R.E. GINNA NUCLEAR POWER PLANT

REINFORCED MASONRY WALL EVALUATION

EVALUATION OF CONTROL BUILDING REINFORCED WALLS

Prepared for:

ROCHESTER GAS & ELECTRIC CORP.
89 East Avenue
Rochester, New York 14649

Prepared by:

COMPUTECH ENGINEERING SERVICES, INC.
2855 Telegraph Ave., Suite 410
Berkeley, California 94705

REPORT NO. R562-N4
December 16, 1985

TABLE OF CONTENTS

1 INTRODUCTION

An evaluation of the reinforced masonry walls in the Control Building of the R. E. Ginna Nuclear Plant (Ginna) has been performed. The scope of this evaluation was described in a letter from RG&E (Mr Roger W Kober) to the NRC (Mr. John A Zwolinski) dated November 6, 1985. The scope of work defined the following:

1. Validation of the inelastic analysis methodology by means of correlation studies using the results of the SONGS-1 test program and actual recorded input motions and material properties. Actual yielding rebar length and rebar eccentricities were also accounted for in the revised model.

2. Elastic analysis of the twelve reinforced walls in the Control Building.

3. Inelastic analysis of the worst-case spanning wall and cantilever wall. An additional spanning wall was also analyzed inelastically to provide information for interpolating to the remaining walls.

4. An assessment of the margins of safety of additional walls for which inelastic behavior was indicated was made.

The correlation studies in item 1 above have been completed and the results reported in Report R562-N3.

2 ELASTIC ANALYSES OF WALLS

The elastic analyses were performed using procedures and criteria developed by CES and used on numerous masonry wall evaluations. The steps involved in the elastic analysis of each wall were as follows:

1. A finite element model of each wall was generated using plate bending elements with the uncracked properties of the walls.

2. A response spectrum analysis was performed using the envelope spectrum appropriate to the wall and maximum masonry mortar tension stresses obtained.

3. In regions where the tension exceeded the cracking strength of the mortar the model was modified to used cracked wall properties based on the transformed section.

4. The response spectrum analysis was repeated and iterations performed if necessary until no further cracking was indicated.

The final masonry and steel stresses were then tabulated and compared with the allowable stress criteria. For the elastic analysis excerpts from the criteria describing materials and allowable stresses are as follows:

Stresses and Strains

Allowable stresses were based on the following compressive strengths:

Hollow Concrete Units		f'_m =	1180 psi
Mortar		m_o =	750 psi
Steel :	Vertical	F_y =	40 ksi
	Horizontal	F_y =	70 ksi

Uncracked, Elastic

Where the results of an elastic analysis showed the stress to be below the following value throughout, then the wall was considered uncracked and thus

acceptable:

Masonry Tension f_t = 22.7 psi normal to bed joint
= 45.5 psi parallel to bed joint

Cracked, Elastic

Where the results of a cracked elastic analysis showed the stresses to be below the following values thoughout then the wall was considered acceptable:

Masonry	f_c =	1003 psi
Vertical Steel	f_s =	36 ksi
Horizontal Steel	f_s =	63 ksi

The Ginna reinforced walls are reinforced with grouted vertical bars and with joint reinforcing horizontally. Joint reinforcing is generally regarded as crack control reinforcement and there is some doubt as to its adequacy to act as primary reinforcing to ensure plate action in walls such as these. In a number of the initial analyses high stresses were indicated in this joint reinforcing. Therefore, a final run was made assuming that the joint reinforcing was not effective in enforcing horizontal spanning between the side supports of the walls. For this final run the side supports were removed and the walls spanned vertically only. Note that the joint reinforcing was still considered effective within the wall span, i.e. in distributing loads above openings and ensuring a vertically spanning plate rather than a series of vertical beams as would occur if the reinforcing was completely neglected.

These final analyses are considered to represent a conservative evaluation of the walls as they determine the maximum forces in the reinforcing and masonry if the wall resists all seismic loads by vertical spanning if the efficiency of the joint reinforcing is reduced due to high stresses.

The final masonry and steel stresses in each wall and the ratio of these stresses to the criteria limit are listed in Table 2.1. Table 2.2 lists the frequency of each wall both in its initial uncracked stage and in the final analysis using transformed properties.

The maximum masonry stresses and the stresses in the joint reinforcing are in all cases lower than the criteria limit. The stresses in the vertical steel exceeded the criteria limit in all 7 spanning walls and in 3 of the 5 cantilever walls. One cantilever wall did not crack and so met the criteria for unreinforced

walls.

The stress ratios in the vertical rebar of the yielding walls ranged from 1.02 to 2.18. Note that the criteria limit on stress is 0.9 times the minimum specified yield stress for the rebar of 40 ksi and so ratios less than (40/36)=1.11) indicate that although the allowable stress is exceeded the rebar is unlikely to actually yield. This applies to Wall 973-3C.

WALL I.D.	MAXIMUM STRESSES				MAXIMUM STRESS RATIOS			
	HORIZONTAL		VERTICAL		HORIZONTAL		VERTICAL	
	Masonry	Steel	Masonry	Steel	Masonry	Steel	Masonry	Steel
SPANNING WALLS								
971-1C	82	19.0	677	78.3	0.09	0.30	0.67	2.18
971-2C	77	17.8	598	69.2	0.17	0.28	0.60	1.92
971-4C	61	14.0	677	55.3	0.06	0.22	0.67	1.54
971-5C	57	13.0	537	62.2	0.06	0.20	0.54	1.73
971-6C	62	14.3	389	45.1	0.06	0.23	0.39	1.25
972-4C/5C	107	24.6	558	64.5	0.11	0.39	0.55	1.79
972-6C	92	20.9	616	71.3	0.09	0.33	0.61	1.98
CANTILEVER WALLS								
972-1C	(1)							
972-2C	57	13.1	513	51.3	0.06	0.20	0.51	1.43
972-3C	84	6.9	337	33.7	0.09	0.11	0.33	0.94
973-3C	40	7.7	450	36.6	0.04	0.12	0.45	1.02
973-4C	92	17.8	579	47.3	0.09	0.28	0.58	1.31

NOTES:

1. Wall 972-1C did not crack.
2. Stresses are in psi for masonry and ksi for steel.
3. Maximum allowable stresses after cracking are assumed to be 0.85f'm for masonry and 0.9Fy for steel. The provides 1003 psi for masonry, 63 ksi for Dur-O-Wal and 36 ksi for vertical steel.

TABLE 2.1 : GINNA CONTROL BUILDING WALLS : ELASTIC ANALYSIS

3 INITIAL ASSESSMENT

From the elastic analyses the 7 spanning walls had stresses in the vertical rebar exceeding the criteria limit by ratios ranging from 1.25 to 2.18. Therefore, all walls would require to be qualified by the inelastic analysis methodology.

One of the cantilever walls did not crack and so is qualified by unreinforced masonry wall criteria. One other cantilever wall was stressed to below the criteria limits. The remaining three walls had stresses in the vertical rebar exceeding the criteria limits by factors ranging from 1.02 to 1.43.

In accordance with the scope of work, one spanning wall and one cantilever wall were selected for detailed non-linear analysis. Of the spanning walls, Wall 971-1C had the highest stress ratio and, as all walls had similar construction details and heights, this wall was selected for the detailed analysis. Of the three yielding cantilever walls, Wall 972-1C had the highest ratio of the 8" thick walls (1.43) and 973-4C had the highest of the 6" thick walls (1.31). Although the latter wall had a slightly lower stress ratio this wall was selected for the cantilever detailed analysis because of its lower thickness. The 6" wall would prove most critical should any instability due to deflections occur.

Wall 971-6C also had stresses exceeding yield but to a lesser extent than Wall 971-1C and so this wall was also selected for an inelastic analysis to provide a basis for interpolation to the remaining walls.

4 INELASTIC ANALYSES OF WALLS

The inelastic analyses followed the procedures of the CES methodology as refined as a result of the correlation studies as documented in Report R562.N3. The procedure required the development of a detailed model including each masonry block and each joint of the wall, derivation of the equivalent sub-structure model, analysis of the sub-structure model and finally back-substitution to obtain the final evaluation parameters.

The appropriate section of the criteria defining the acceptability of the wall in its yielded state is as follows:
Cracked, Inelastic For walls evaluated based on an inelastic, dynamic analysis, the following material limits applied.

Masonry Strain $E_m = 0.003$

Vertical Steel Strain Ratio $E_s/E_y = 45$

The cantilever wall analyzed, Wall 973-4C, has two layers of vertical rebars rather than being centrally reinforced as for the spanning wall. This produces a force-deflection curve which is more closely related to the "typical" hysteresis for concrete beam type members. For this reason the hysteresis for centrally reinforced members with zero stiffness on reversing cycles was not appropriate. Therefore, the detailed model was itself used for the dynamic analyses and the evaluation parameters obtained directly from the analyses.

The dynamic analyses used direct step-by-step integration and used acceleration time histories as input. To obtain these time histories the CES spectral matching program SYNTH was used. This program uses an actual recorded time history as a starting point and successively iterates on the Fourier coefficients until the response spectrum of the time history provides a close match to the target spectrum. This program was used to generate three time histories for each floor spectrum, based respectively on the starting time histories of El Centro, 1940, Olympia and Taft earthquakes. For the spanning walls motions appropriate to the spectra at both the top and bottom of the wall were used. For the cantilever wall only the motion for the spectrum at the base of the wall was used.

The following sub-sections describe the analyses performed on each of the walls analyzed inelastically and the results obtained from these analyses.

4.1 Spanning Wall 971-1C

Wall 971-2C is a 16'-10" high wall 38'-1" long between elevations 253'-8" and 271'-0" in the Control Building. As is typical of all spanning walls it is reinforced vertically with #3 bars at 32" centers and horizontally with DUR-O-WALL joint reinforcing.

The detailed model represented all 25 courses of the wall and each masonry joint. The force deflection behavior of this model is as shown on Figure 4.1. Also shown on Figure 4.1 is the corresponding non-linear curve for the substructure model. The correlation between the two models is excellent for deflections up to 3" with slight variations beyond this level.

The time history of the analysis of this wall using the time histories based on the El Centro, Olympia and Taft time histories respectively are as shown in Figures 4.2, 4.3 and 4.4. As the correlation studies has shown that the SONGS-1 walls were sensitive to any rebar eccentricity the El Centro analysis was repeated using eccentricities of 0.4" and 0.8", the latter value being the maximum value used for SONGS-1. The displacement time histories of these analyses are shown in Figures 4.5 and 4.6.

The results for all 5 analyses are summarized in Table 4.1. The important parameters are as follows:

1. Maximum displacement of the wall, which ranged from a minimum of 1.72" to a maximum of 2.38".

2. Masonry strain ratio, at the base and upper hinge position. The strains varied over a relatively small range of 0.0011 to 0.0013. The criteria limit on this parameter is 0.0030.

3. Steel strain ratios reached a maximum of 3.7 at the upper hinge and 6.2 at the lower hinge. The limiting value on the strain ratio is 45.

4. The extent of yielding at the base was over one to two blocks (8" to 16") and over three to five blocks at the upper hinge (24" to 40").

5. The maximum gaps at the mortar joints were 0.08" at the upper hinge and 0.14" at the upper hinge.

The maximum difference in response due to the three different earthquake records used was approximately 15% as measured by maximum displacements. Therefore the walls are relatively insensitive to this parameter. Increasing eccentricity of the rebar tended to increase the response with a maximum effect of 27% on the mid-height displacements and lesser effects on the detailed material parameters (20% on steel strain ratios, 10% on masonry strains.

4.2 Spanning Wall 971-6C

The analysis of Wall 971-6C followed the same steps as for Wall 971-1C. The walls are of similar construction and at the same elevation and the major differences are in the amount of openings, which affected the properties up the height of the wall, and the seismic input which was N-s for 971-6C.

The displacements for the analyses using the input motions based on the El Centro, Olympia and Taft record are shown in Figures 4.7, 4.8 and 4.9 respectively. Note that the earthquakes are the starting time histories which are frequency scaled to match the different floor spectra. Therefore, the El Centro based motion, for example, differs for the analyses for walls 1-1C and 1-6C.

Table 4.2 summarizes the major results and material parameters for the three analyses. The maximum displacement, 2.20", was approximately the same as for Wall 1-1C and the other parameters were of similar magnitude and well below the criteria limits of 0.0020 for masonry strain and 45 for steel strain ratio. For this wall the response for the El Centro based record was somewhat less than for the other two records, and in fact yielding occurred only at the base hinge position and not within the height of the wall as happened for the other two records.

4.3 Cantilever Wall 973-4C

As discussed above, the cantilever walls have a different hysteresis shape

because of the two layers of rebar and so the detailed model rather than the sub-structure was used for the non-linear analyses. This model was analyzed for three different time histories frequency scaled using the same three starting time histories as for the previous analyses. To account for uncertainties in rebar placement two eccentricities were considered:

1. Considering each bar to be placed against the inside of the face shell rather than with 0.5" cover.

2. Assuming one of the two bars to be misplaced by 0.5" towards the center of the grouted cell.

The El Centro based analysis was repeated using these two eccentricities. Figures 4.10, 4.11 and 4.12 show the displacement time histories of the top of the cantilever walls for the El Centro, Olympia and Taft based analyses respectively. Figures 4.13 and 4.14 are the displacements for the El Centro record using the two eccentricities listed above.

Table 4.3 summarizes the important parameters from the analyses. Maximum displacements ranged from 2.95" to 3.59", slightly over one-half the displacement of 6" which would be the stability limit of a 6" wall. The maximum compressive strains of 0.0017 were somewhat higher than those of the spanning walls even though the gap widths (maximum 0.05") were smaller. This increased strain was caused by the smaller lever arm (6" wall versus 8" wall) and by the higher tensile force of two bars which was equilibrated by the face shell compression. The higher lever arm reduced the maximum steel strain ratios which reached maximum values of 4.2. In all cases the rebar yielded only through the first block, reflecting the steepness of the bending moment diagram at the base location.

RESPONSE	EL CENTRO BASED RECORD			OLYMPIA BASED RECORD	TAFT BASED RECORD
	ECC = 0"	ECC = 0.4"	ECC = 0.8"		
CENTER DISPLACEMENT					
Maximum (Inches)	1.82	1.88	2.38	1.72	1.98
MASONRY COMPRESSIVE STRAIN					
Mid-Height	0.0011	0.0011	0.0012	0.0011	0.0011
Base	0.0013	0.0013	0.0013	0.0013	0.0013
STEEL STRAIN RATIOS					
Mid-Height	2.2	2.5	3.7	1.9	2.3
Base	5.1	5.6	6.2	4.8	5.5
LENGTH OF YIELDING REBAR (Inches)					
Mid-height	32	40	40	24	32
Base	8	8	816	8	8
MAXIMUM GAP OPENING					
Mid-Height (Inches)	0.05	0.06	0.08	0.05	0.05
Base (Inches)	0.13	0.12	0.12	0.12	0.14

TABLE 4.1 : RESULTS FOR WALL 971-1C

RESPONSE PARAMETER	EL CENTRO BASED RECORD	OLYMPIA BASED RECORD	TAFT BASED RECORD
CENTER DISPLACEMENT			
Maximum (inches)	1.26	1.89	2.20
MASONRY COMPRESSIVE STRAIN			
Mid-Height	0.0011	0.0011	0.0012
Base	0.0012	0.0013	0.0013
STEEL STRAIN RATIOS			
Mid-Height	-	2.5	2.9
Base	3.8	5.8	6.5
LENGTH OF YIELDING REBAR (inches)			
Mid-height	-	40	40
Base	8	8	8
MAXIMUM GAP OPENING			
Mid-Height (inches)	0.02	0.06	0.07
Base (inches)	0.09	0.14	0.16

TABLE 4.2 : RESULTS FOR WALL 971-6C

RESPONSE	EL CENTRO BASED RECORD			OLYMPIA BASED RECORD	TAFT BASED RECORD
	ECC = 0"	ECC = 0.375"	ECC = 0.50"		
TOP DISPLACEMENT Maximum (inches)	2.95	3.23	3.22	3.50	3.59
MASONRY COMPRESSIVE STRAIN Base	0.0014	0.0017	0.0013	0.0016	0.0017
STEEL STRAIN RATIOS Base	2.2	2.5	1.3	3.0	4.2
LENGTH OF YIELDING REBAR (inches) Base	8	8	8	8	8
MAXIMUM GAP OPENING Base (inches)	0.03	0.03	0.02	0.04	0.05

TABLE 4.3 : RESULTS FOR WALL 973-4C

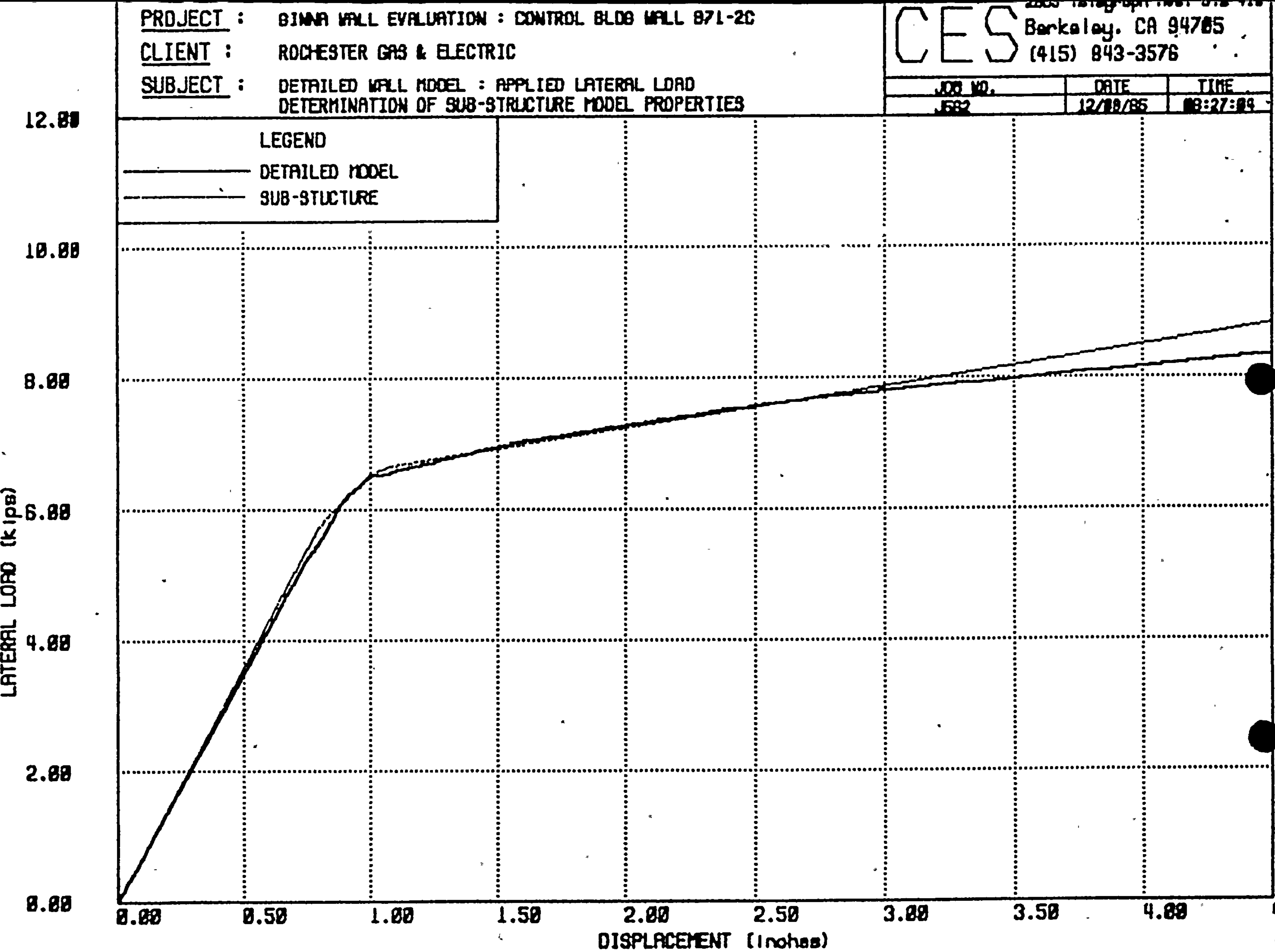

FIGURE 4.1 : WALL 971-1C : FORCE-DEFLECTION RELATIONSHIP

CES

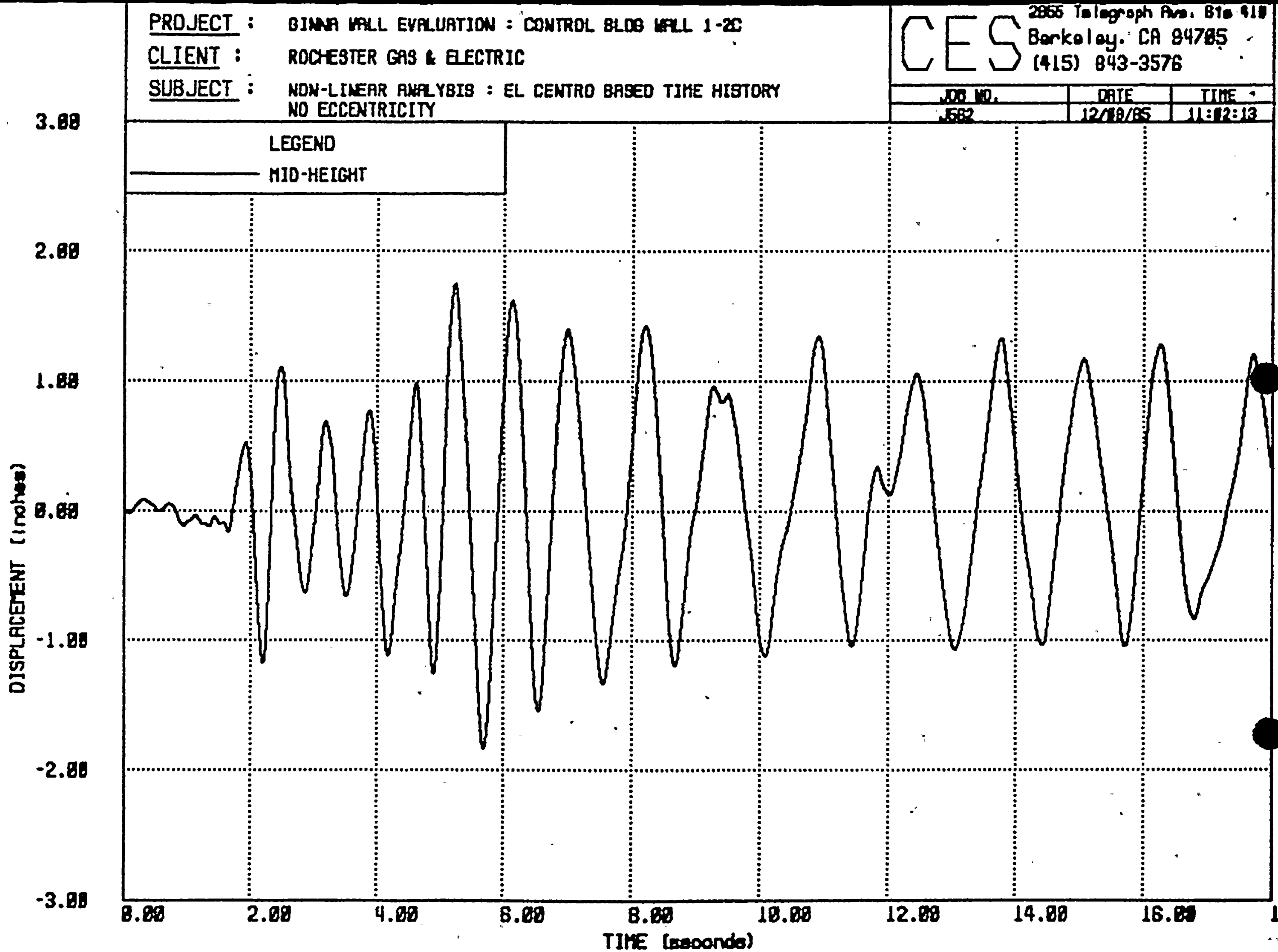

FIGURE 4.2 : WALL 971-1C : EL CENTRO BASED RECORD, NO ECCENTRICITY

R562-N4

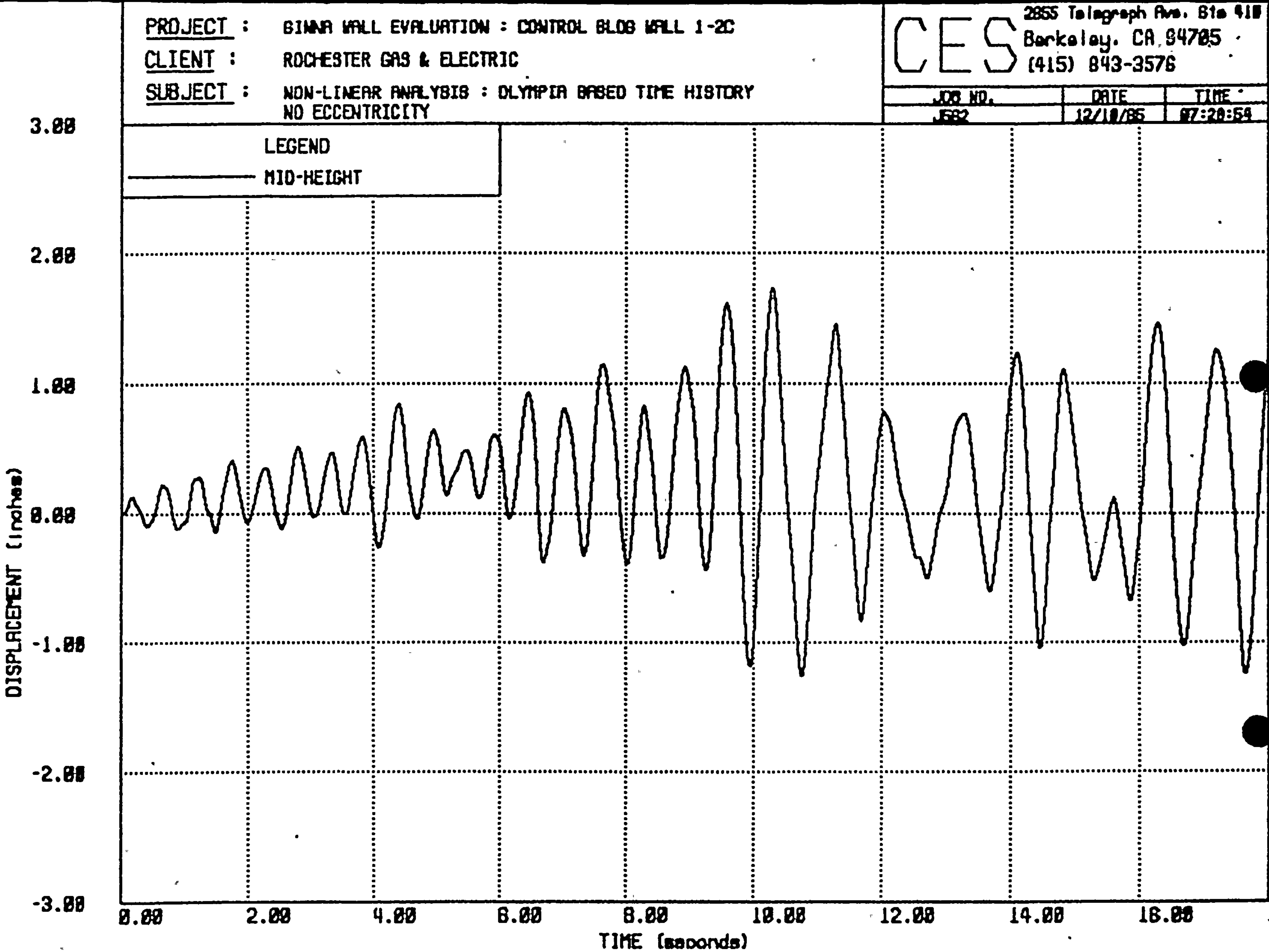

FIGURE 4.3 : WALL 971-1C : OLYMPIA BASED RECORD, NO ECCENTRICITY

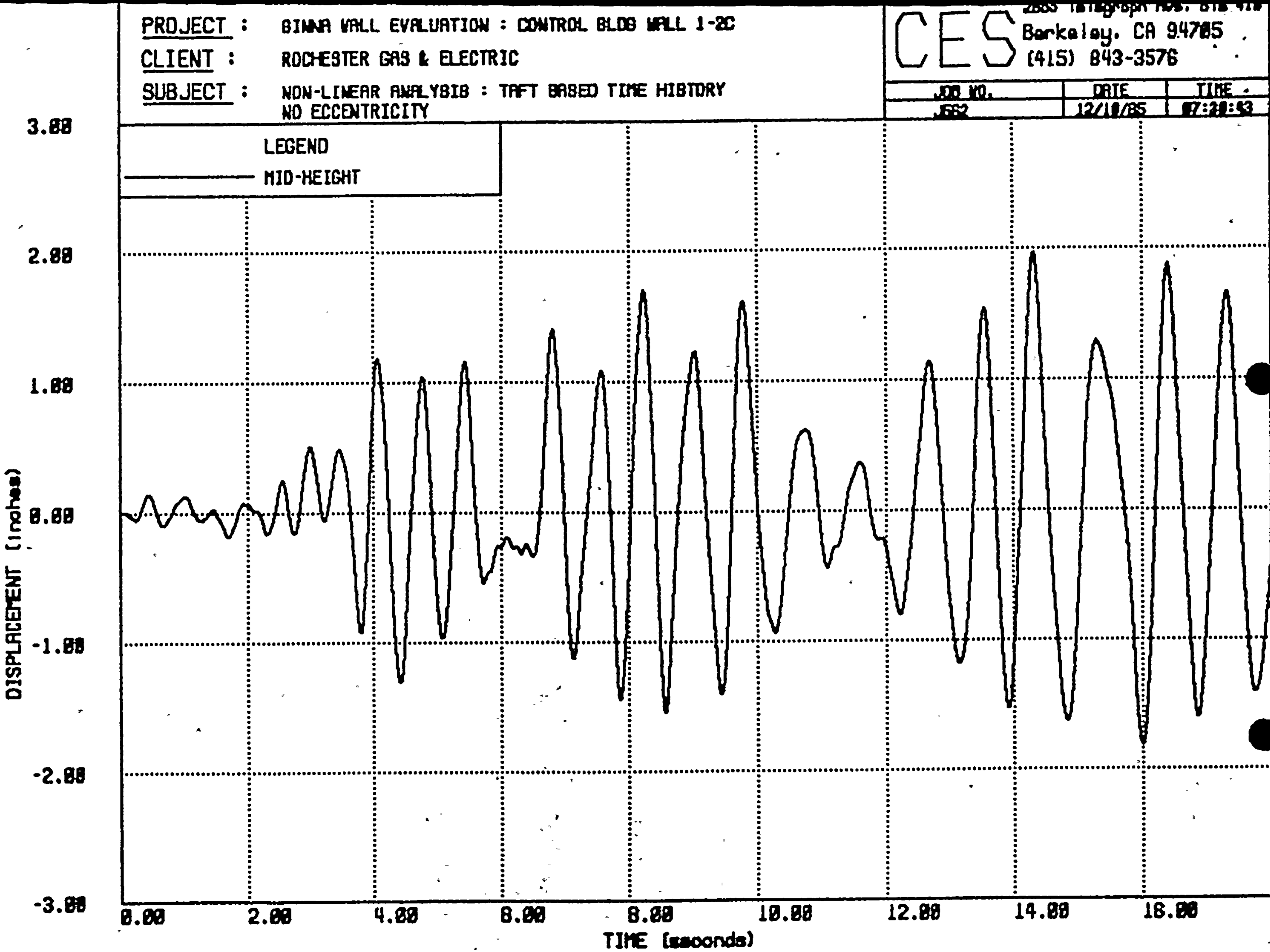

FIGURE 4.4 : WALL 971-1C : TAFT BASED RECORD, NO ECCENTRICITY

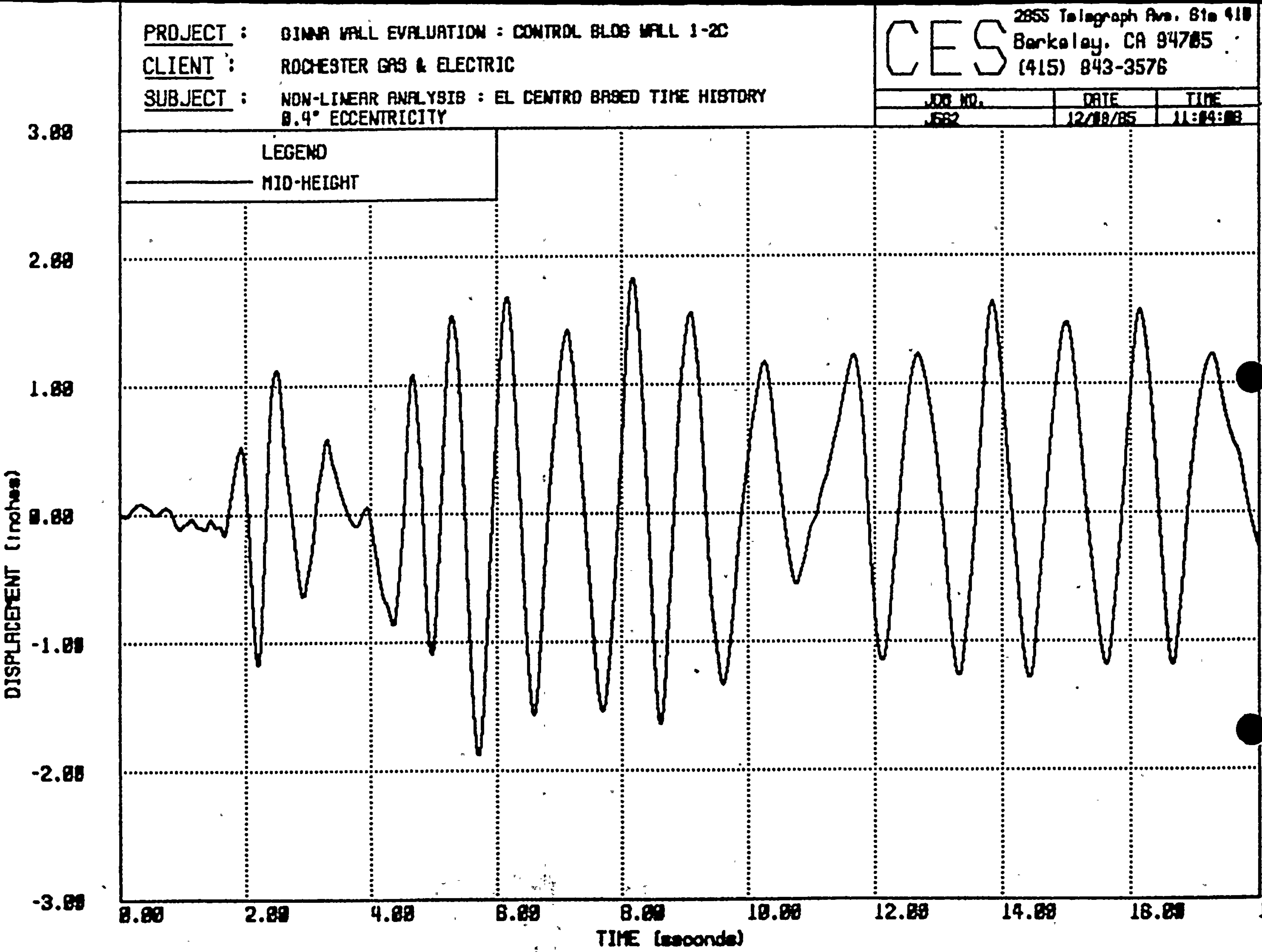

FIGURE 4.5 : WALL 971-1C : EL CENTRO BASED RECORD, 0.4° ECCENTRICITY

CES

- 18 -

R562-N4

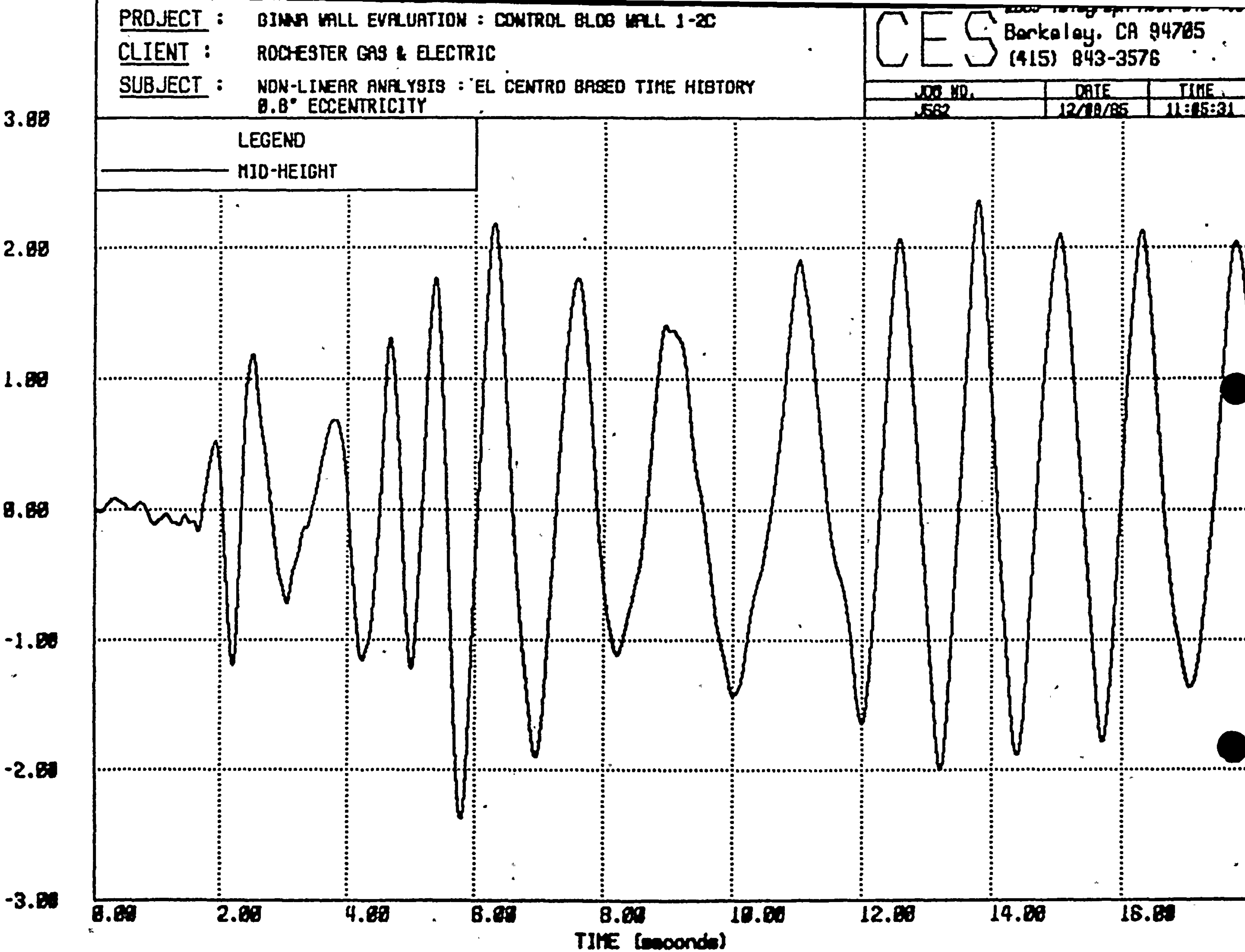

FIGURE 4.6 : WALL 971-1C : EL CENTRO BASED RECORD, 0.8" ECCENTRICITY

PROJECT : BINNA WALL EVALUATION : CONTROL BLDG WALL 1-6C
CLIENT : ROCHESTER GAS & ELECTRIC
SUBJECT : NON-LINEAR ANALYSIS : EL CENTRO BASED TIME HISTORY
NO ECCENTRICITY

CES
Berkeley, CA 94705
(415) 843-3576

JOB NO.	DATE	TIME
J562	12/12/85	10:38:12

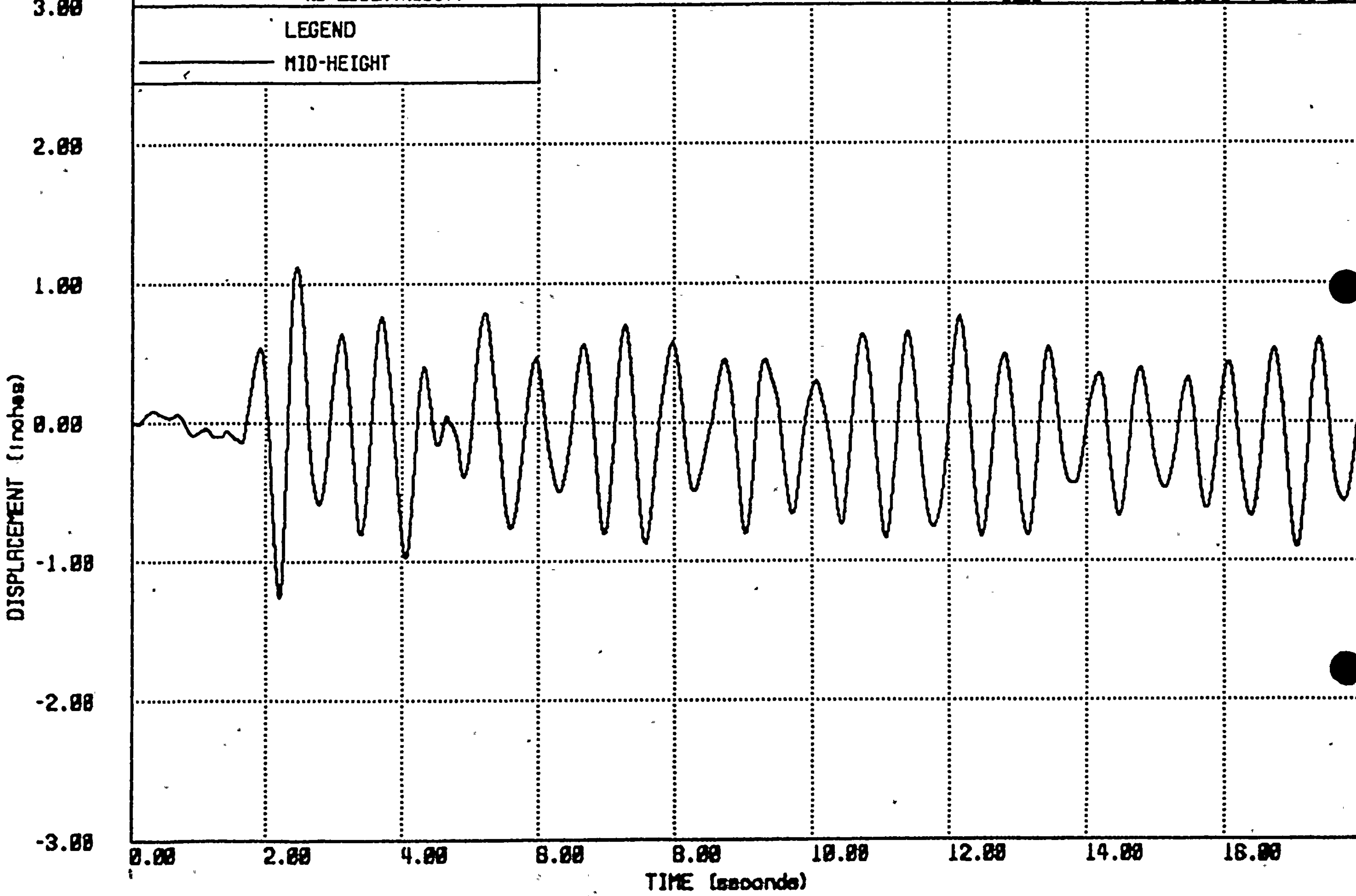

FIGURE 4.7 : WALL 971-6C : EL CENTRO BASED RECORD, NO ECCENTRICITY

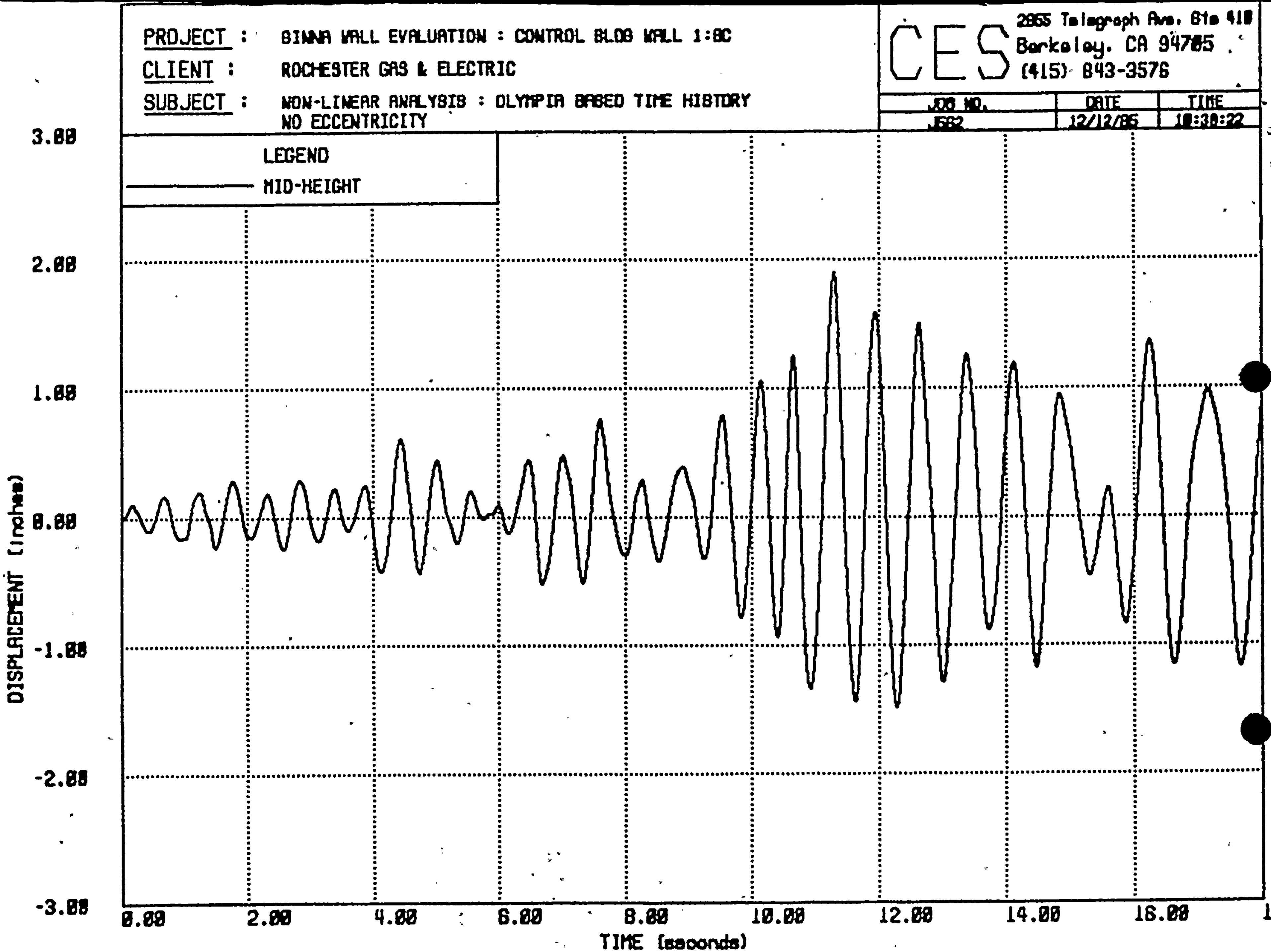

FIGURE 4.8 : WALL 971-6C : OLYMPIA BASED RECORD, NO ECCENTRICITY

CES

- 21 -

R562-N4

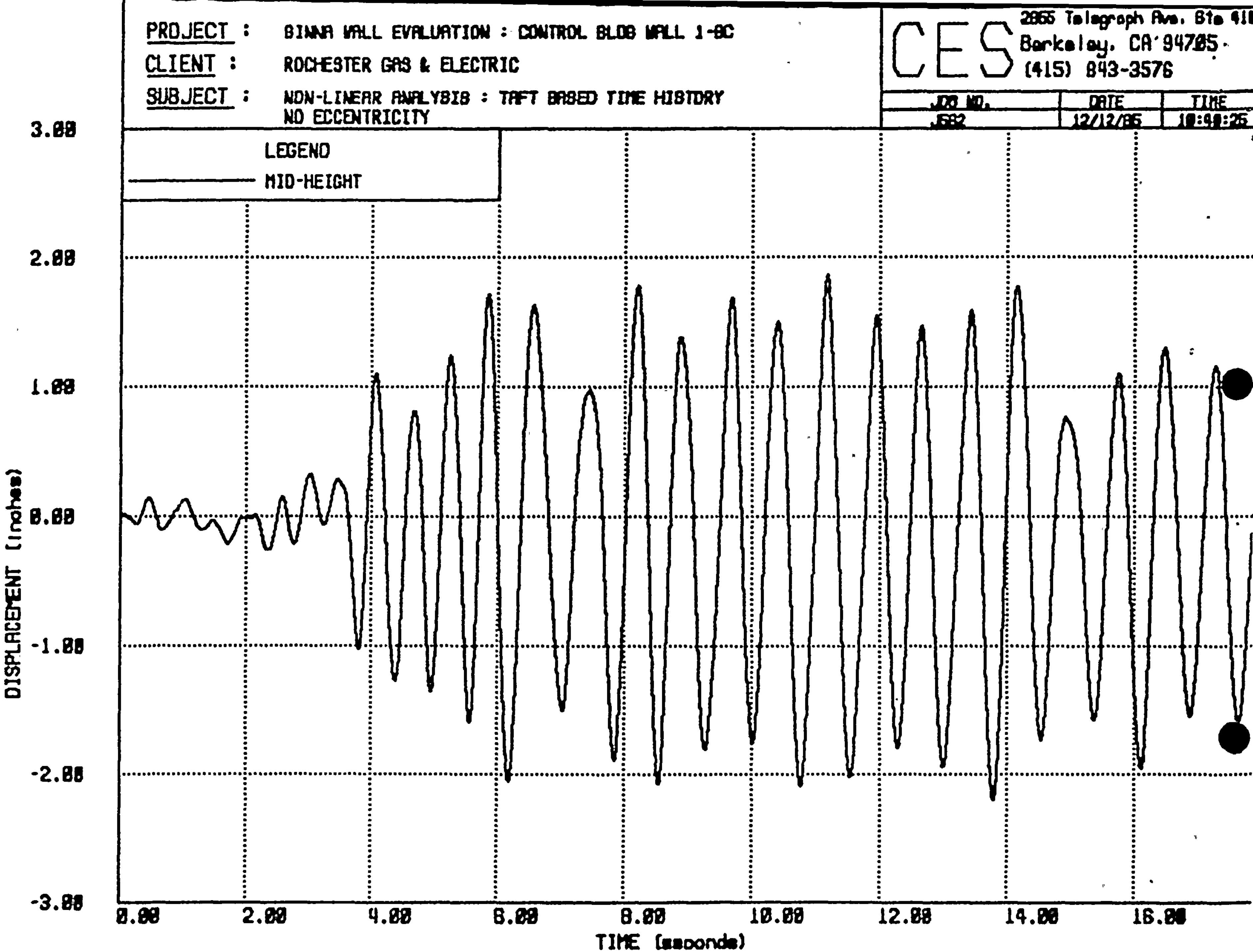

FIGURE 4.9 : WALL 971-6C : TAFT BASED RECORD, NO ECCENTRICITY

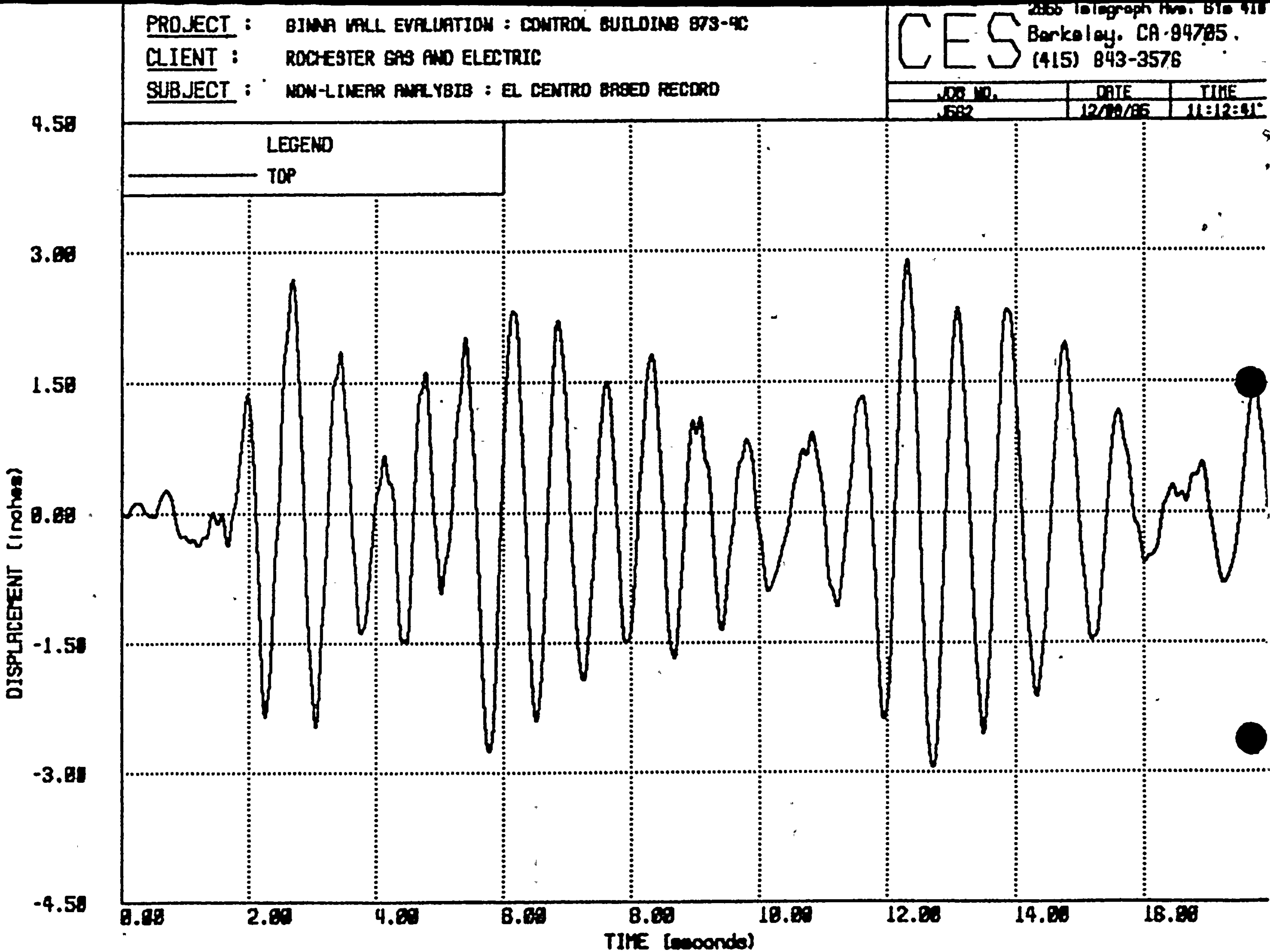

FIGURE 4.10 : WALL 973-4C : EL CENTRO BASED RECORD, NO ECCENTRICITY

CES

R562-N4

PROJECT : SINNA WALL EVALUATION : CONTROL BUILDING 973-4C
CLIENT : ROCHESTER GAS AND ELECTRIC
SUBJECT : NON-LINEAR ANALYSIS : OLYMPIA BASED RECORD

CES
2855 Telegraph Ave. Ste 410
Berkeley, CA 94705
(415) 843-3576

JOB NO.	DATE	TIME
J582	12/10/85	15:28:30

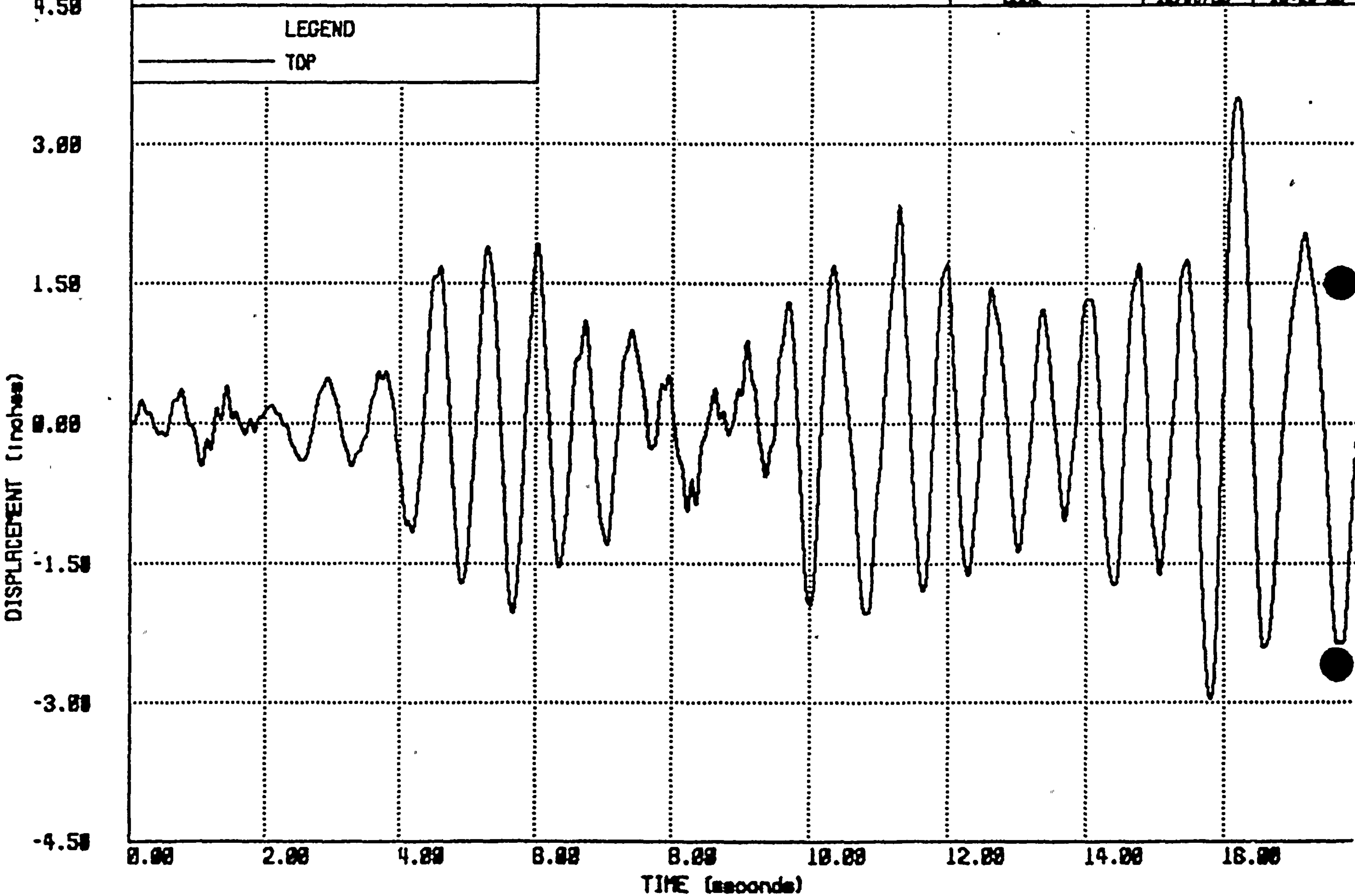

FIGURE 4.11 : WALL 973-4C : OLYMPIA BASED RECORD, NO ECCENTRICITY

PROJECT : BINNA WALL EVALUATION : CONTROL BUILDING 973-4C
CLIENT : ROCHESTER GAS AND ELECTRIC.
SUBJECT : NON-LINEAR ANALYSIS : TAFT BASED RECORD

CES
2855 Telegraph Ave. Ste 410
Berkeley, CA 94705
(415) 843-3576

JOB NO.	DATE	TIME
J562	12/10/85	07:23:31

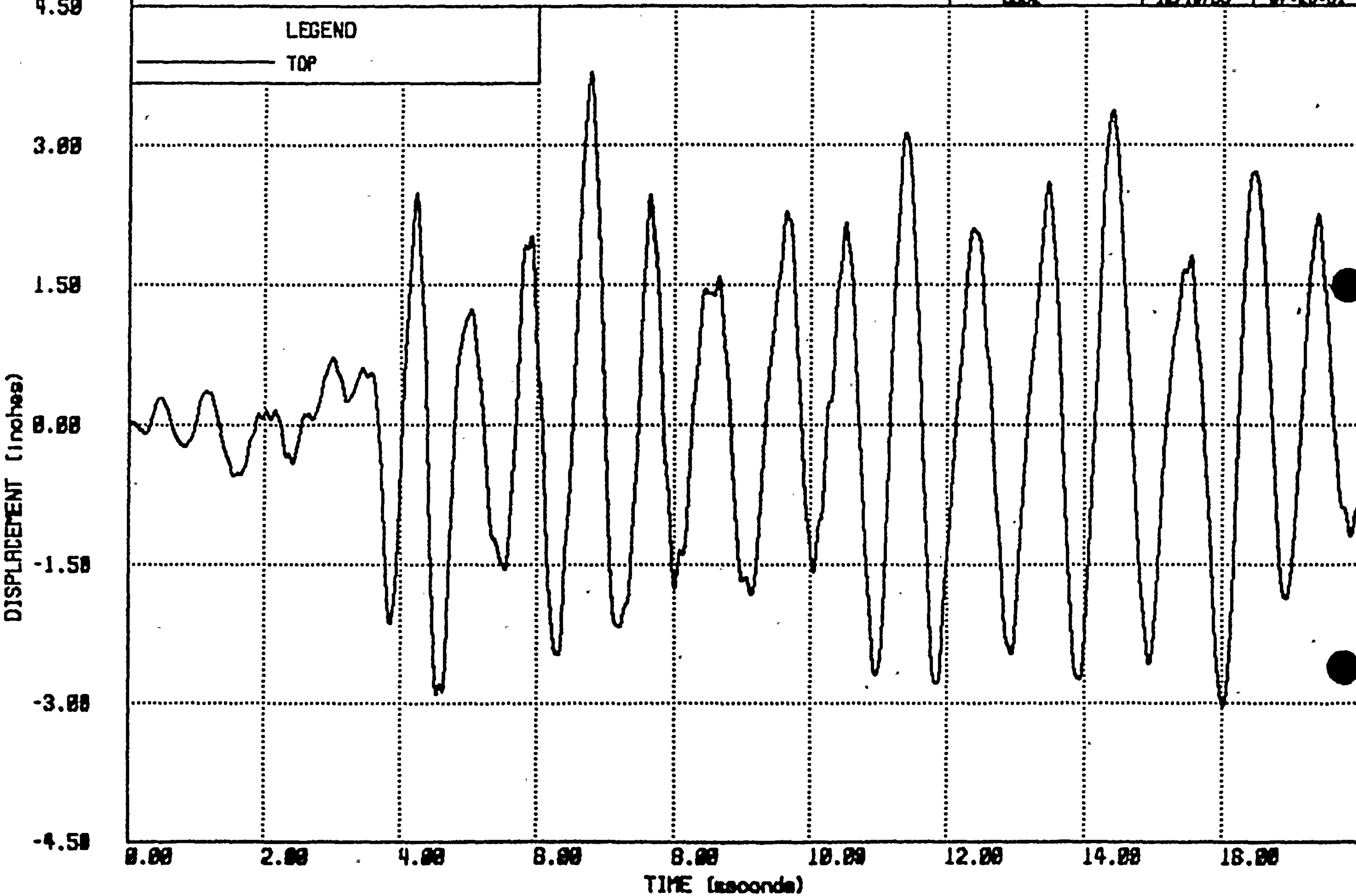

FIGURE 4.12 : WALL 973-4C : TAFT BASED RECORD. NO ECCENTRICITY

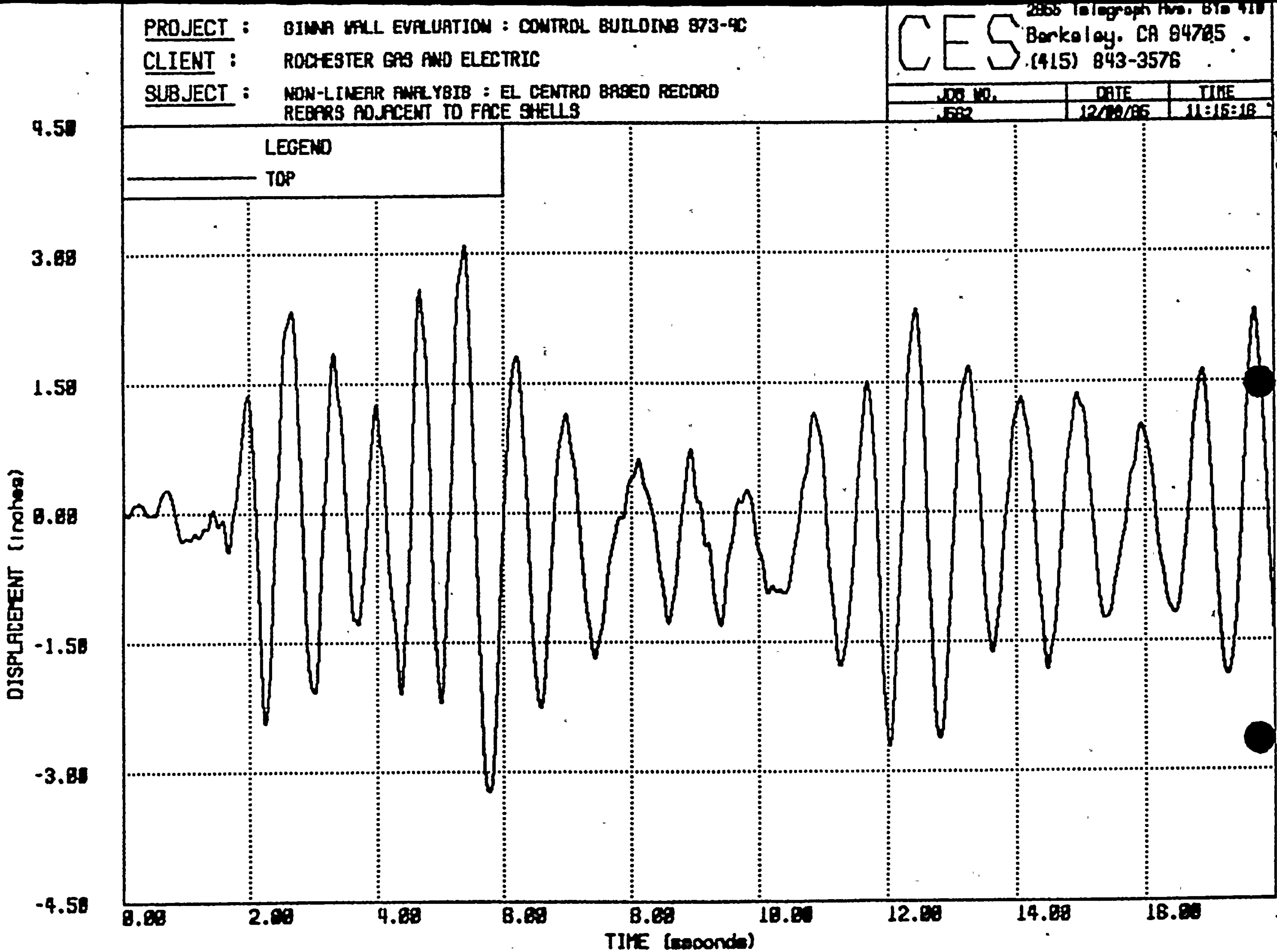

FIGURE 4.13 : WALL 973-4C : EL CENTRO BASED RECORD. 0.375" ECCENTRICITY

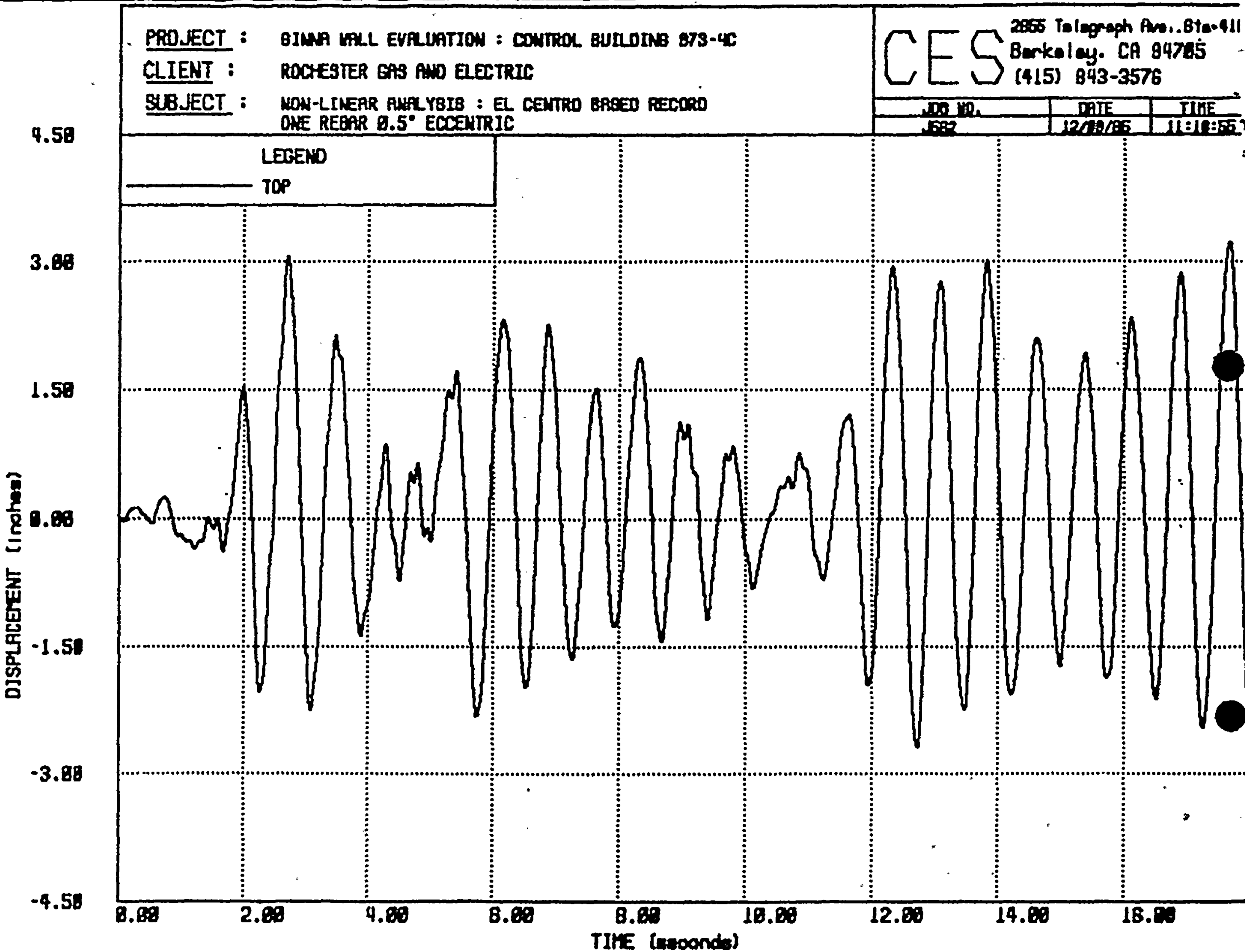

FIGURE 4.14 : WALL 973-4C : EL CENTRO BASED RECORD, 0.5" ECCENTRICITY

5 FINAL WALL EVALUATION

The preceding sections have presented the results from the elastic analyses of all walls and the inelastic analyses of the most severely stressed spanning and cantilever wall. In this section the results from these two series of analyses are used to evaluate the overall safety of the Ginna reinforced masonry walls under the seismic motions corresponding to those of the SEP criteria. The following sub-sections present the results of this evaluation for the spanning and cantilever walls.

5.1 Spanning Walls

The 7 spanning walls were analyzed elastically using the conservative assumption that the joint reinforcing would be ineffective in providing a load path to the edge supports. This required the walls to act as vertical plates and caused maximum stresses in the rebar of from 1.25 to 2.18 times the allowable value of 0.9 times the yield stress.

Of these 7 walls, two were analyzed non-linearly using time histories appropriate to the floor spectra. The two walls represented the highest and lowest levels of overstress thus enabling results for other walls to be obtained by interpolation between the results of the two analyses.

The interpolation of the results from the inelastic analyses of Walls 971-1C and 971-6C has been performed using the following steps, which are tabulated in Table 5.1:

1. For each of the 7 spanning walls, the ratio of maximum steel stress to yield stress as obtained from the elastic analysis is listed. These ratios range from 1.13 to 1.96.

2. The wall frequency at which these stresses is listed. The frequencies are similar for all walls, as would be expected because of the similar height and construction, with values ranging from 1.6 to 1.9 hz.

3. The spectral acceleration from the floor spectra used for each wall is obtained at the cracked frequency. These acceleration levels range from 0.35g to 0.47g.

4. The yielded frequency for the two walls analyzed inelastically is estimated to be 1.3 hz based on the time history of response. Based on these values, the yielded frequency in the remaining walls is estimated to be between 1.3 and 1.4 hz taking into account the similar cracked frequencies and the extent of inelasticity (as measured by the steel ratios). The interpolated values are shown in parentheses.

5. Based on the yielded frequency the spectral acceleration from the floor spectra are computed. These fall in a very narrow range of 0.30 to 0.34g.

6. For each wall the yield displacement is 1.0".

7. The inelastic displacement is then estimated based on the results of the two wall analyzed and the spectral accelerations and frequency of the remaining walls.

8. The inelastic displacement divided by the yield displacement provides an estimate of the overall displacement ductility for each wall.

The final estimated values of overall ductility range from 2.1 to 2.4 for the 7 walls. The ductility values for the two walls analyzed were 2.4 and 2.2. The results from the Wall 971-1C analysis listed in Table 4.1 show that for a ductility ratio of 2.4 the steel and masonry strains are well below the criteria limits. Therefore, it is apparent that the remaining walls with lower overall ductility demands will have similarly low material strain ratios.

Based on this it is considered that all spanning walls would perform satisfactorily under the SSE loading with degrees of non-linearity well within the capability of reinforced masonry.

5.2 Cantilever Walls

The five cantilever walls were analyzed elastically using the same conservative assumption that the joint reinforcing was ineffective in carrying loads to the

side supports. Under these conditions, one wall remained uncracked under maximum seismic loading and one had maximum stresses less than the criteria. Of the remaining three, one had a steel stress ratio of 1.02 which indicates that yielding would be unlikely. The other two had stress ratios of 1.54 and 1.68, indicating some inelastic response. The wall with the ratio of 1.54 was selected for a non-linear analysis as it was only 6" thick and therefore would be more critical for the nonlinear evaluation criteria (strains and displacements). This wall response was well within criteria limits on all parameters.

In comparing the parameters of the wall analyzed, 973-4C, with the yielding cantilever wall which was not analyzed inelastically, 972-2C, the same procedure as outlined above for the spanning walls was used. The overall ducility factor for the wall analyzed was only 1.1 as the yield displacement of the 6" wall is 3.2" (compared to a maximum inelastic displacement of 3.59"). For Wall 972-2C the cracked frequency is 1.8 hz compared to 1.1 hz for Wall 973-4C and the spectral accelerations in the yielded state are estimated to be 0.45g compared to 0.33g. However, Wall 972-2C is 8" thick compared to 6" for Wall 973-4C and therefore has a yield moment capacity approximately 4 times higher. Based on this additional strength it is considered that the wall would have a ductility lower than for the wall analyzed. Therefore, as the tabulated results for Wall 973-4C are much lower than the criteria limits (Table 4.3) the response for Wall 972-2C would also produce material strains well below the criteria limits.

	SPANNING WALLS						
	971-1C	971-2C	971-4C	971-5C	971-6C	972-4C/5C	972-6C
RATIO MAXIMUM STEEL STRESS TO YIELD	1.96	1.73	1.39	1.56	1.13	1.61	1.78
CRACKED FREQUENCY, hz	1.9	1.8	1.8	1.6	1.6	1.6	1.6
SPECTRAL ACCELERATION AT CRACKED FREQUENCY	0.47	0.45	0.38	0.35	0.35	0.40	0.42
YIELDED FREQUENCY, hz	1.3	(1.3)	(1.4)	(1.3)	1.3	(1.3)	(1.3)
SPECTRAL ACCELERATIONS AT YIELDED FREQUENCY	0.34	0.34	0.32	0.30	0.30	0.33	0.34
YIELD DISPLACEMENT	1.0	1.0	1.0	1.0	1.0	1.0	1.0
INELASTIC DISPLACEMENT	2.4	(2.4)	(2.2)	(2.1)	2.2	(2.3)	(2.4)
OVERALL DUCTILITY	2.4	(2.4)	(2.2)	(2.1)	2.2	(2.3)	(2.4)

TABLE 5.1 : EVALUATION OF SPANNING WALLS

6 CONCLUSIONS

The 12 reinforced masonry walls at the Ginna plant were analyzed elastically and based on the stresses in the vertical rebar the worst case spanning and cantilever walls were selected for a detailed non-linear analysis. Two of the wall qualified elastically using SGEB criteria and so were excluded from the inelastic evaluation.

The non-linear analyses considered variations in time history input and rebar placement and in all cases the maximum response parameters were well within the criteria limits specified in Section 4. The analytical methodology used was that based on the SONGS-1 test program and the results obtained from this series of analyses were in all cases much less than was observed in those tests and which was predicted using the same analysis methodology.

Based on these analyses it is concluded that the reinforced masonry walls have ample ductility to resist the design SSE input motions.