonry structure with a northern section 16 feet high and 32 feet by 21 feet in plan and a southern portion 20 feet by 17 feet in plan and comprised of two storys each approximately 10'-6" in height.

Of the walls SB-1 through SB-7 preliminary elastic analyses indicated the possibility of the elastic limit being exceeded for all except SB-5. These walls are both one and two span although the spans of the two span walls are separated by a concrete diaphragm at elevation 30'-10". The building is laterally supported by adjacent concrete up to elevation 32'-8" and so it may be assumed that the ground response spectrum applies up to this level. For this reason the two span walls may be considered as two separate walls.

The clear spans of the walls vary from 8'-0" to 14'-8" with the individual spans of the two span walls being approximately 10 feet high. For the evaluation of the masonry walls in this building two models were analyzed with heights corresponding to the upper and lower bound values of 8'-0" and 14'-8" respectively. For the purposes of this report the former is classified as Group I and the latter Group II. These two groups therefore encompass all safety related walls in the building. Typical sections for these two groups are shown in Figures 5.1 and 5.2.

The reinforcement in the walls varies but is a minimum of #5 bars at 48" vertically and horizontally and this area has been assumed for the models in each group. Added weights from equipment are relatively small. A number of openings exist in the walls for door openings.

5.2 Inelastic Analysis

Each wall group was modelled for analysis using the DRAIN-2D program in accordance with the methodology developed in Volume 2 of this report. The three time histories as described in Section 2.2 were used for the analyses.

As the wall models assumed simple supports top and bottom and the added mass was low, advantage was taken of symmetry to reduce the number of degrees of freedom in each model. Therefore only one half of the wall in each group was modelled. A rigid strut was added at the support position to enable support reaction forces to be extracted

5.3 Results of Analysis

Tables 5.1 and 5.2 summarize the maximum response parameters for the Group I and Group II walls respectively. Figures 5.3 to 5.11 are the time histories of displacement, steel ductility and reaction forces for the El Centro, Olympia and Taft records for Group I. In Figures 5.12 to 5.20 the same response parameters for the Group II walls are reproduced.

Displacements in the 8'-0" walls of Group I were relatively small and as shown by steel strain ratios less than 1.0 the wall remained elastic for all time histories. The steel strain reached a maximum value of 84% of yield for the El Centro time history and less for the other two records used. Because of the relatively low reinforcing ratio the masonry compressive stresses were low. The maximum support reaction varied considerably from a high of 779 lb/ft under El Centro to a low value of 240 lb/ft under the Taft record. The time histories of response for this group reflect the elastic response with the frequency being relatively high and remaining constant throughout each analysis.

For the Group II walls Table 5.2 shows that yield occurred in the walls for each of the three time histories. The results show a comparatively low variation between each of the time histories, with the maximum displacements varying from 2.38" to 2.99" and the steel ductility from 2.70 to 3.78. In general El Centro and Olympia produced results of similar magnitude and the Taft record was slightly less. The effect of yielding is evident in the time history plots where the effective period of the wall lengthens after yield occurs.

5.4 Evaluation of Results

The acceptance criteria for the transverse analysis of masonry walls at the San Onofre, Unit 1 plant are given in Section 2.1 of Volume 1 of this report. In the following sections the results of the Reactor Auxiliary Building walls are evaluated in terms of this criteria.

5.4.1 Reinforcement Ductility

The acceptance criteria set an allowable limit of 45 on the maximum steel strain ratio. For Group I yield did not occur and so this limit is not applicable. For Group II the maximum value of 3.8 was well within the limit.

5.4.2 Masonry Compressive Stress

The criteria limit of 0.85 f'm based on a uniform stress block provides for a maximum compressive stress of 1147 p.s.i. for San Onofre, Unit 1. Maximum values were 168 psi and 237 psi for the Group I walls and Group II walls respectively well below the limit. The relatively low values are due to the low steel area as the maximum masonry force

is limited by the yield force in the rebars.

5.4.3 Wall Support

The maximum wall support forces are 779 lbs and 708 lbs for the 8'-0" and 14'-8" walls respectively. For walls of height between these two values interpolation was used to obtain design forces for evaluating the adequacy of connections.

5.4.4 Wall Stability

The time history of response in Figures 5.3 to 5.20 demonstrates that for both groups the response is stable and the displacements oscillate about the original undeflected baseline.

The maximum displacements did not exceed the wall thickness for either of the two groups. Therefore the static check for stability was not required.

5.4.5 Wall Openings

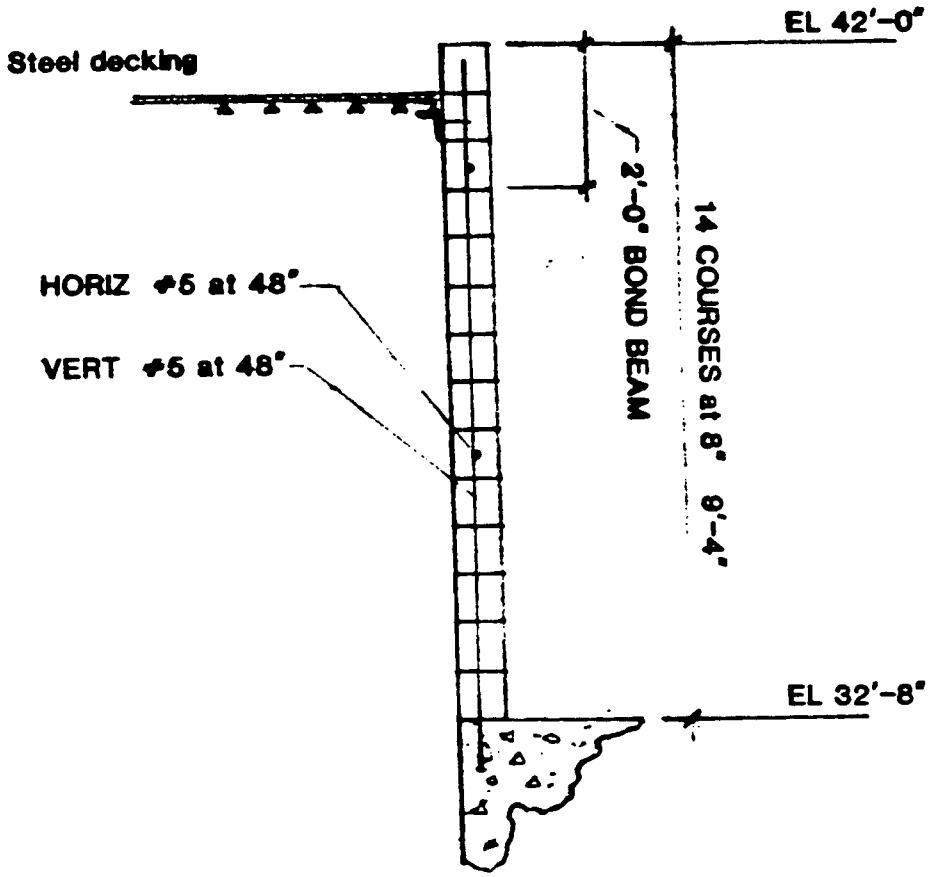
The wall openings are generally less than 4'-0" in width and the trimmer bars provide considerably more additional steel than is curtailed by openings of this size. In general 2 #5 trimmer bars are added on each side of the opening and only 1 #5 bar is curtailed. The maximum masonry stress obtained from the inelastic analyses did not exceed 237 psi, less than 25% of the allowable value. Therefore the allowable stress on the masonry will not increase beyond the allowable limit for the proportion of openings in these walls.

	EARTHQUAKE RECORD		
	EL CENTRO 1940	TAFT 1952	OLYMPIA 1949
DISPLACEMENTS (inches)			
Mid-Span Maximum	0.37	0.12	0.38
Mid-Span Minimum	-0.32	-0.14	-0.34
STEEL STRAIN RATIO (Eu/Ey)	0.84	0.25	0.79
SUPPORT REACTION (lbs)	779	240	563
MASONRY COMPRESSIVE STRESS (psi)	168	49	159

TABLE 5.1 : SUMMARY OF RESULTS : REACTOR AUXILIARY BUILDING GROUP I

	EARTHQUAKE RECORD		
	EL CENTRO 1940	TAFT 1952	OLYMPIA 1949
DISPLACEMENTS (inches)			
Mid-Span Maximum	2.99	2.38	2.00
Mid-Span Minimum	-2.33	-2.23	-2.86
STEEL STRAIN RATIO (Eu/Ey)	3.78	2.70	3.67
SUPPORT REACTION (lbs)	708	598	525
MASONRY COMPRESSIVE STRESS (psi)	237	237	237

TABLE 5.2 : SUMMARY OF RESULTS : REACTOR AUXILIARY BUILDING GROUP II

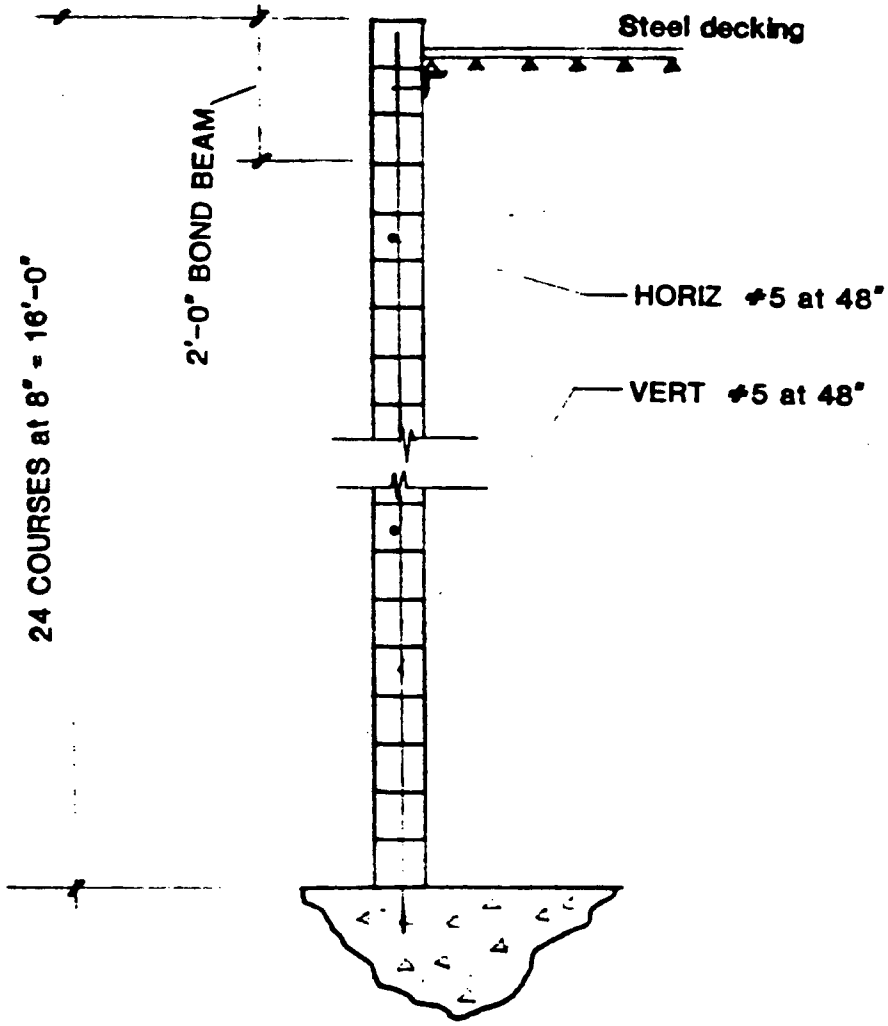


A
—

568122-2

SB-5

PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
DRAWN	TYPICAL SECTION : GROUP I	5.1
CHECKED		



568122-2

NORTH ELEVATION
SB-2

PROJECT NO 543	SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1	FIGURE NO
DRAWN		
CHECKED	TYPICAL SECTION : GROUP II	5.2

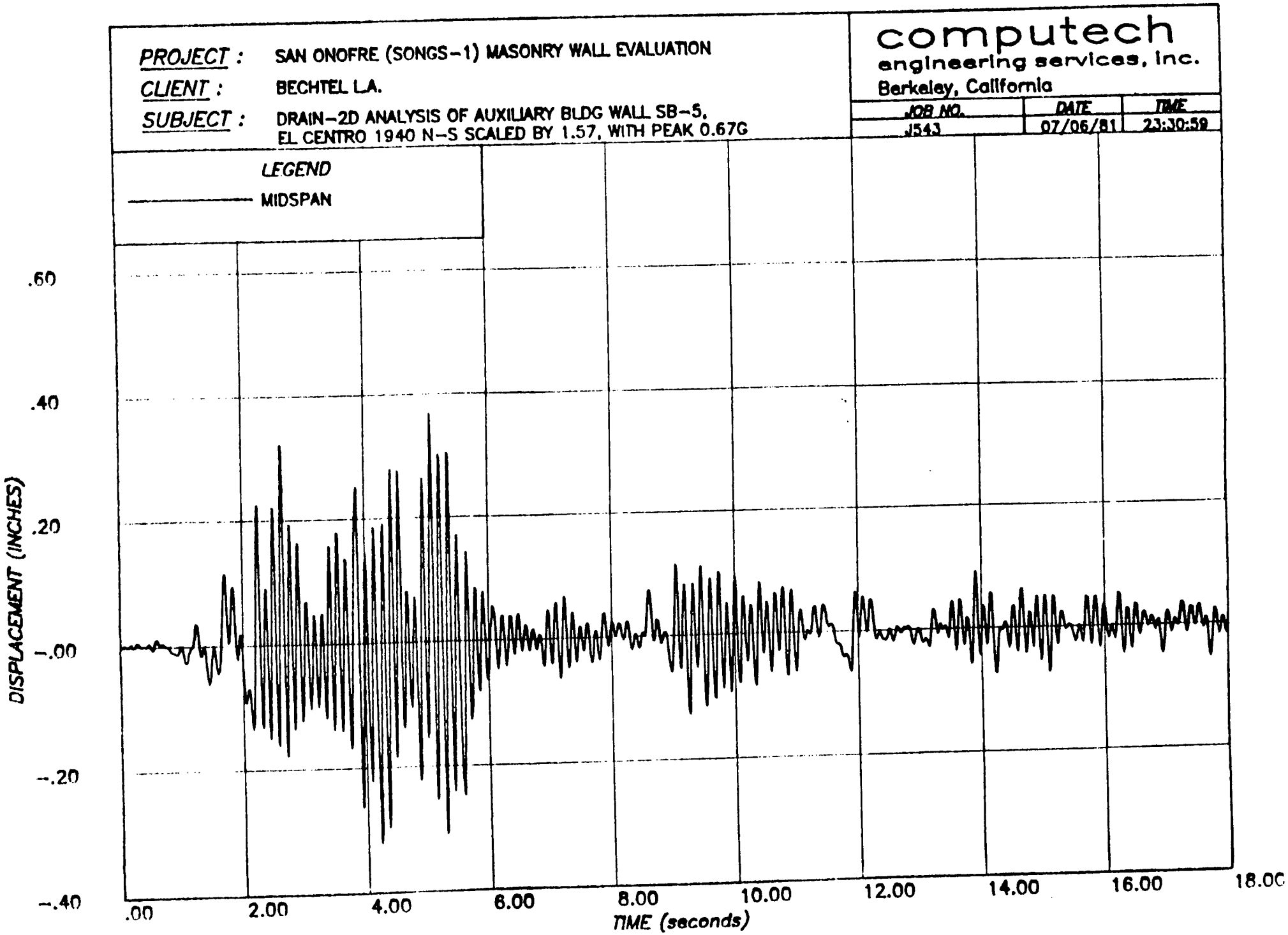


FIGURE 5.3 : REACTOR AUXILIARY BUILDING GROUP I: EL CENTRO

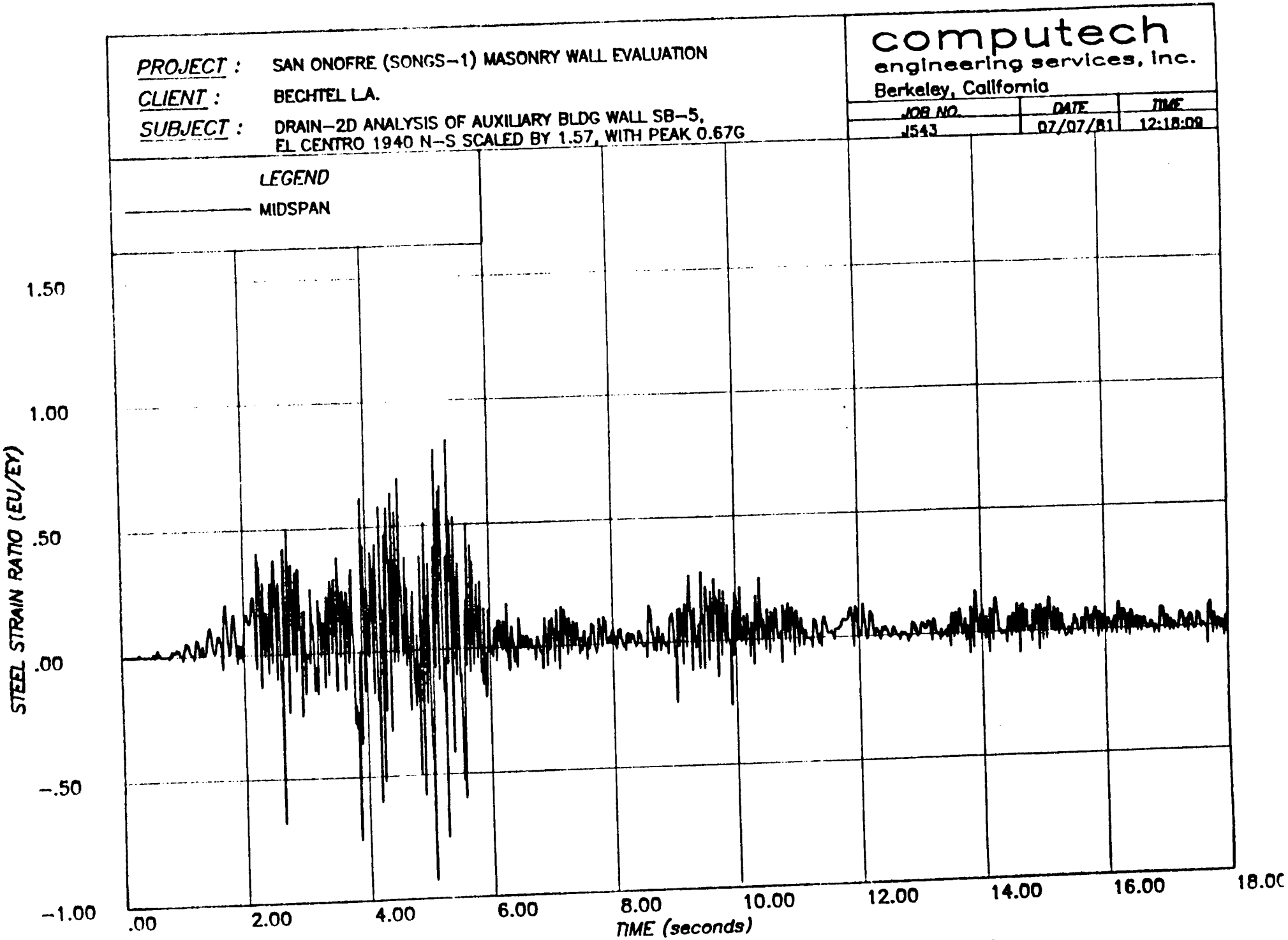


FIGURE 5.4 : REACTOR AUXILIARY BUILDING GROUP I: EL CENTRO

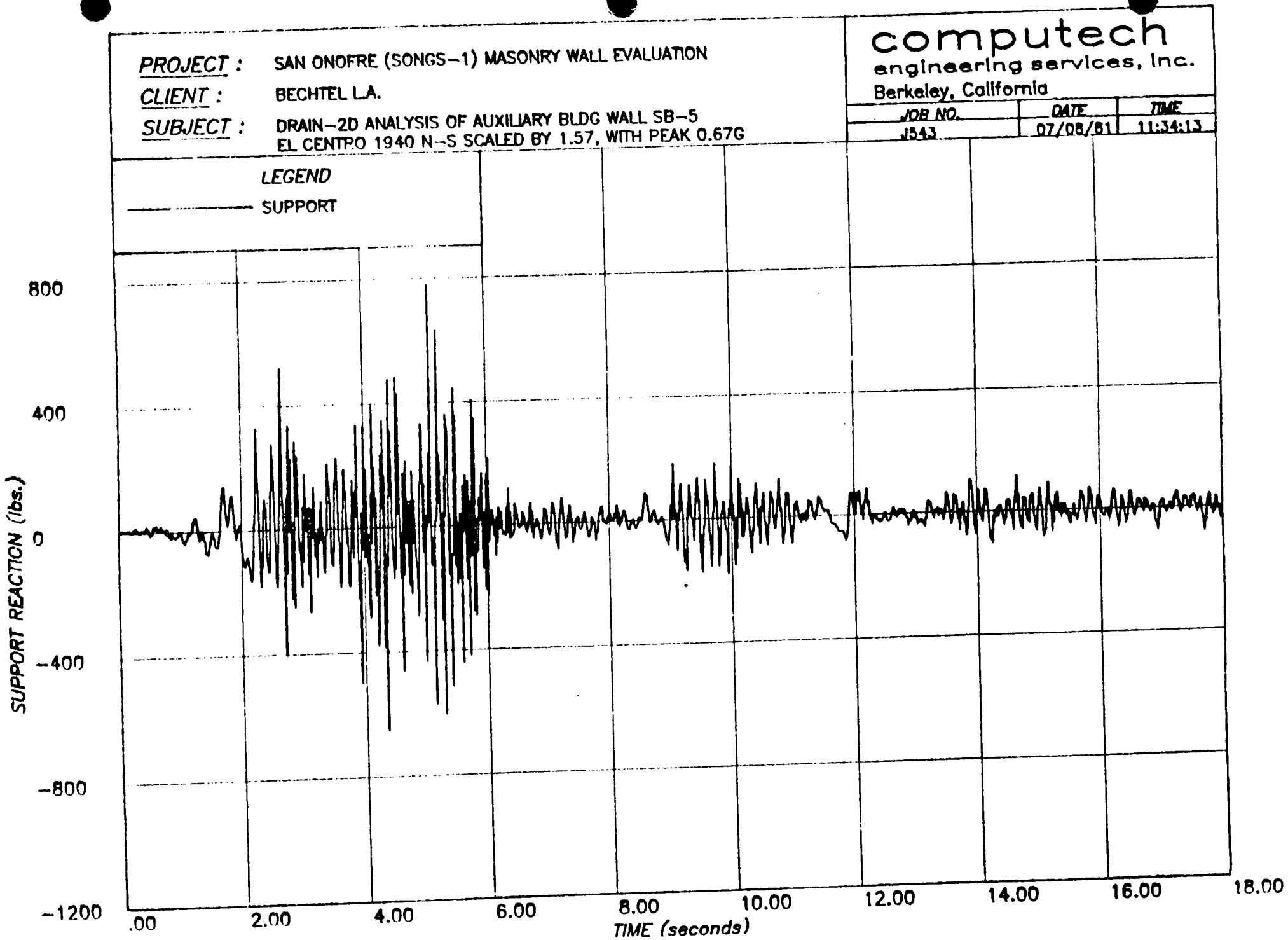


FIGURE 5.5 : REACTOR AUXILIARY BUILDING GROUP I: EL CENTRO

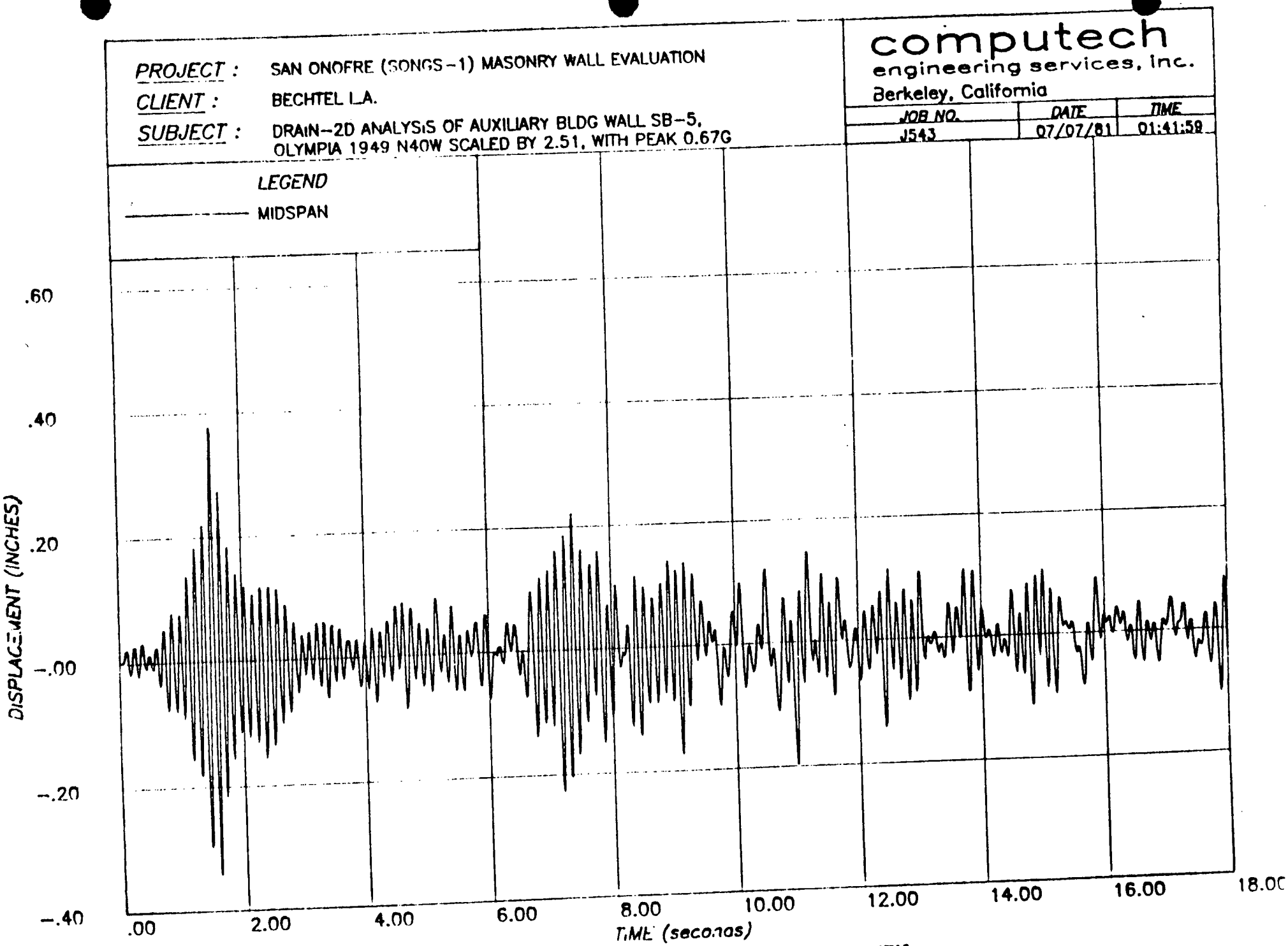


FIGURE 5.6 : REACTOR AUXILIARY BUILDING GROUP I: OLYMPIA

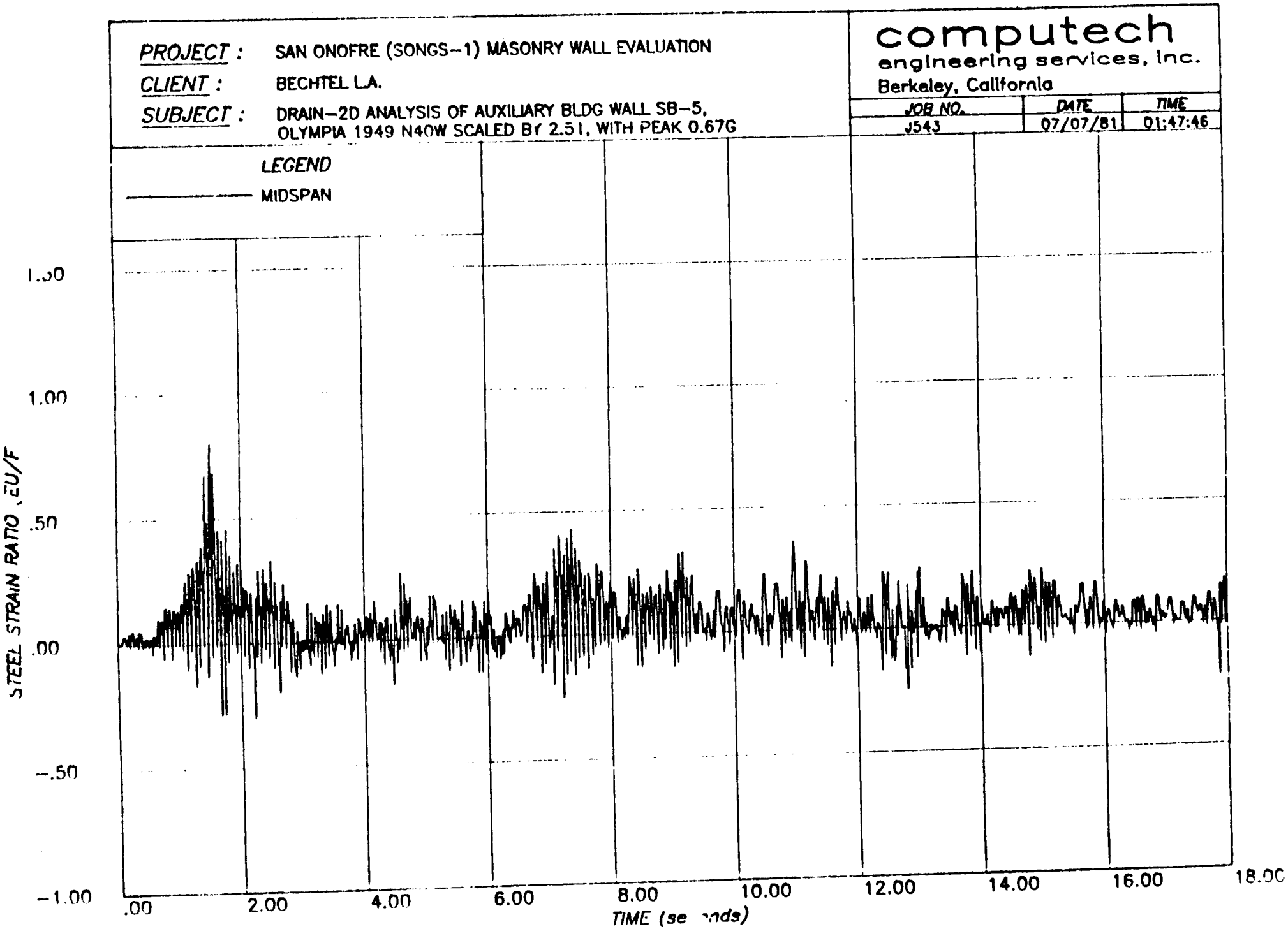


FIGURE 5.7 : REACTOR AUXILIARY BUILDING GROUP I: OLYMPIA

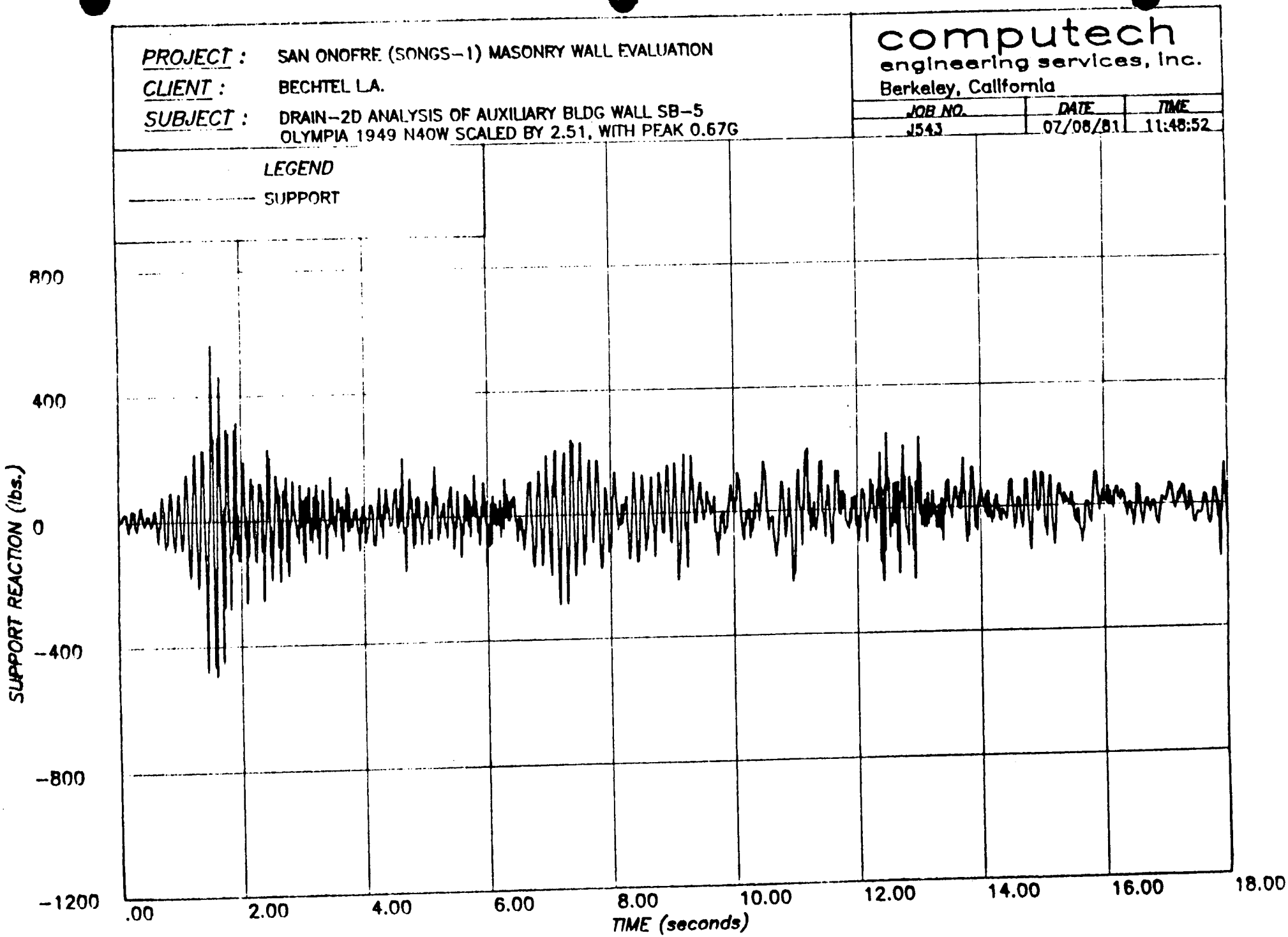


FIGURE 5.8 : REACTOR AUXILIARY BUILDING GROUP I: OLYMPIA

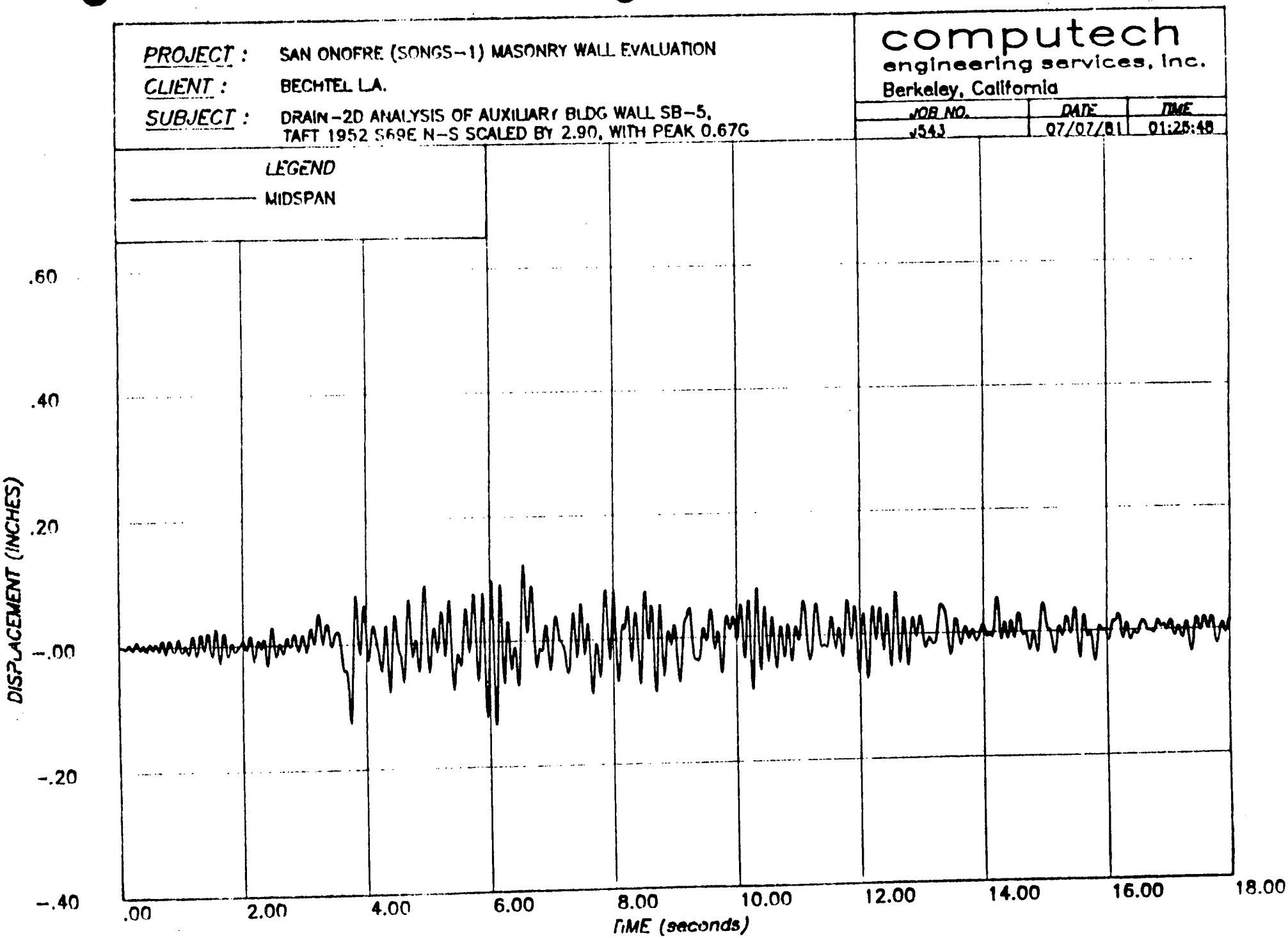


FIGURE 5.9 : REACTOR AUXILIARY BUILDING GROUP I: TAFT

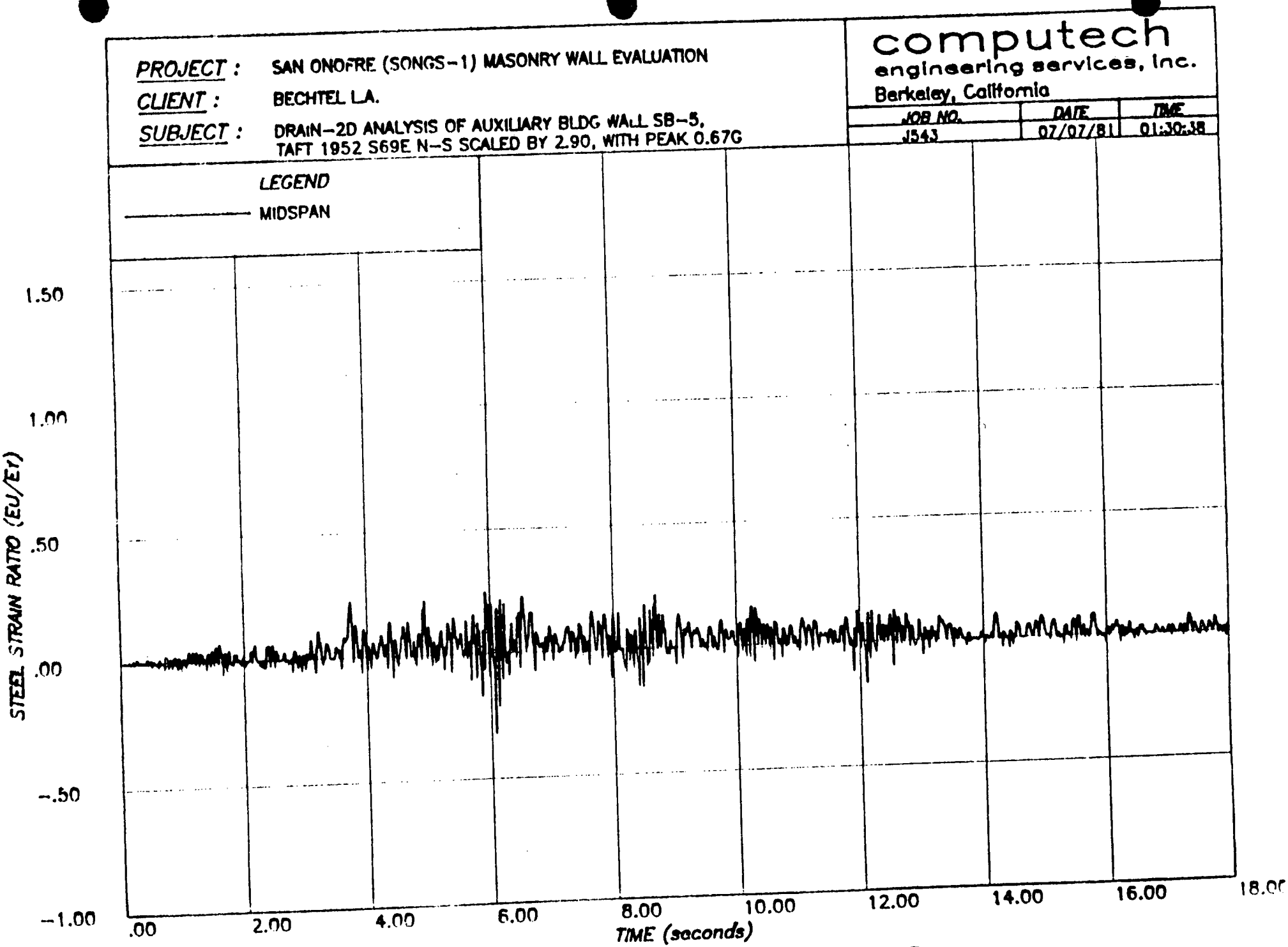


FIGURE 5.10 : REACTOR AUXILIARY BUILDING GROUP I: TAFT

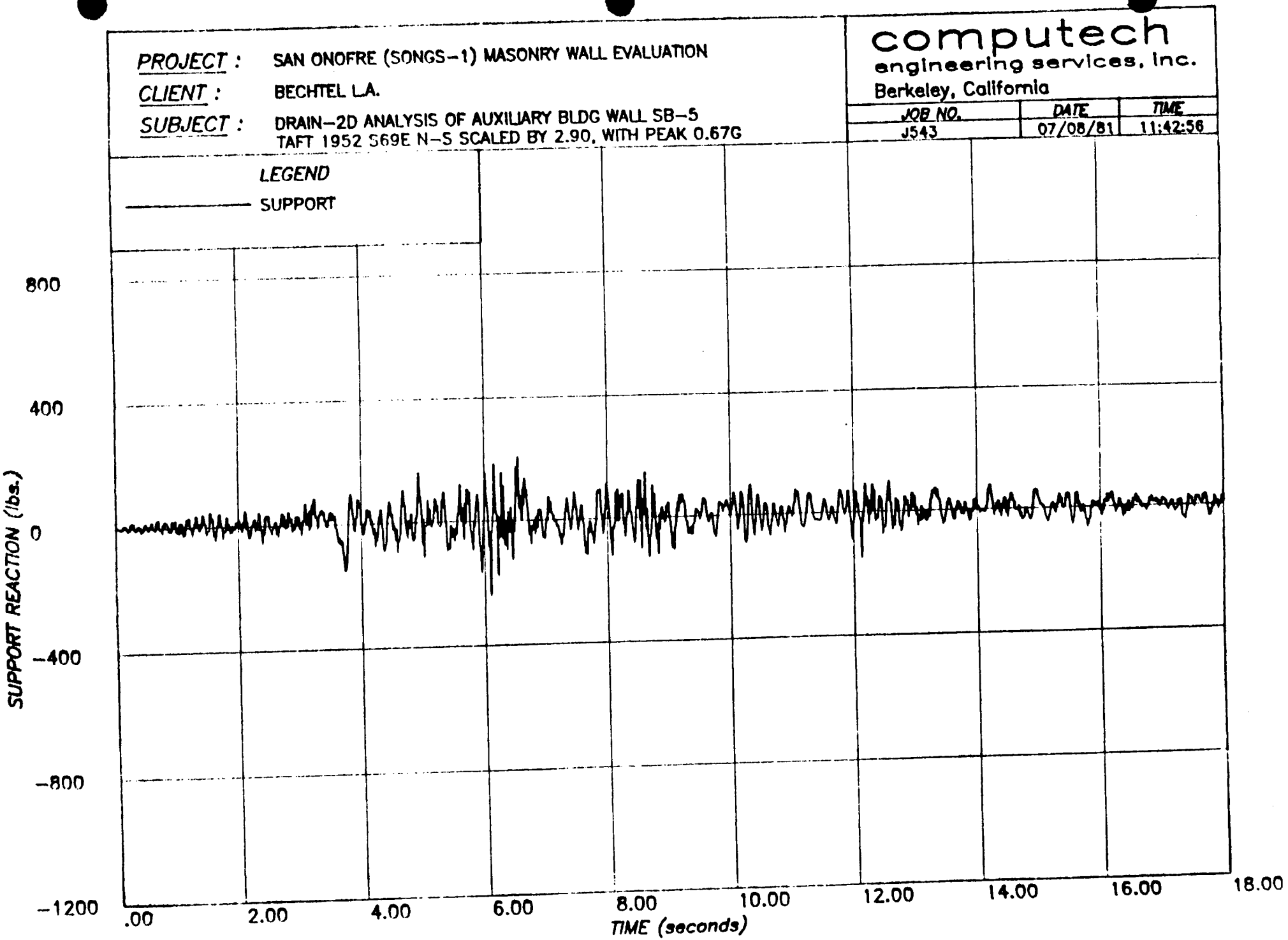


FIGURE 5.11 : REACTOR AUXILIARY BUILDING GROUP I: TAFT

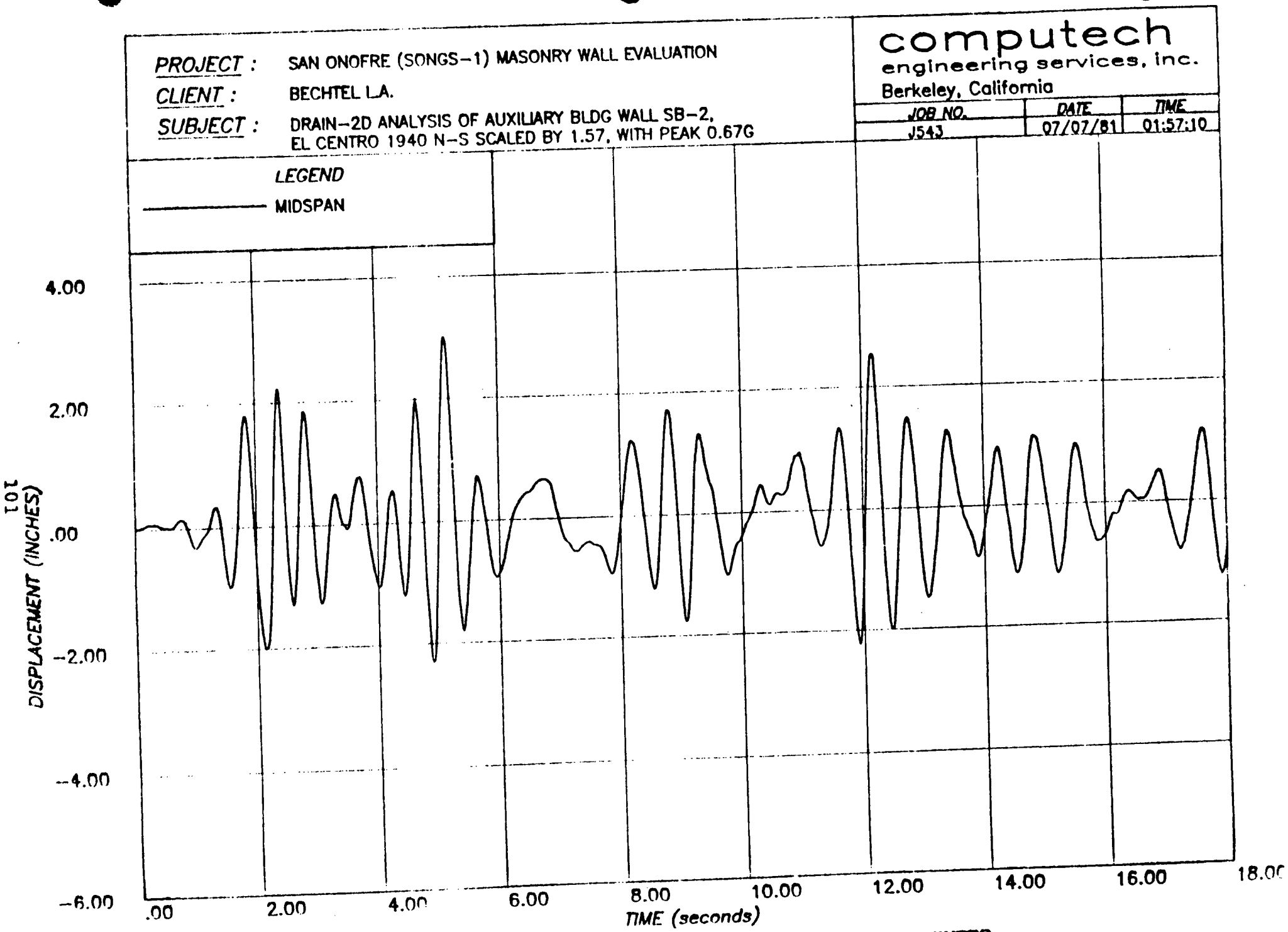


FIGURE 5.12 : REACTOR AUXILIARY BUILDING GROUP II: EL CENTRO

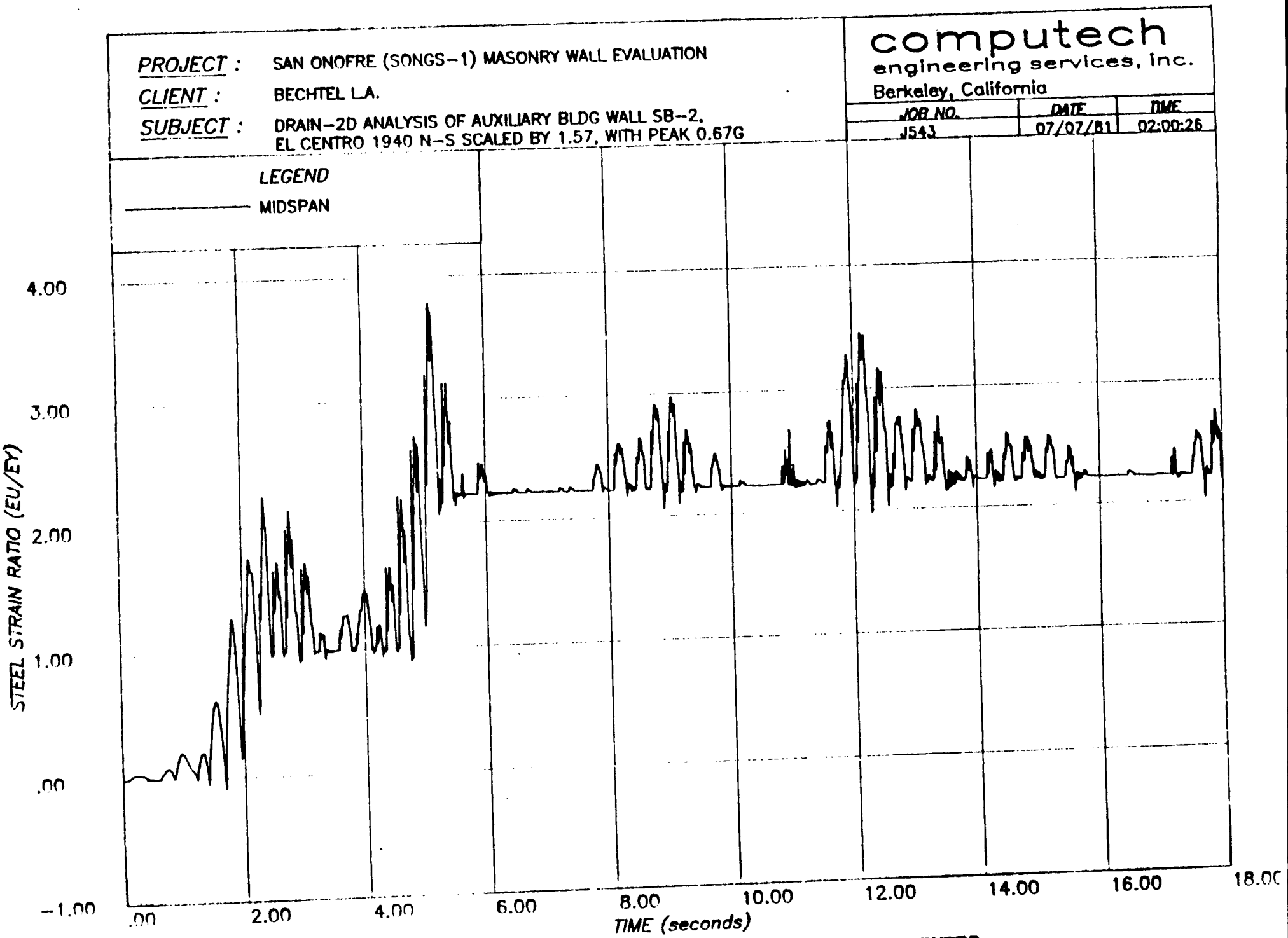


FIGURE 5.13 : REACTOR AUXILIARY BUILDING GROUP II: EL CENTRO

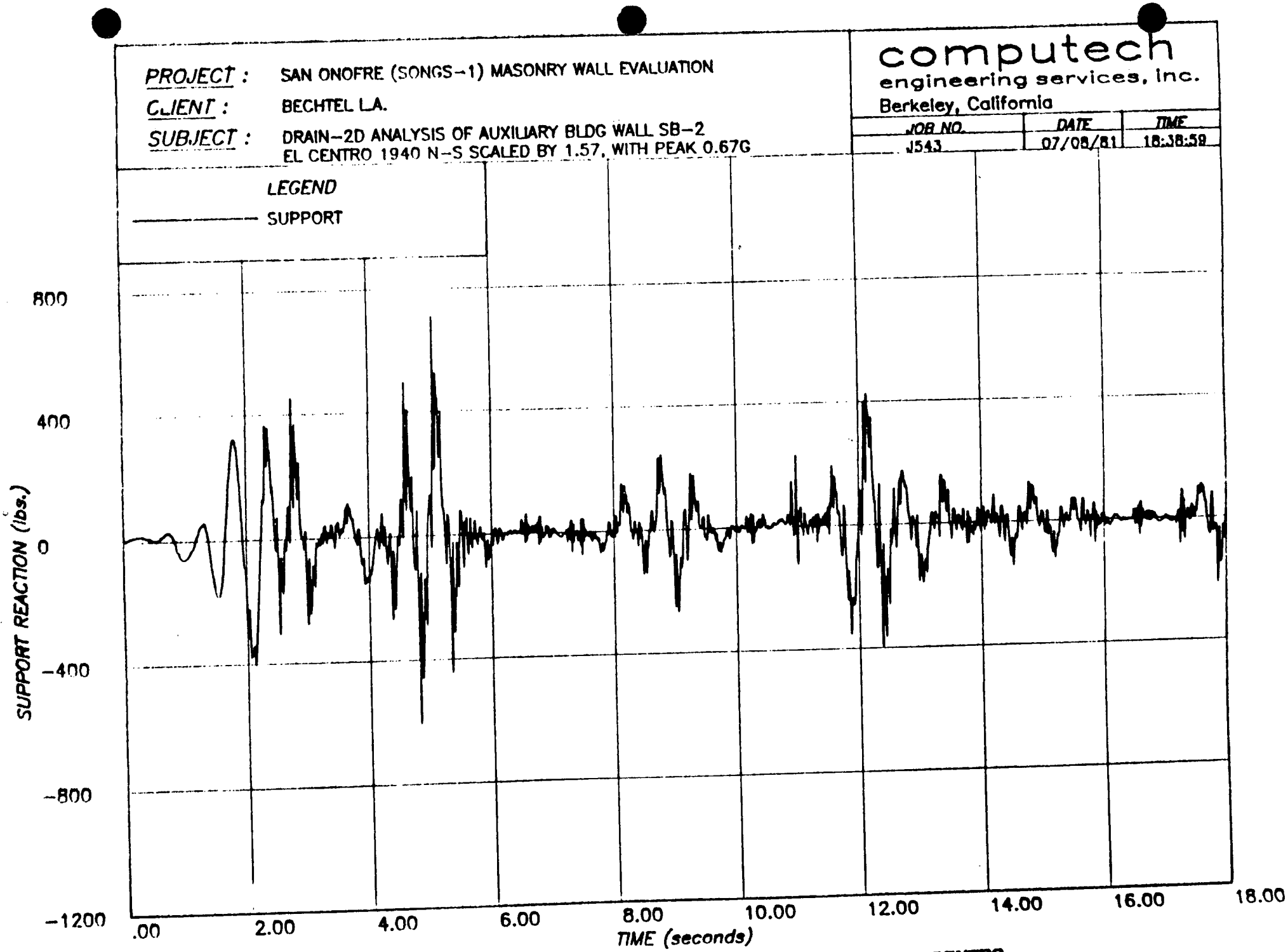


FIGURE 5.14 : REACTOR AUXILIARY BUILDING GROUP II: EL CENTRO

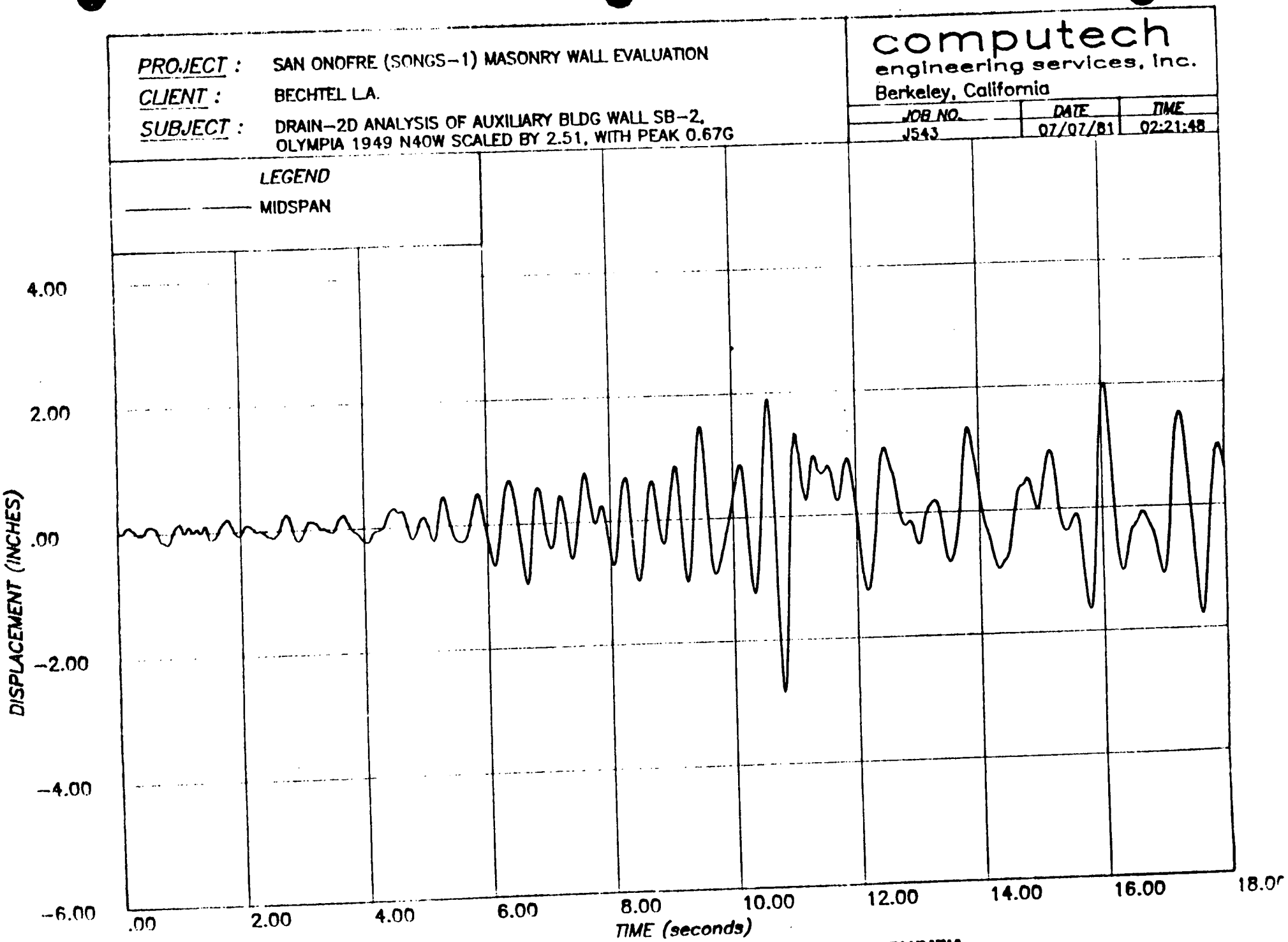


FIGURE 5.15 : REACTOR AUXILIARY BUILDING GROUP II: OLYMPIA

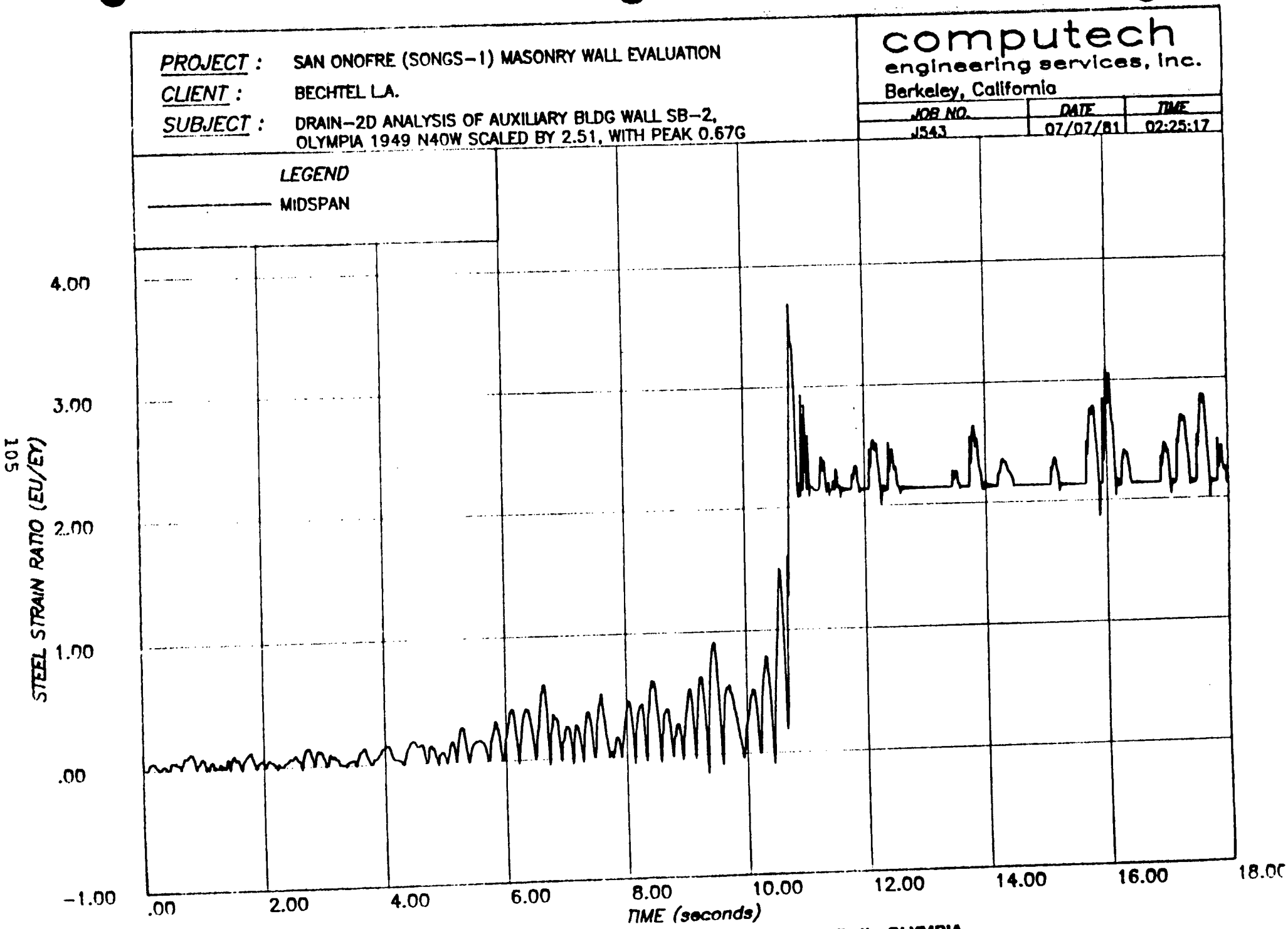


FIGURE 5.16 : REACTOR AUXILIARY BUILDING GROUP II: OLYMPIA

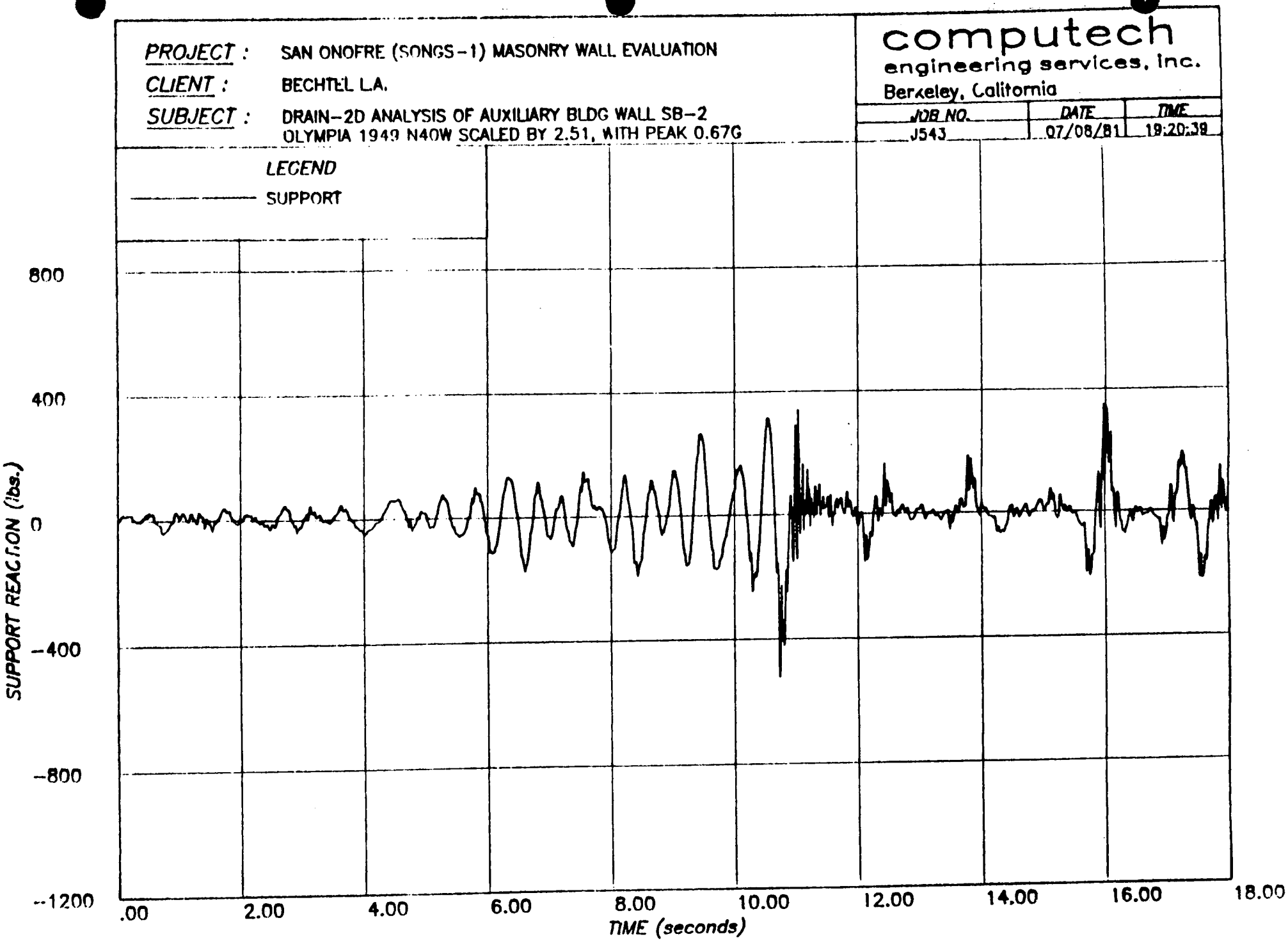


FIGURE 5.17 : REACTOR AUXILIARY BUILDING GROUP II: OLYMPIA

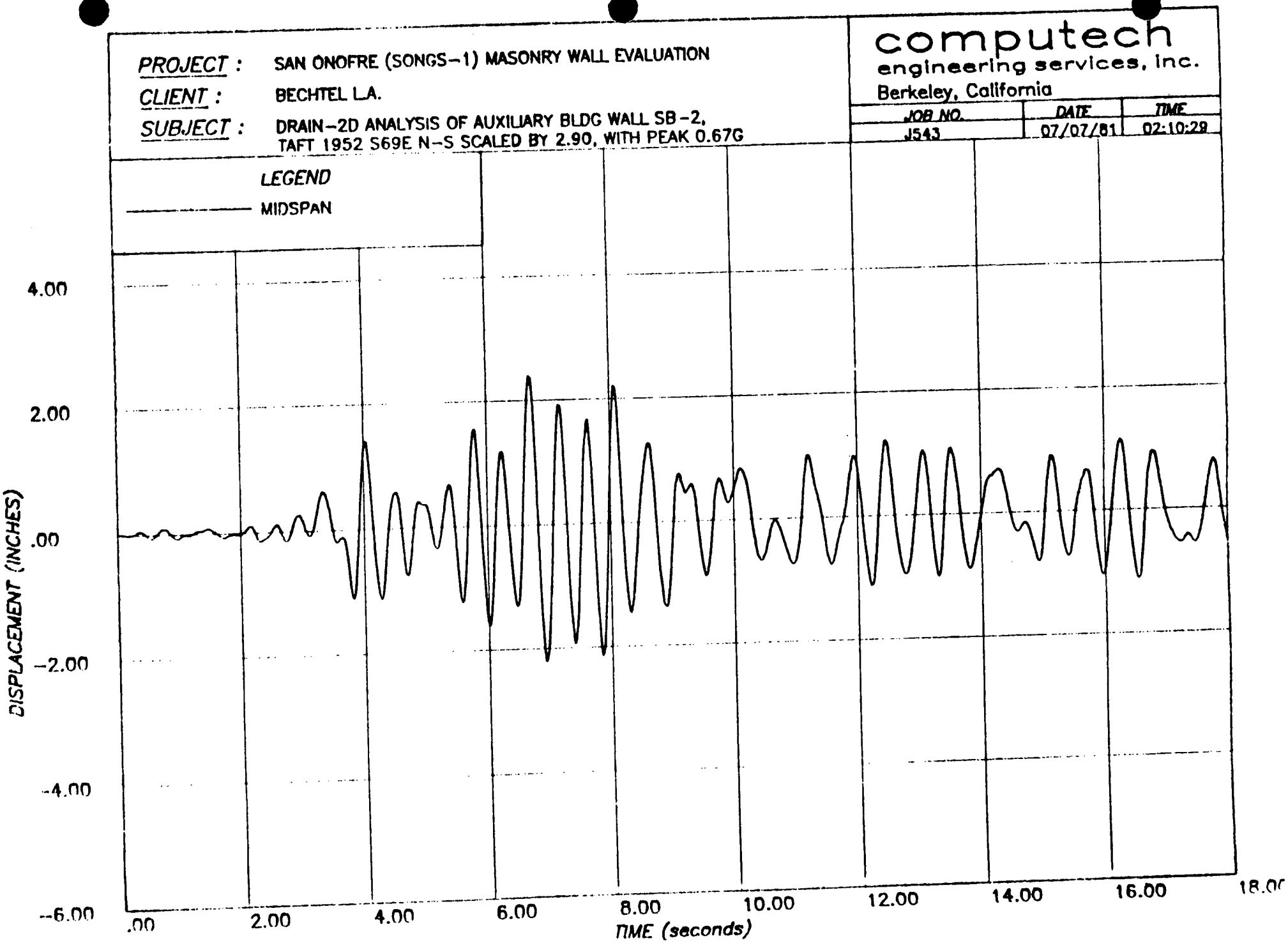


FIGURE 5.18 : REACTOR AUXILIARY BUILDING GROUP II: TAFT

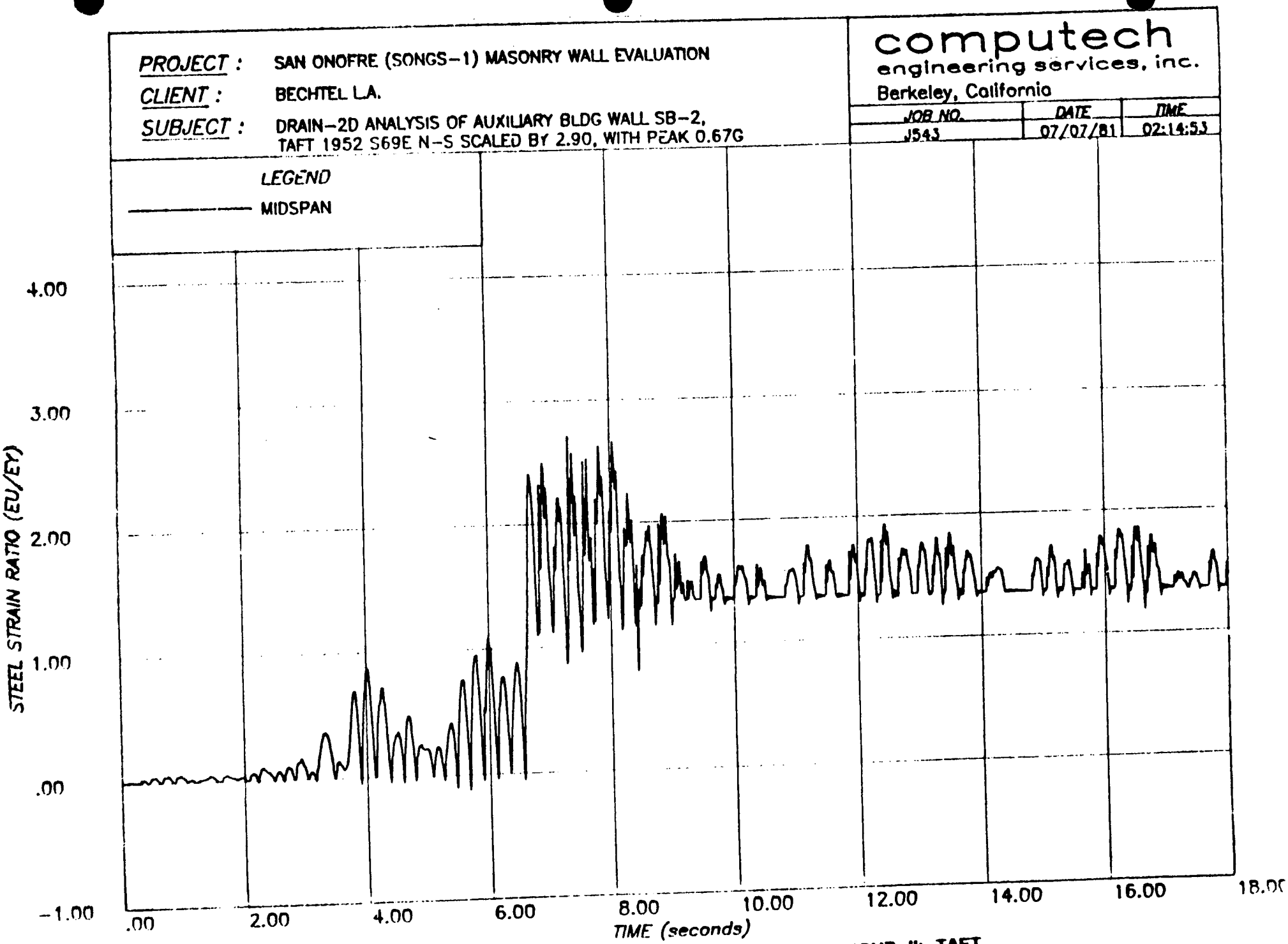


FIGURE 5.19 : REACTOR AUXILIARY BUILDING GROUP II: TAFT

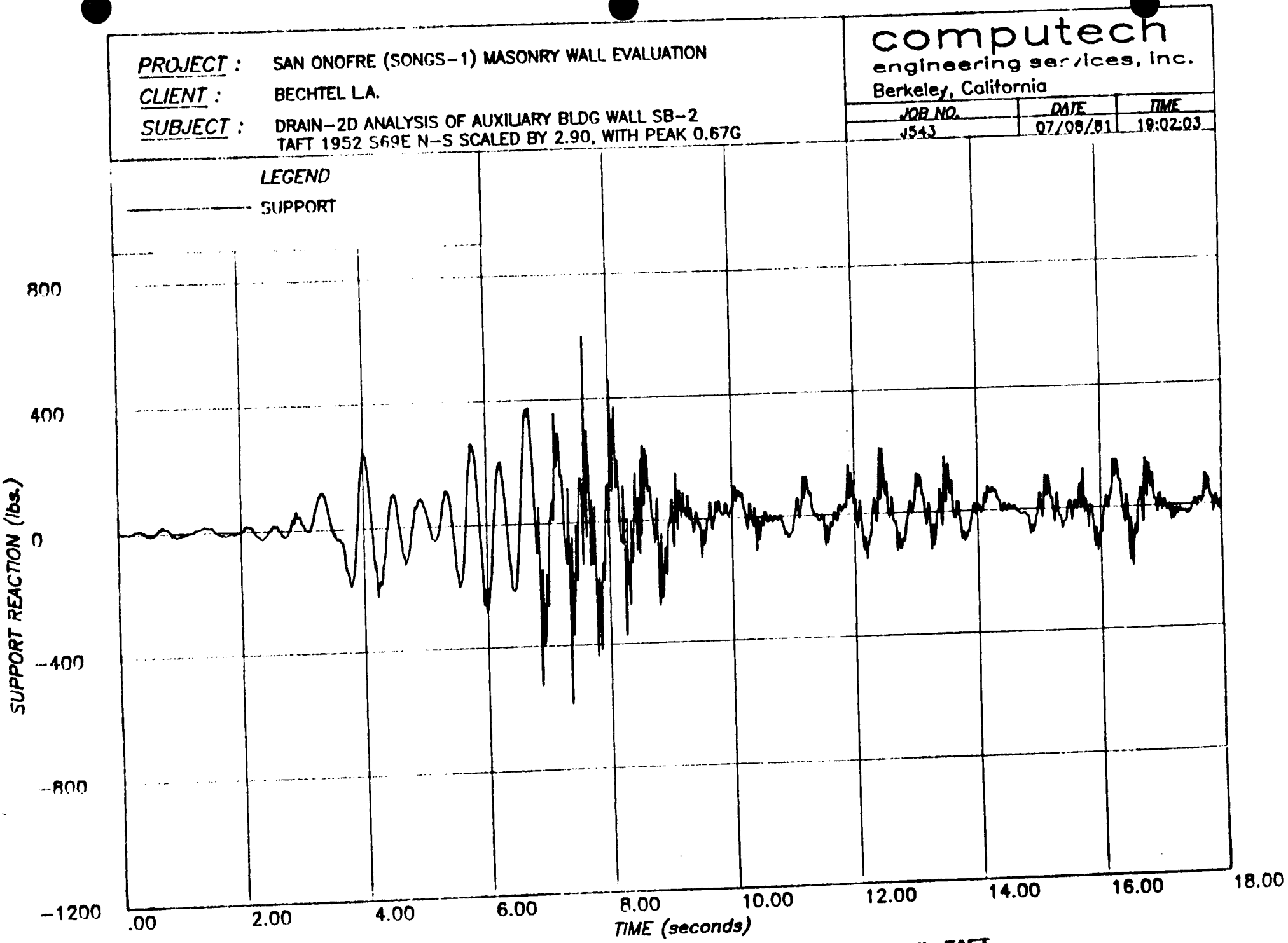


FIGURE 5.20 : REACTOR AUXILIARY BUILDING GROUP II: TAFT

6 CONCLUSIONS

All the masonry walls at the San Onofre. Unit 1 plant for which the preliminary elastic analyses indicated yield may occur were analyzed using inelastic numerical procedures to predict their response to a suite of time history acceleration records. The analyses were carried out in accordance with the methodology developed for San Onofre. Unit 1 and reported in Volume 2 of this report. The evaluation was carried out in terms of the criteria presented in Volume 1 of the report.

In the following sections conclusions based on the evaluation of the walls in each of the four buildings for which walls were analyzed are reported.

6.1 Turbine Building

The walls in this building were divided into three groups for the analysis based on their height. Two groups were comprised of walls supported top and bottom and the third of cantilever walls. Each wall group was analyzed for the worst case of added equipment loads and the former two groups were also analyzed for the case of no added mass. This was effectively an upper and lower bound of the actual wall conditions.

All walls were satisfactory in respect to the maximum steel ductility, masonry compressive stress and wall stability. Reaction forces based on a weighted average approach were provided for use in the check of the connection capacity.

A portion of wall TB-1 has a large bus opening the effect of which is to cause additional loads on the adjacent strips of solid wall. The size of opening relative to the wall length is such that the simplified procedures proposed for evaluating openings were not considered valid. For this wall a more detailed elastic plate analysis was performed to incorporate the effect of both horizontal and vertical spanning. To enable the effect of the opening to be isolated the analysis was repeated without the opening. The opening was found to increase the elastic moments and deflections by about 30%. The inelastic response of the group to which this wall belonged had shown that maximum values were one fifth of the limits set in the criteria. Therefore the effect of the openings on the inelastic response would not cause a five fold increase in the ductility needed to exceed the specified limit.

On the basis of this conclusion and the results of the inelastic analyses all walls in the Turbine Building were concluded to be satisfactory in terms of the San Onofre. Unit 1 criteria.

6.2 Ventilation Equipment Building

The four walls comprising the perimeter of the Ventilation Equipment Building were analyzed using the inelastic methodology as developed and in addition the two longer walls were analyzed elastically to determine the effect of

the openings on the response.

The inelastic time history analyses showed a relatively low level of non-linear response and the walls were rated satisfactory in terms of the criteria. The elastic analyses using the response spectrum method produced an increase in both horizontal and vertical moments because of the presence of the openings but most of the higher moments were in wall areas where the strength was augmented by the trimmer bar reinforcement. In view of the relatively low levels of response the effects of the openings were judged not to adversely effect wall behavior. Therefore all walls were rated satisfactory for out of plane loading.

6.3 Reactor Auxiliary Building

The 6 safety masonry walls analyzed in the Reactor Auxiliary Building ranged from approximately 8 feet to 16 feet in height. Analysis was carried out on twc models representing each of these extremes.

The shorter of the walls analyzed did not yield under the three earthquake records and the extent of inelastic action was moderate for the longer wall. All walls were judged satisfactory in terms of the criteria.