NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

State University of New York at Buffalo

EXPERIMENTAL STUDY OF 'ELEPHANT FOOT BULGE' INSTABILITY OF THIN-WALLED METAL TANKS

by

Z-H. Jia and R.L. Ketter

Department of Civil Engineering
State University of New York at Buffalo
Buffalo, NY 14260

Technical Report NCEER-89-0004

February 22, 1989

This research was conducted at the State University of New York at Buffalo and was partially supported by the National Science Foundation under Grant No. ECE 86-07591.

NOTICE

This report was prepared by the State University of New York at Buffalo as a result of research sponsored by the National Center for Earthquake Engineering Research (NCEER) and the National Science Foundation. Neither NCEER, associates of NCEER, its sponsors, State University of New York at Buffalo, nor any person acting on their behalf:

a. makes any warranty, express or implied, with respect to the use of any information, apparatus, method, or process disclosed in this report or that such use may not infringe upon privately owned rights; or

b. assumes any liabilities of whatsoever kind with respect to the use of, or the damage resulting from the use of, any information, apparatus, method or process disclosed in this report.

EXPERIMENTAL STUDY OF 'ELEPHANT FOOT BULGE' INSTABILITY OF THIN-WALLED METAL TANKS

by

Zi-He Jia[1] and Robert L. Ketter[2]

February 22, 1989

Technical Report NCEER-89-0004

NSF Master Contract Number ECE 86-07591

1 Post Doctoral Fellow, Department of Civil Engineering, State University of New York at Buffalo

2 Director, National Center for Earthquake Engineering Research and Leading Professor of Engineering and Applied Sciences, State University of New York at Buffalo

NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
State University of New York at Buffalo
Red Jacket Quadrangle, Buffalo, NY 14261

PREFACE

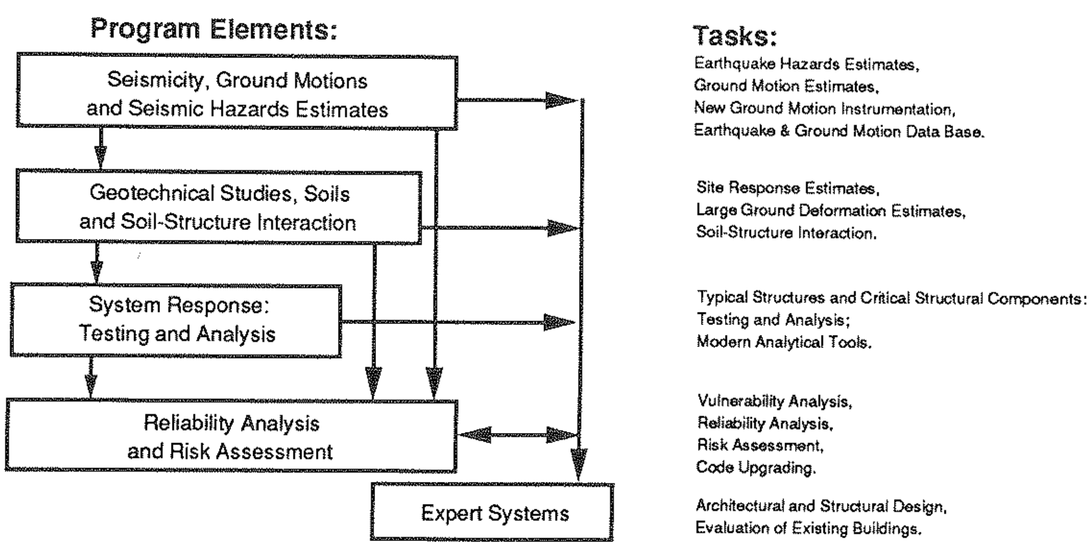
The National Center for Earthquake Engineering Research (NCEER) is devoted to the expansion and dissemination of knowledge about earthquakes, the improvement of earthquake-resistant design, and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures and lifelines that are found in zones of moderate to high seismicity throughout the United States.

NCEER's research is being carried out in an integrated and coordinated manner following a structured program. The current research program comprises four main areas:

- Existing and New Structures
- Secondary and Protective Systems
- Lifeline Systems
- Disaster Research and Planning

This technical report pertains to Program 1, Existing and New Structures, and more specifically to system response investigations.

The long term goal of research in Existing and New Structures is to develop seismic hazard mitigation procedures through rational probabilistic risk assessment for damage or collapse of structures, mainly existing buildings, in regions of moderate to high seismicity. The work relies on improved definitions of seismicity and site response, experimental and analytical evaluations of systems response, and more accurate assessment of risk factors. This technology will be incorporated in expert systems tools and improved code formats for existing and new structures. Methods of retrofit will also be developed. When this work is completed, it should be possible to characterize and quantify societal impact of seismic risk in various geographical regions and large municipalities. Toward this goal, the program has been divided into five components, as shown in the figure below:



System response investigations constitute one of the important areas of research in Existing and New Structures. Current research activities include the following:

1. Testing and analysis of lightly reinforced concrete structures, and other structural components common in the eastern United States such as semi-rigid connections and flexible diaphragms.
2. Development of modern, dynamic analysis tools.
3. Investigation of innovative computing techniques that include the use of interactive computer graphics, advanced engineering workstations and supercomputing.

The ultimate goal of projects in this area is to provide an estimate of the seismic hazard of existing buildings which were not designed for earthquakes and to provide information on typical weak structural systems, such as lightly reinforced concrete elements and steel frames with semi-rigid connections. An additional goal of these projects is the development of modern analytical tools for the nonlinear dynamic analysis of complex structures.

In this shaking table study of cylindrical liquid storage tanks, researchers successfully reproduced the buckling mechanism associated with the 'elephant foot bulge' phenomena in the laboratory. Two scaled aluminum tank models were subjected to both horizontal and vertical seismic excitations. The intensity of the shaking was incrementally increased until 'elephant foot bulge' occurred. All significant tank responses were monitored and studied, and the results showed clear agreement with previous 'elephant foot bulge' occurrences in the field.

ABSTRACT

In almost all strong earthquakes, ground supported liquid storage tanks experience considerable damage. The failure mechanism is one of instability of the tank shells, and the most significant type is known as 'elephant foot bulge'. The true cause of this type of failure is still not fully understood. There exists a question as to whether or not the current design criteria are conservative, reasonably accurate, or unsafe. One of the most important reasons for these uncertainties is the lack of experimental data associated with this type of failure mechanism.

This study was primarily intended to identify and quantify the 'elephant foot bulge' failure mechanism. Shaking table studies were carried out to investigate the buckling behavior of cylindrical liquid storage tanks under base excitations. Two specially designed aluminum tank models were fabricated, and they were subjected to various seismic loadings. The intensity of the shaking was gradually increased until the 'bulge' occurred. Other dynamic response behaviors of cylindrical tanks also were studied as part of the overall program.

During the tests, the phenomena of 'elephant foot bulge' was clearly observed for both model tanks. The post buckling failure patterns were almost identical to those observed in the field. Test results of this study indicate that current seismic design criteria do not necessarily safeguard against this type of failures. New design criteria and design alternatives are needed.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

SECTION 1
INTRODUCTION

Liquid storage tanks are vital parts of lifelines and industrial facilities. Their satisfactory performance is important both for regular and emergency services. Experience tells us, however, that during strong earthquake motion, these types of structures perform poorly. In recent decades, almost every strong earthquake has caused serious damage to ground supported cylindrical metal storage tanks [1, 2, 3, 4, 5]. Beyond their own value losses, failure of such tanks has resulted in other problems; e.g., fires, pollution of surrounding areas, and water shortage.

There are two types of tank failures which involve shell instability. The first, and most significant one is called an 'elephant foot bulge'. The second type has been termed 'diamond buckling'. The failure mode of 'elephant foot bulge' exhibits one or several permanent bulges near the base of the tank. These may extend around part or the entire shell circumference. One typical shape of 'elephant foot bulge' is illustrated in Figure 1-1, which was observed in the San Fernando, California Earthquake of 1971 [3]. Figure 1-2 is a sketch of another example encountered in the Tangshan, China earthquake of 1978 [22]. There, two bulges are uniformly distributed around the shell circumference.

Despite recognition of the importance of the 'elephant foot bulge' phenomena, there has been a great deal of uncertainty and misunderstanding concerning its causes and failure mechanisms. Research has been limited, and this has lead to significantly different explanations of why it occurs, and under what loadings. This is mainly due to the fact that controlled experimental data has been lacking.

In design, the differences between buckling of the 'diamond' type and the 'elephant foot bulge' type have not been recognized. The same criteria - allowable uniform compressive stress, are prescribed to safeguard against both of these possibilities. Recent research [32, 34], however, has indicated that the currently prescribed limiting values may be overly conservative for 'diamond' type of shell buckling. How adequate they are for the 'elephant foot bulge', on the other hand, remains a question. There is the possibility that they may not prevent instability.

Significant experimental studies of liquid storage tanks under dynamic excitations started shortly after the 1964 Alaska earthquake. Shaking table tests were conducted at the University of California at Berkeley to study tank behavior under seismic excitations [33]. The earlier tests of 'broad type' as well as 'tall type' tank models did not show any damage [18, 20]. Later tests of wine tanks exhibited buckling failure of the 'diamond' type. Other experiments on tank buckling behavior under both static and dynamic loadings have been performed at the California Institute of Technology and in Japan. A summary of the features associated with these tests is given in Table I-1.

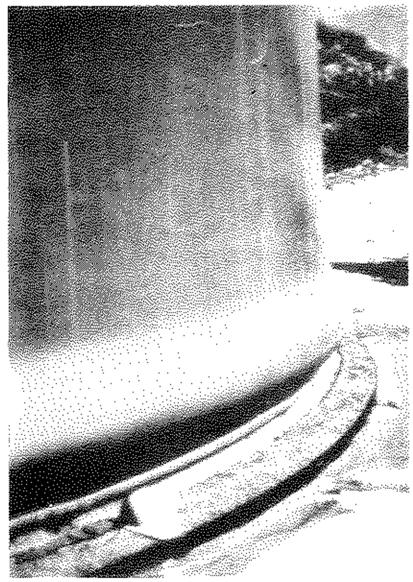


FIGURE 1-1 Failure Mode Due to 'Elephant Foot Bulge,' Observed in 1971 San Fernando Earthquake (Photo Courtesy of Canadian Journal of Civil Engineering 12, 12-23, 1985.)

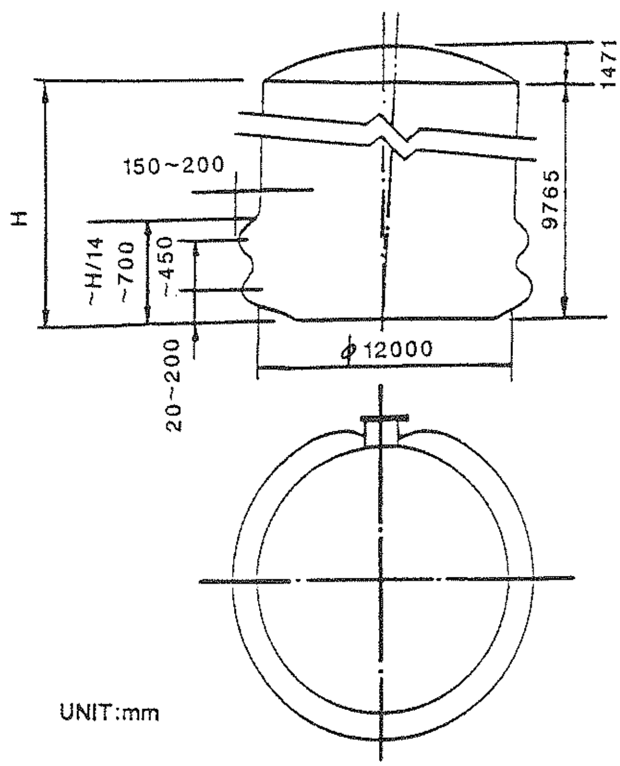


FIGURE 1-2 'Elephant Foot Bulge' Observed in 1976 Tangshan Earthquake, China

TABLE I-1 Buckling Tests of Cylindrical Tank Shells

Yr.	Author (Inst.)	Model Dimension R(mm)	H(mm)	t(mm)	R/t	H/R	Material	B.C.	Load Type	Results
1987	Nagashima (Japan)	80.0	320.0	0.05 0.098 0.241	1600 800 325	4.0 4.0	polyester sheet	fixed	top lateral sine base	shear and bending buckle $\sigma = 1.3\sigma_{cl}$
1985		500	2000	0.8	625	4.0	Alum.	fixed	hori. excit.	diam. buckle
1987	Reinhorn & Wang (UB)	1140 570	2865 1435	9.4 6.4	120 90	2.5 2.5	Plstic	unanc. & anc.	base excit.	no damage
1987	Zui (Japan)	800	1400 850	0.6 0.4	1333 2000	1.7 1.0	alum.	fixed free	base excit.	no damage
1986	Choi (Japan)	150	712 575 420 275	1.0	150	4.74 3.83 2.80 1.83	alum. & Steel	fixed	top lateral mono & cyclic	plastic shear and bending buckle
1982	Niwa & Clough (UCB)	1450	6100	2.0	725	4.20	stailess steel	unanc. & anc.	base excit.	diam. buckle
				4.0	363					no damage
1982	Shih (CalTech)	101.6	381 457 488	0.127	800	3.75 4.5 4.8	Mylar sheet	fixed	axl. + pres. base excit.	diam. buckle
1980	Yamaki (Japan)	100	22.8–227.4	0.247	405	0.23 2.28	polyester film	fixed	top lateral	platic shear buckle
1979	Kana (UTX)	314	762	0.51	620	2.43	alum	fixed	base excit.	No damage
1978	Niwa (UCB)	1180	4570	2.3	520	3.9	alum	anc. & unanc.	base excit.	no damage
1976	Clough (UCB)	1830	1830	2.0	900	1.0	alum	anc. & unanc.	base excit.	no damage

Compared to other structural forms, experimental data on the dynamic behavior of tanks, especially the behavior associated with failure, is extremely limited. Moreover, no literature has been found by the authors which indicates successful reproduction of the 'elephant foot bulge' under seismic laboratory test conditions. Understanding by the engineering profession of this phenomena, therefore, has been limited to what could be deduced from observations of actual failures caused by 'real' earthquakes.

To afford a better understanding, a test program was undertaken. The objectives were to reproduce the 'elephant foot bulge' phenomena under laboratory seismic simulation, and to study various response characteristics associated with such a failure mechanism. By mean of shaking table tests, two carefully planned, scaled aluminum tank models were subjected to known base excitations. Various wave forms were applied, and the intensity of the shaking was incrementally increased until 'elephant foot bulge' occurred. Major response parameters were monitored and studied.

SECTION 2
DYNAMIC BUCKLING TEST OF CYLINDRICAL TANKS

2.1 Tank Model

To simulate the stress state of tanks which were known to have failed in the 'elephant foot bulge' mode during past earthquakes, the material used in the models tested in this study should have elastic-plastic stress strain relationships similar to those of mild structural steel. The ideal material would be, of course, mild steel. Unfortunately, this requires either a very large model, or an extremely thin-walled shell. The former was prohibited by the size of the shaking table; the latter by the difficulties in fabrication. Considering both of these limiting conditions, the material for the model tank shell was finally chosen as aluminum alloy 1100-O, which, according to manufacturer's specification, has elastic strain-hardening properties with a minimum yielding stress and ultimate stress of 3.5 ksi. and 11 ksi, respectively.

Tension coupon specimens were prepared from the same sheet that was used to construct the tank models, according to ASTM standard B557. The tests were conducted in the material test laboratory of the Department of Civil Engineering of SUNY/Buffalo, using an Instron tension/compression machine. The loading rate was 0.05in/sec. A total of 15 specimens were tested. The specimens included some cut in the longitudinal direction, as well as in the transverse direction. No significant differences were observed between the two different directional cuts. The average yielding stress (at 0.2% strain offset) was 7.2 ksi, which was considerably higher than that specified in the manufacturer's catalog. The average Young's modulus was 9.0×10^6 psi (compared to the catalog specified value of 10×10^6 ksi). The obtained average stress-strain curve could be approximated quite closely by the Ramberg-Osgood material model. (Repeated loading and unloading tests of the tension coupons indicated that the material closely followed the isotropic hardening rule.)

Geometry and Size: The geometry of the test tank was selected so that the test could be used to describe the behavior of a reasonably large scale of tanks in the intermediate range of height/radius ratio. The size of the cylinder chosen was 36 inches in diameter, 42 inches in height, and 0.01 inches in shell thickness. The design liquid depth was 36 inches (full level), which gave ratios of $H/R = 2.33$, and $h/R = 2.0$. (To realize a reasonable shell hoop-tension stress under hydrostatic loading, a very thin shell wall was required.) For $t = 0.01$ inch, the R/t ratio equals 1800. The maximum hoop tension with the design water depth of 36 inches was about 65% of the nominal yielding stress (2.3 ksi).

It should be noted that by using such a thin, soft aluminum sheet, tremendous difficulty arose in the fabrication of the model. This sacrifice was required in order to increase the possibility of failure in the 'elephant foot bulge' mode. The stress state realized in the model tanks was comparable to that which existed in real tanks known to have failed by 'elephant foot bulge' instability.

Boundary Conditions: One unique feature of the model tanks used in this series was the design of the boundary conditions at the base. As shown in Figure 2-1, the model was designed to be fixed at the bottom with a cast-in-place concrete slab. Although most anchored tanks do not have such complete fixity, it was considered desirable for both the analysis and the experiment to test this extreme case. Such an approximation not only simplified the problem, but also eliminated many other unpredictable errors, such as modeling uncertainty of the bottom connections, etc. On the top of the tank, a 3/4 inch thick plywood stiffener ring was added to the shell, and a plastic glass lid was provided to approximate a flat-top roof.

Model Scaling: A detailed description of dynamic modeling considerations for ground supported, cylindrical liquid storage tanks can be found in reference [18]. With the exception of some minor scale factor differences, a similar approach was used in the present study.

Several different model scaling factors were selected to interpret the model tank response. First, the model tank was considered as the prototype. (Although such a prototype is seldom found in real engineering applications, model scaling and interpretation errors can be avoid if the model is the prototype.) Secondly, the model was interpreted as a 1/3 scale model of a steel wine tank. Thirdly, the model was interpreted as a 1/10 scale model of a tank that failed in the 1964 Alaska earthquake. The corresponding scale factors can be found in Table II-1. The resulting prediction relationships for prototype response are included in Table II-2.

Instrumentation: Accelerometers, displacement transducers, pressure meters, strain gauge rosettes and regular, one-directional electrical resistance strain gauges were used in the instrumentation of the tank model tests. The instrument arrangement on model tank No. 2 was slightly different from that on model tank No. 1. This was because more effective locations were realized only after testing model tank No. 1. Figure 2-2 shows the distribution and location of the various instruments on model No. 2. The total number of channels utilized for instrumentation was 45.

Recognizing the high frequency contents exhibited by the model's dynamic characteristics, the cut-off frequency of the filter was set to 100 hz for all of the instruments.

2.2 Excitation and Test Procedure

Ground waveforms selected to drive the shaking table include the following:

1. White noise (WN) - for structural identification purposes.

2. Sinusoid wave (SN) - for simplicity, to compare with analytical solutions. (Both low frequency (5 hz) and high frequency (20 hz) waves were used in order to study the dynamic behavior of the model tanks at different frequency ranges.)

TABLE II-1 Scale Factors of Buckling Test Models

COMPONENT	VARIABLE	SCALING FACTOR		
		1/1 model	1/3 model	1/10 model
TANK SHELL	mass density	1	1	1
	dimension	1	1/3	1/10
	elastic modulus	1	1/3	1/3
LIQUID	mass density	1	1	1
	depth	1	1/3	1/10
	viscosity	1	$1/\sqrt{27}$	$1/\sqrt{1000}$
EXCITATION	accel. mag.	1	1	10/3
	time duration	1	$1/\sqrt{3}$	1/5.77

TABLE II-2 Prediction Factors to Prototype Response

COMPONENT	RESPONSE QUANTITY	PREDICTION RELATIONSHIP		
		1/1 model	1/3 model	1/10 model
TANK SHELL	stress	1	3	3
	displacement	1	3	10
	period	1	$\sqrt{3}$	5.77
	acceleration	1	1	0.3
LIQUID	sloshing displacement	1	3	10
	period	1	$\sqrt{3}$	5.77
	pressure	1	3	3

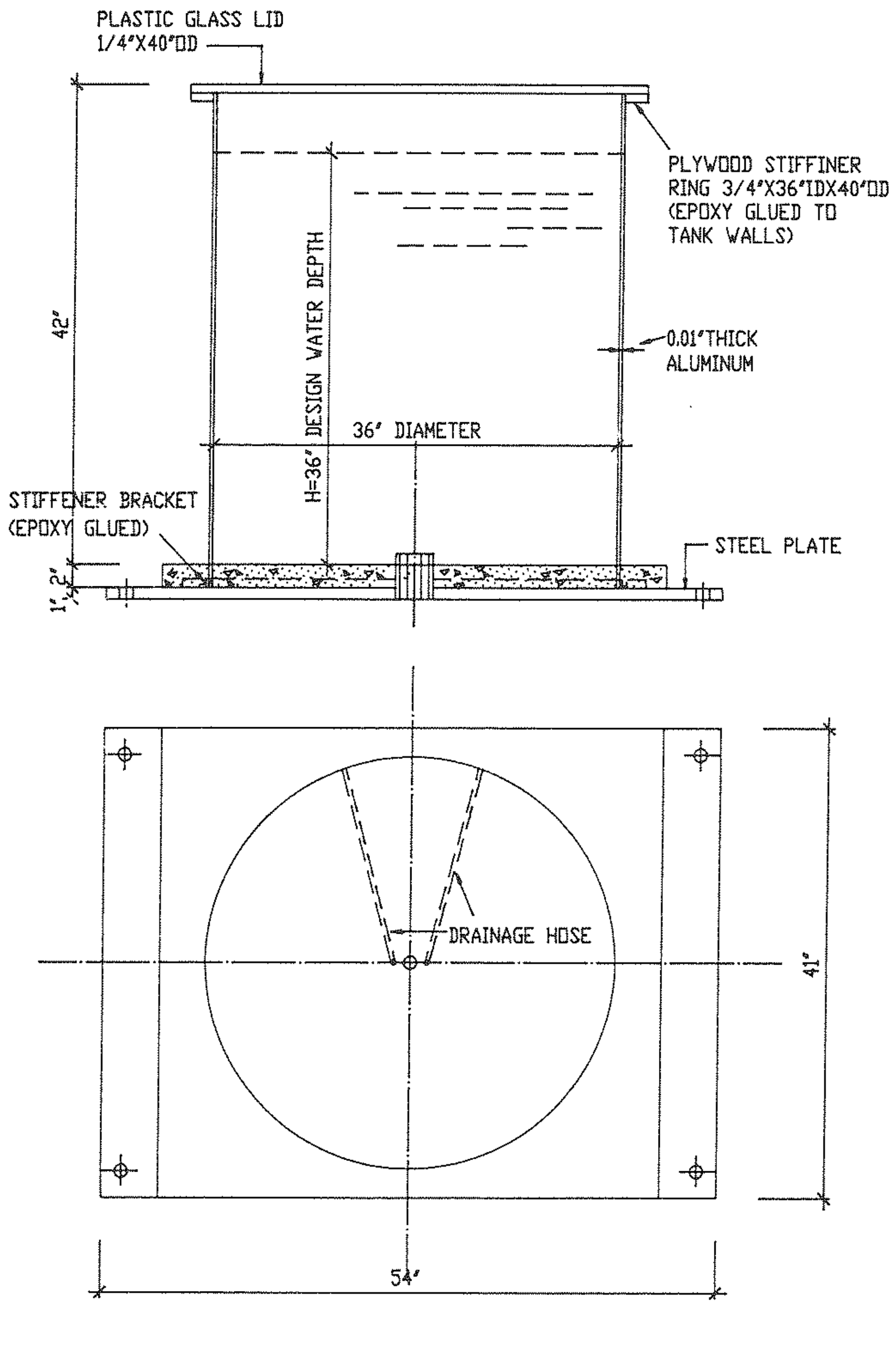


FIGURE 2-1 Design of the Tank Model and the Test Setup

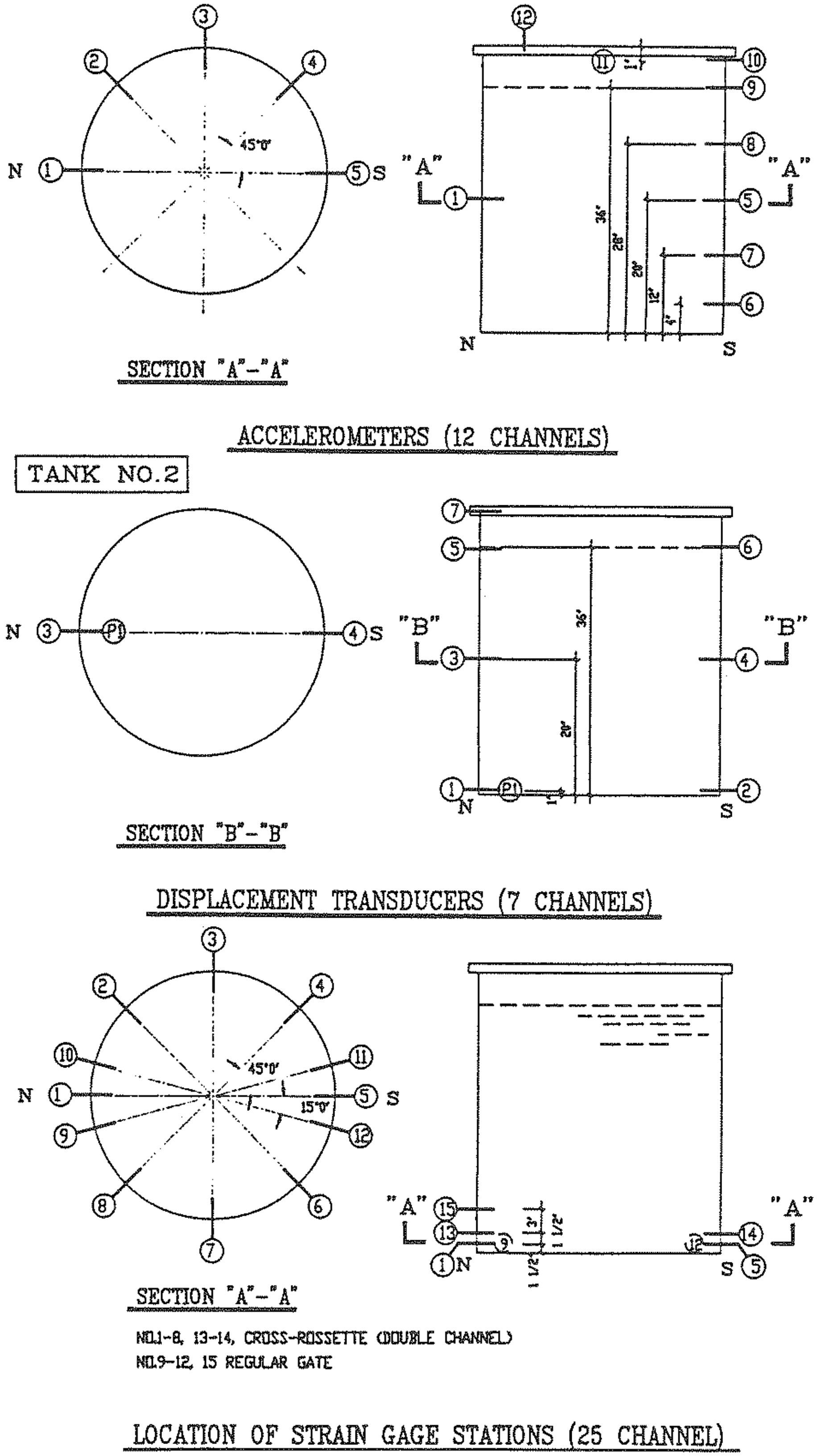


FIGURE 2-2 Instrumentation of Model Tank No. 2

3. El Centro record (EC) - for studying tank behavior under strong earthquake excitation. (In addition to the horizontal component, vertical records also were used to study vertical excitation effects.)

The control parameters originally planned were different water levels, and different intensities of shaking. However, from the preliminary runs of model No. 1, it was determined that all response parameters were considerably smaller when the tank was only partially filled. Therefore, all later tests were conducted with only one standard liquid level - 36 inch, fully filled. The intensities of the dynamic excitation were selected such that the response behavior of the tank could be monitored both in the linear elastic range and at the dynamic instability level.

The test procedure for both models was the following: First, a wide band white noise was applied to identify the dynamic properties of the model - for both 'empty' and 'full' tanks. Next, small amplitude sinusoidal excitations, containing either high or low frequency content, were used to study the general dynamic response of the tanks. Finally, El Centro earthquake records were applied to study the tank response to actual seismic excitations. Shakings using the El Centro records involved the horizontal component, the vertical component, and their combinations.

SECTION 3
TEST RESULTS OF TANK MODEL NO. 1

3.1 General

Tank model No. 1 was the first fabricated, and had larger imperfections than model No. 2. Moreover, the fixity at the base of tank model No. 1 was not complete. This was due to over-vibration during casting of the concrete, which caused moisture concentration at the boundary between the tank shell and concrete plate. The evaporation of this moisture left a small gap. Although there were a number of remedies that could have been used to correct this situation, it was decided to keep 'as it is' since it was considered that this might be closer to the boundary condition of real structures. Also, this would provide a comparison with the results of tank model No. 2, which was closer to the assumed fixed boundary.

The primary test concern of tank model No. 1 was whether or not the phenomena of 'elephant foot bulge' could be successfully reproduced in the laboratories.

3.2 High-Frequency Sine Wave Excitation

After preliminary tests had been carried out, horizontal sinusoidal waves SN20H containing high frequency contents at 20 hz and 0.2 g were applied to the full tank model. The duration of the excitation lasted 4.0 second, starting at 3.80 seconds. Selection of this particular magnitude of excitation was intended to produce significant response of the tank shell - but not buckling. This was based on analytical estimates and results of the preliminary tests. Unfortunately, buckling resistance of the tank shell was over-estimated, and the influence of imperfections at the boundary and in the general geometry of the shell were underestimated. Buckling occurred!

Strong shell membrane type of resonance, with audible noise, happened during the excitation. Violent vibrations were observed, which were concentrated primarily around the middle part of the tank shell. This indicated that the contribution of various circumferential waveforms of the cos $n\theta$ type was considerable. (The fixity at the bottom and the constraint of the top stiffener ring were sufficient to prevent circumferential vibration at the bottom and top, respectively.) After the test was stopped, it was discovered that slight permanent buckling of the 'elephant foot bulge' type had occurred near the bottom of the tank on both the north and south sides. Physical examination indicated a visible bulge extending about 60 degrees both clockwise and counter-clockwise from north and south, at a level of about 1.0 inch above the base. The observed buckling pattern is shown in Figure 3-1. The maximum radial outward bulge displacement was estimated to be about 3/16 to 1/4 inch, with the south side being slightly less than the north side. Except for some local small irregularities, the buckling pattern can be considered to be symmetric.

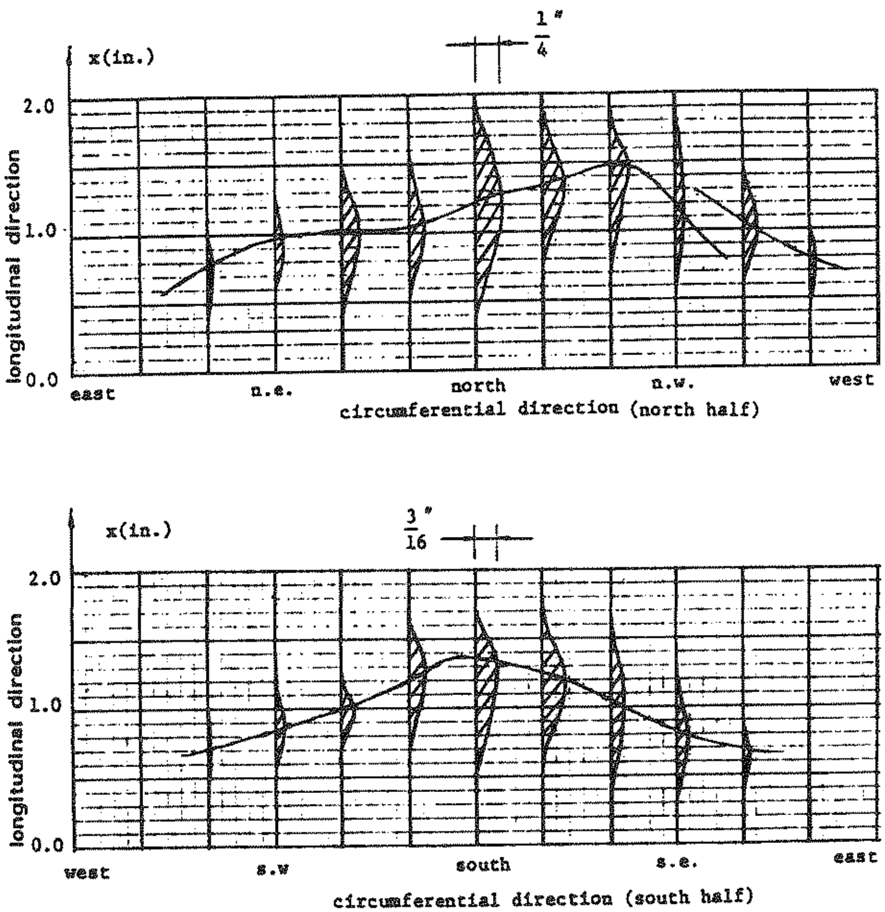


FIGURE 3-1 The Buckling Pattern of Tank No. 1 Due to SN20H Excitation

Strain gauges: Buckling also was observed from the instrumented strain gauge data. Figure 3-2 shows the axial and hoop strains at 3.0 inch above the base at the north and south sides. Although the major bulge occurred much lower than this level on the tank, the strain offset still indicates when the buckling happened. Strain gauges at other locations on this level behaved in quite similar fashion. (Surprisingly, strains on the east and west sides (see Figure 3-3), which were perpendicular to the excitation axis, demonstrated similar residual offsets. This suggests that slight buckling, although not visible, occurred at these locations as well.) By examining the dynamic strain histories, it can be recognized that buckling started in a gradual, resonance fashion. The maximum axial strain obtained along the excitation axis was about 0.012% (at the north side, around t = 5.6 sec.). This gave a critical buckling value of 0.216 Et/R, which is about 36% of the classical buckling stress under uniform axial compression. The maximum dynamic hoop tensile strain corresponding to this point was 0.018%. When combined with the existing stress induced by hydrostatic loadings, the maximum hoop tensile stress ratio σ_θ/σ_y was equal to 0.55, and the maximum compressive stress ratio σ_{axial}/σ_y was 0.15.

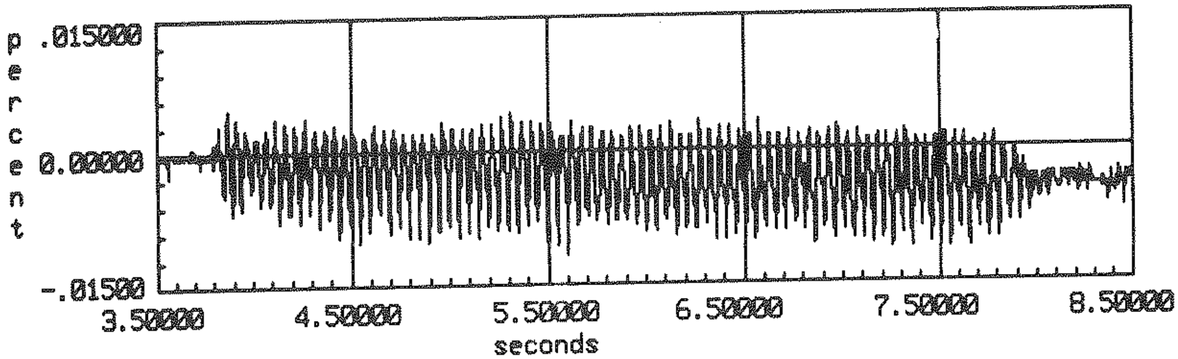
Displacement and acceleration: Displacements were monitored at three levels, - bottom, midheight and top, at both the north and south sides of the excitation axis. The maximum displacement at the top was observed to be about six times the thickness of the tank shell, indicating that the lateral stiffness of the tank was relatively strong. There was a small amount of residual displacement reflecting the rigid body movement of the tank shell associated with the buckling.

The acceleration responses were much stronger than expected due to the shell membrane resonance. The acceleration responses at the east and west sides should be especially noted, since these components were perpendicular to the excitation axis. Normally, if cantilever bending dominates the tank response, they should be quite small. However, during this test, their magnitudes were comparable with those of the north and south components. (Peak accelerations along and perpendicular to the excitation axis all exceeded the accelerometer's full scale range of 2.0 g's.) This indicated that the higher order waveforms of the cos nθ type contributed very much to the resonance buckling process.

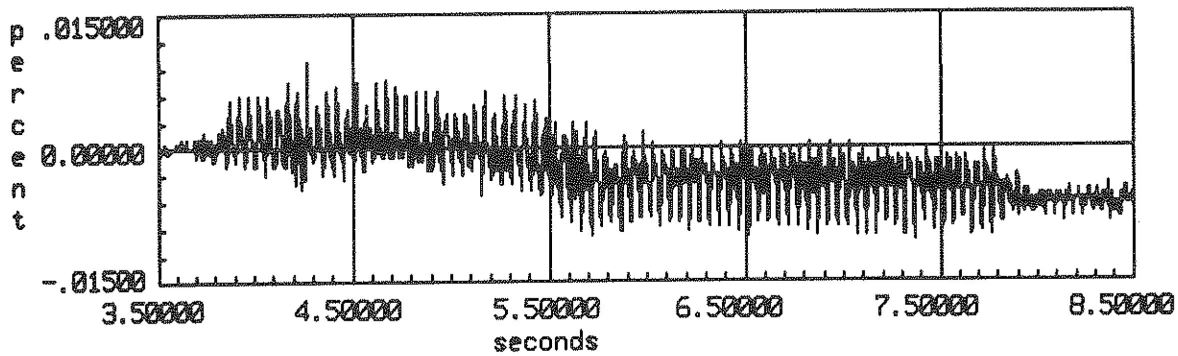
3.3 Post Buckling Excitations

Tests were continued to study the post-buckling behavior of the system after tank No. 1 suffered the slight buckling described above. Three additional test runs were performed. The excitations selected were: SN5H - sin5hz with 0.2 g magnitude; and full magnitude El Centro NS with time scale of 1.73 (EC1H1) and 5.77 (EC2H1) shorter in excitation duration, respectively.

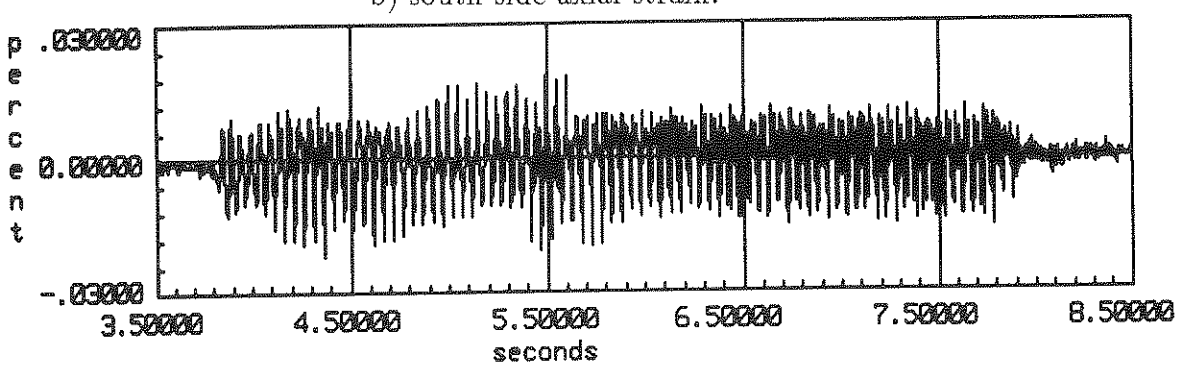
Response to SN5H excitation: Since the frequency content of this excitation was significantly removed from the natural frequencies of the tank system, the shell behaved as if it were a rigid body. During the excitation, no obvious shell vibrations were observed. The instrumented data also indicated a linear elastic response. (Comparison of this test to the same magnitude excitation



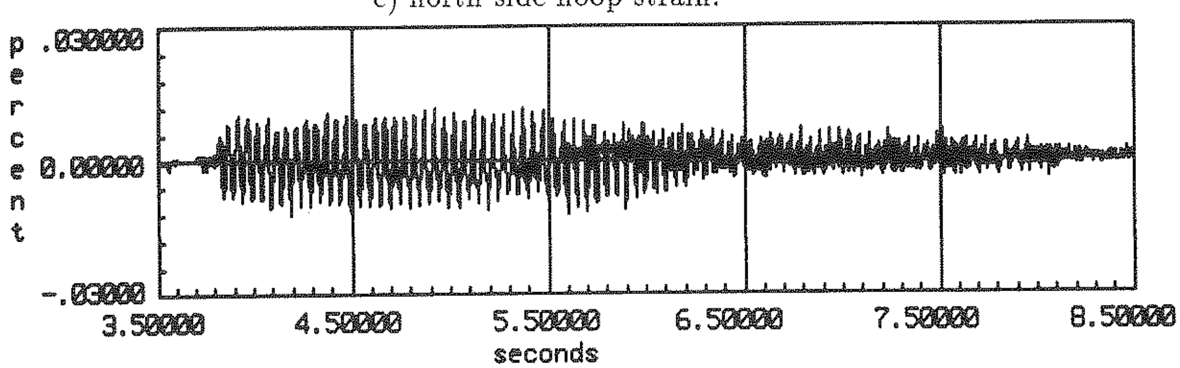
a) north side axial strain.



b) south side axial strain.

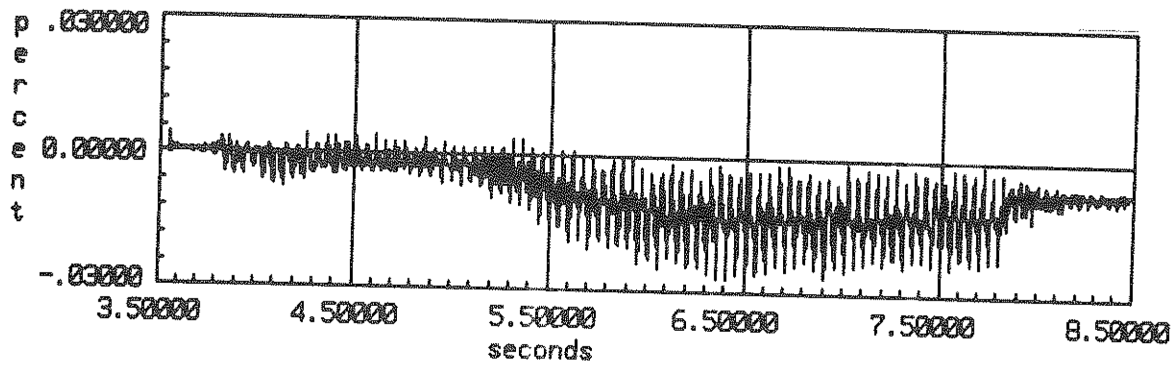


c) north side hoop strain.

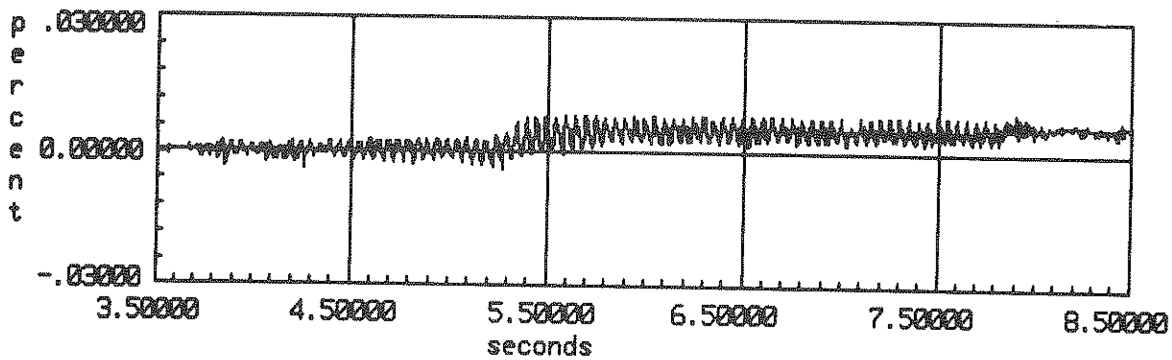


d) south side hoop strain.

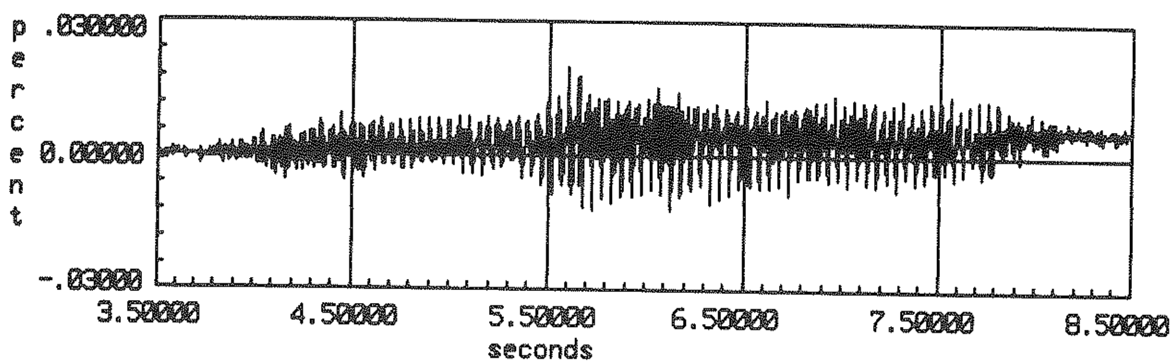
FIGURE 3-2 Strains Along the Excitation Axis (3.0) Inches Above the Base), SN20H Excitation, Tank No. 1



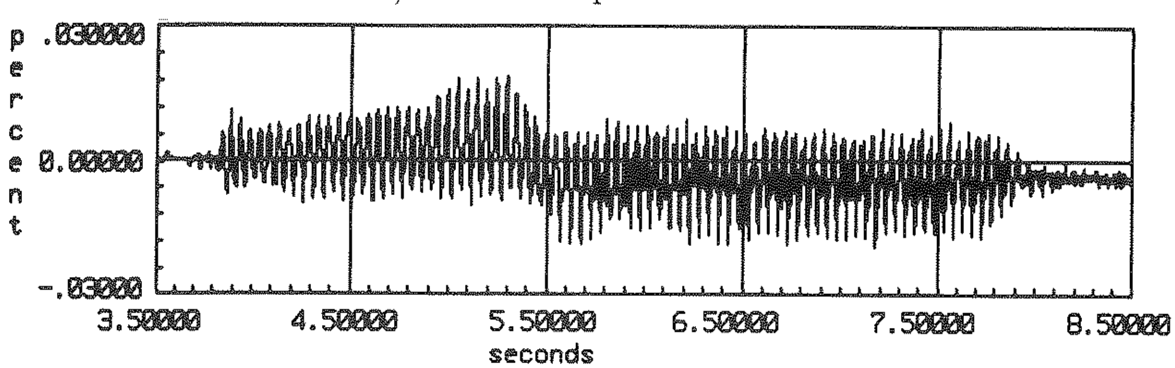
a) east side axial strain.



b) west side axial strain.



c) east side hoop strain.



d) west side hoop strain.

FIGURE 3-3 Strains Perpendicular to Excitation Axis (3.0 Inches Above the Base), SN20H Excitation, Tank No. 1

of undamaged tank No. 2 indicates that the post-buckling behavior of this shell is similar to that of an undamaged original tank subjected to the same excitation.)

Response to EC1H1 excitation: This test was for the purpose of observing the post-buckling behavior of the tank system subjected to real seismic excitation. Although the time scale was 1.73 shorter in duration, the energy input was still in the low frequency range. The tank shell behaved more-or-less like a rigid body with very slight membrane vibration. The instrumented data indicated that the tank shell response was larger than that of the previous SN5H excitation. No further deterioration was physically observed or instrumentally identified. This suggests that the tank, even after slight buckling, can withstand quite strong earthquakes.

Response to EC2H1 excitation: The time scale of this excitation was 5.77 shorter in shaking duration, which gave energy input peaks in the higher frequency domain. According to the model scaling laws of Table II-1, the response of the model tank due to such excitation was intended to simulate the response of a 10 times larger Alaska tank. During excitation, the response mechanism of the tank shell appeared to be a combination of rigid body bending and local membrane vibration. Moderate shell membrane vibration was observed. No indication of further buckling development was identified. Instrumented data showed stronger response than that of the earlier EC1H1 excitation, but within the linear elastic range.

In summary, the post-buckling behavior of the cylindrical tank was satisfactory. Slight 'elephant foot bulge' of the tank shell did not noticeably decrease its lateral resistance.

3.4 Apparent 'Bulge' Failure Excitation

Although a small amount of bulging of the 'elephant foot' type was obtained during the earlier SN20H test, the shape and the size of the buckle was not as apparent as those encountered in field observation. In order to study the complete failure mechanism associated with this type of instability, and to avoid the incremental deterioration of the shape due to gradual increase in the shaking, a very strong seismic excitation was selected to exaggerate the 'elephant foot bulge' process.

The strong excitation contained a 3.0 times larger amplitude in magnitude scale, and 5.77 times shorter duration in time scale of the El Centro NS record. The excitation was identified as EC2H3. The achieved maximum table acceleration was 0.89 g.

At the first moment the shaking reached its peak accelerations, all of a sudden the tank shell 'looked as if it had melted near the bottom' with significant inelastic shell deformation. The lateral resistance of the tank was exhausted at that moment, with very apparent rigid body movement of the upper part relative to the base. The high frequency contents of the shaking limited the excessive displacement. No major water leaks or eruptions were observed.

After the shaking, 'elephant foot bulge' was clearly observed in the direction of excitation, extending over a considerably wide range. The overall buckled shape of the model tank is shown in Figure 3-4. The more complete detailed views of the buckling is presented by pictures shown in Figure 3-5. The lower part b. of this figure shows the extended front view at the base of the tank around its periphery. The upper part a. shows typical sectional side views of the buckling at the southeast, east, northeast, north, northwest, west, southwest and west, respectively. By comparing these figures (side views) to Figure 1-1, the obtained post buckling shape of the model tank was in close agreement with those observed in field damage reports. A plastic shear type of buckling, instead of outward 'elephant foot bulge', was observed perpendicular to the excitation axis at the east and west sides.

Some of the instrumented data (mostly accelerations) exceeded their full scale range and malfunctioned due to the induced strong excitation. Most strain gauges worked well, and a few representative traces are presented in Figures 3-6 and 3-7. Looking at the residual strains of these figures, the axial component on the north and south excitation axis was 2-3 times greater than the hoop tension component, indicating a compression type of buckling; while the axial component on the axis perpendicular to the excitation axis was of the same order or smaller than the component in the circumferential direction, indicating a plastic shear type of buckling.

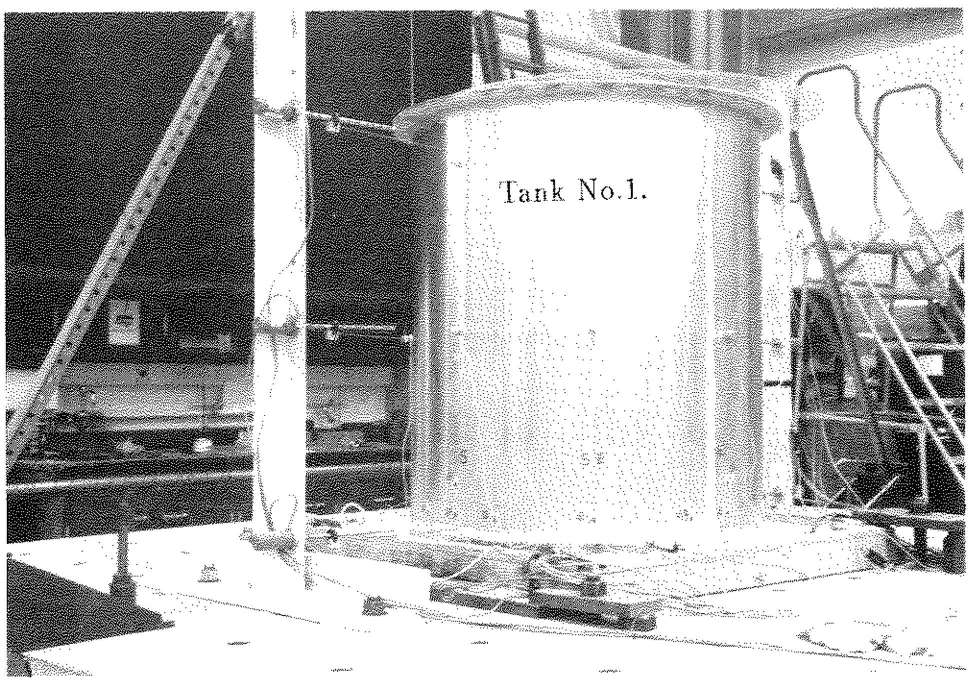


FIGURE 3-4 Overall Buckling Pattern of Tank No. 1 After EC2H3 Excitation

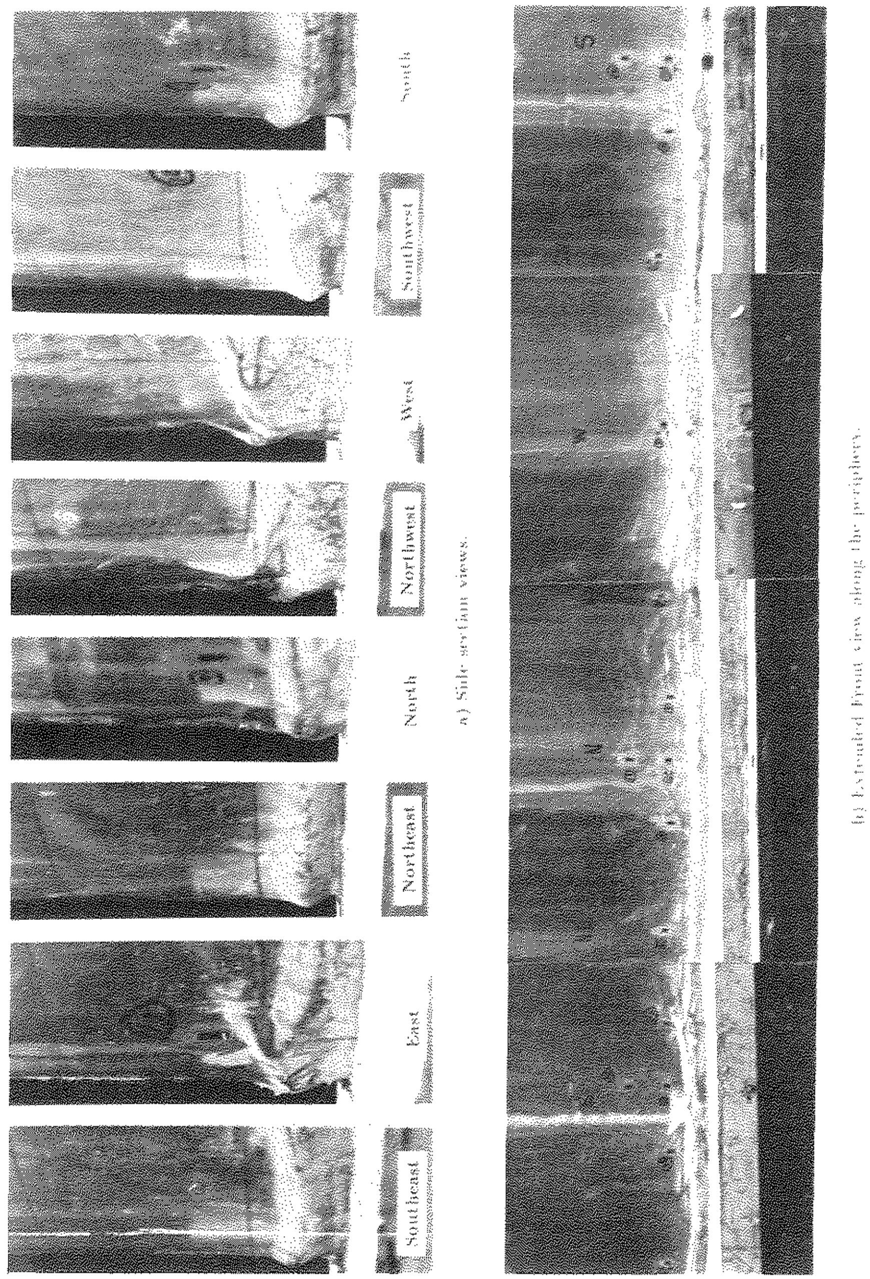
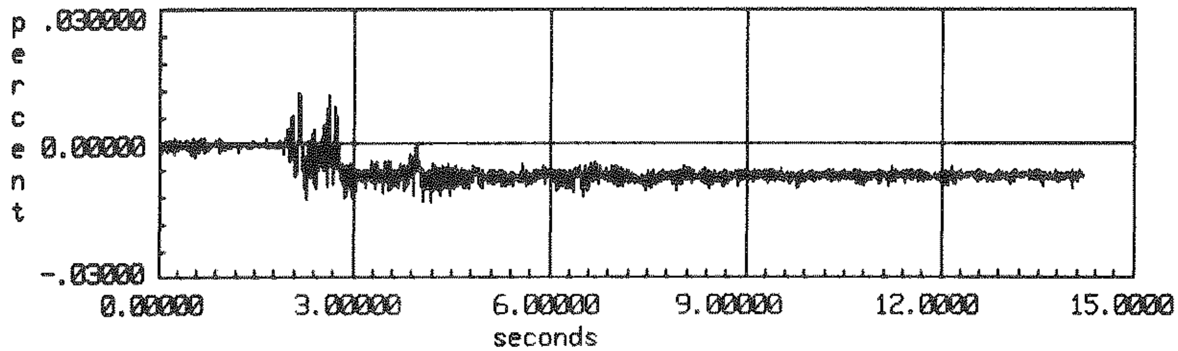
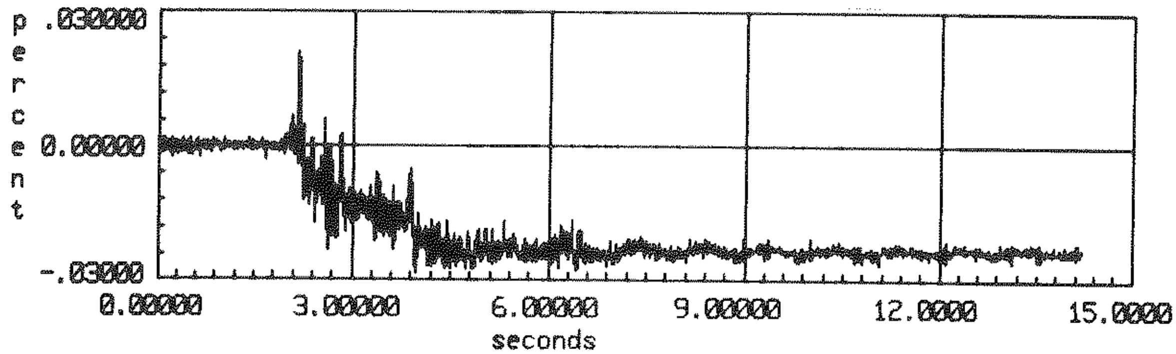


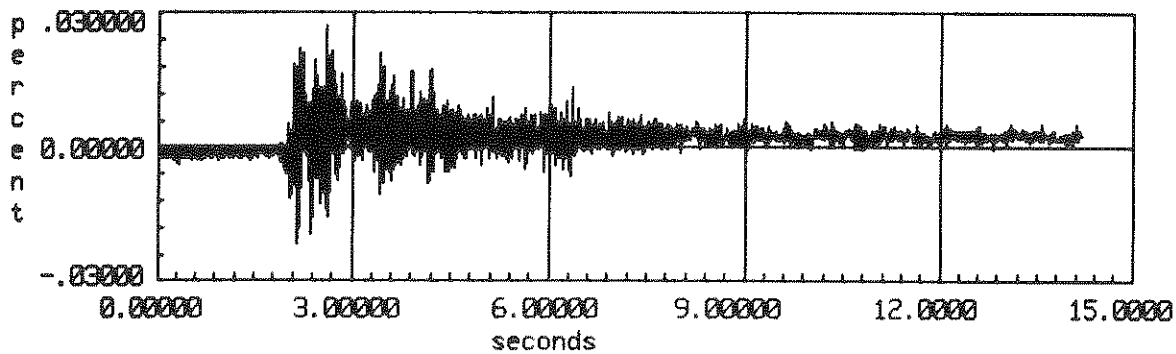
FIGURE 3-5 Buckling Shape of Tank No. 1



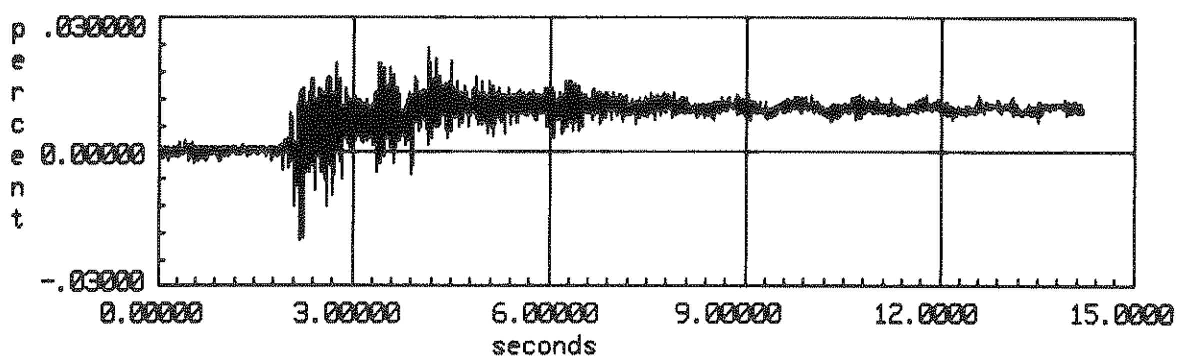
a) north side axial strain.



b) south side axial strain.

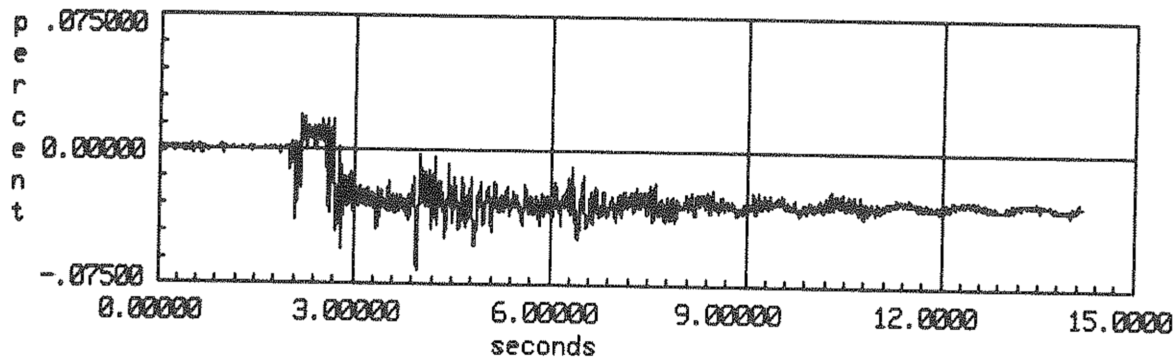


c) north side hoop strain.

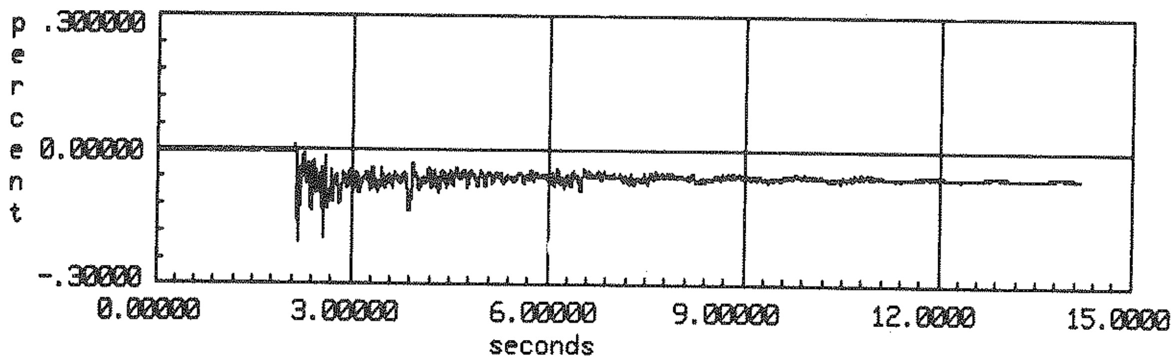


d) south side hoop strain.

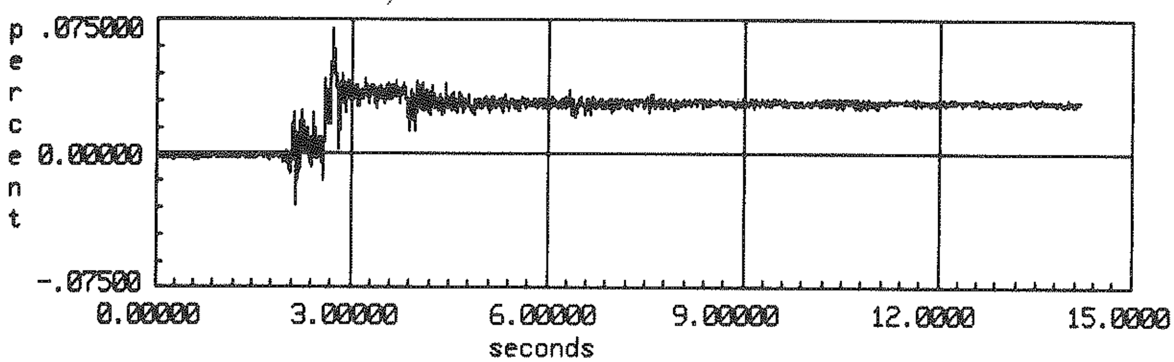
FIGURE 3-6 Strain Histories Along the Excitation Axis (3.0 Inches Above the Base), EC2H3 Excitation, Tank No. 1



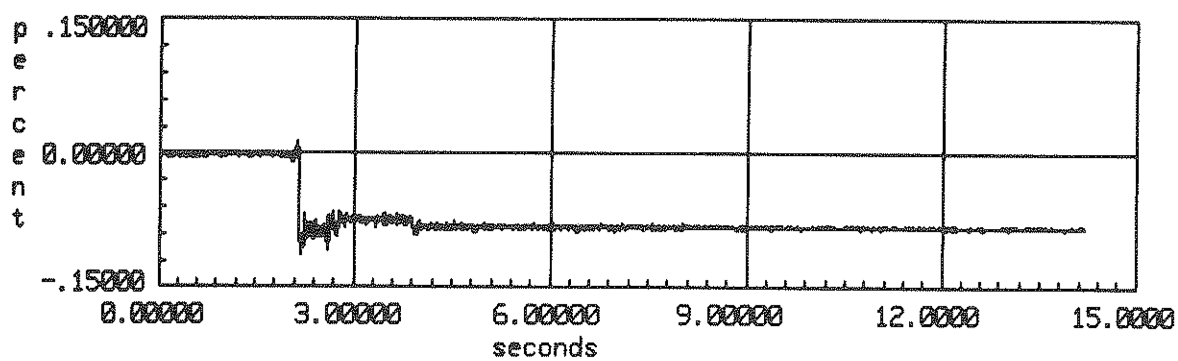
a) east side axial strain.



b) west side axial strain.



c) east side hoop strain.



d) west side hoop strain.

FIGURE 3-7 Strain Histories Perpendicular to the Excitation Axis (3.0 Inches Above the Base), EC2H3 Excitation, Tank No. 1

SECTION 4
TEST RESULTS OF TANK MODEL NO. 2

4.1 General

Tank model No. 2 was fabricated and tested after gaining experience from tank model No. 1. Better geometry and fixation at the base were achieved. Location of the instrumentation also was modified. The strain gauges were lowered to 1.5 inch above the base, and some of the accelerometers in symmetric circumferential positions were moved to south side vertical positions.

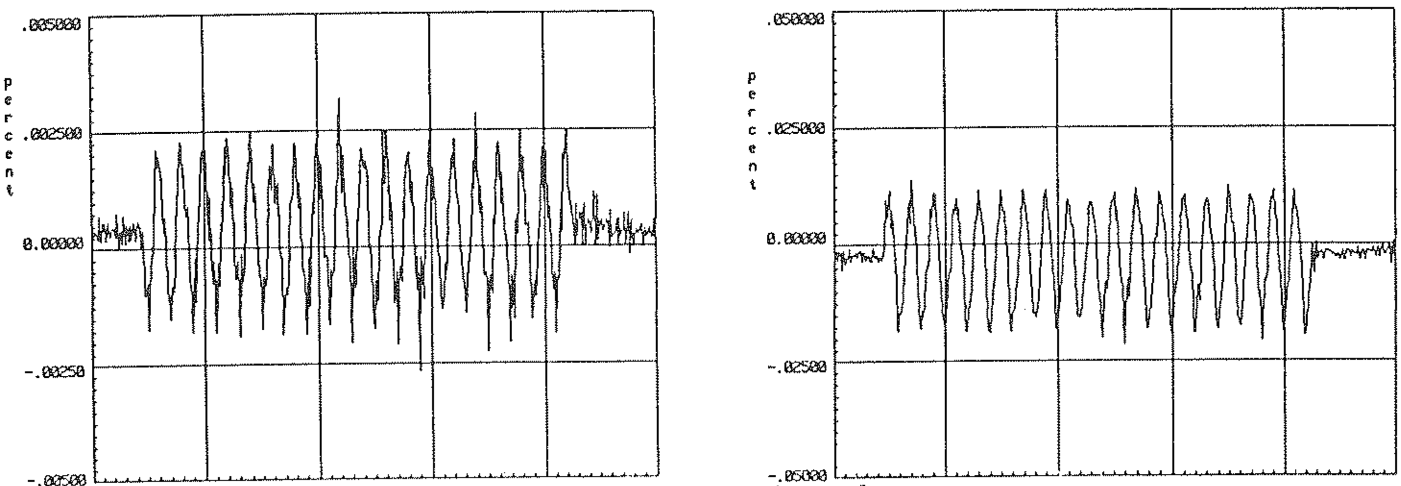
There were two major differences in the test program between tanks No. 1 and No. 2. The first was that in tank No. 2, the frequency content of the excitations contained more low, rather than high, frequencies. (The shell membrane resonance was thus avoided in this model, to facilitate study of the tank response characterized with relatively stiff shell walls.) The second difference in test series 2 was that both horizontal and vertical components of excitations were applied; first separately, and then in combination.

4.2 Low Frequency Sine Wave Excitations

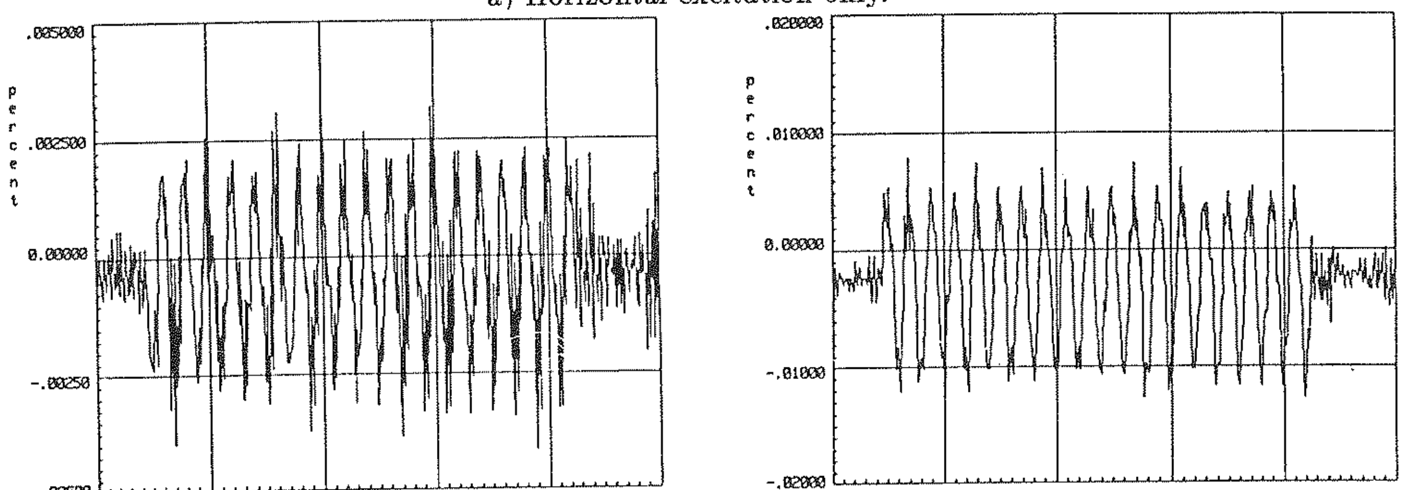
Linear elastic response of tanks subjected to non-resonance types of sine wave excitation was the major interest of this part of the program. The primary concerns included general response, vertical and combined excitation effects. A sin5hz wave form was first applied to drive the table in the horizontal NS direction (SN5H). Then that same pattern was applied in the vertical direction (SN5V). Finally, combined horizontal and vertical motions of the same sine wave (SN5HV) were applied.

Response to SN5H excitation: No obvious membrane resonance was observed during the shaking. Both observation and instrumentation indicated that cantilever bending was the major response mechanism. Axial strain distribution at the bottom ring followed normal, first order bending theory, with maximum values at the north and south sides, and almost zero at the east and west sides. Strains in the circumferential direction demonstrated a similar distribution, with the same order of magnitude, but of opposite sign. (This clearly demonstrated that the stress state of the tank shell element due to horizontal excitation is bi-axial rather than uni-axial. The signs of the two directional stresses are always opposite. When the axial stress reaches its maximum value of compression, the hoop tension reaches its maximum value as well, giving a maximum shear stress combination. Figure 4-1a shows typical strain responses at the bottom of the south side of the tank to this excitation.)

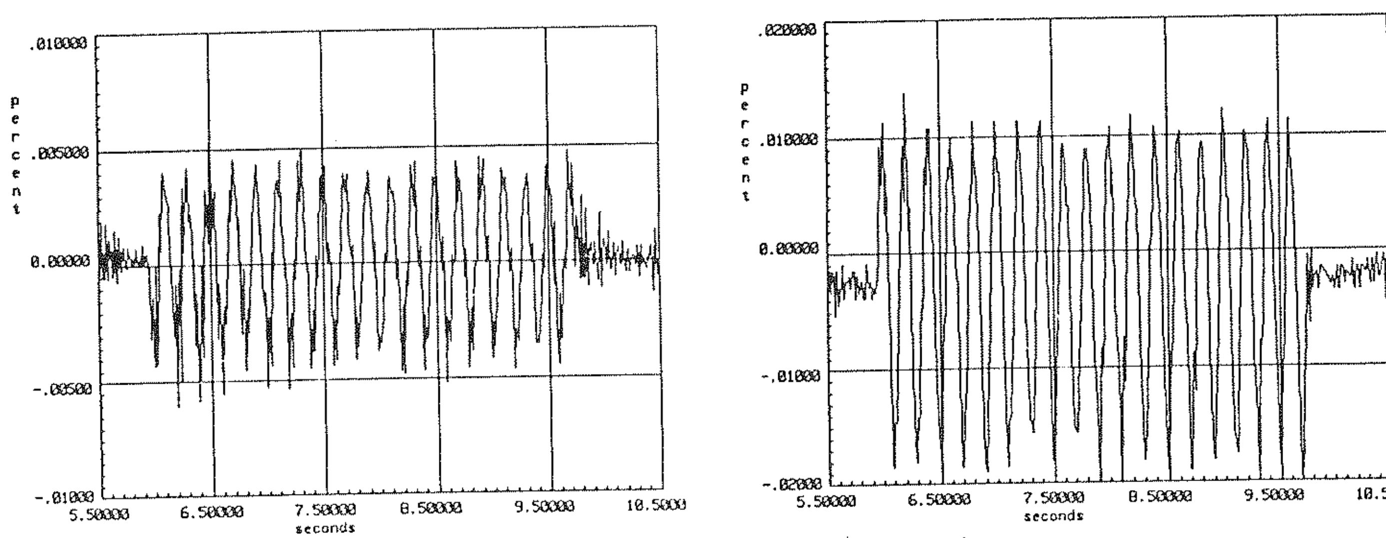
Response to SN5V excitation: Rigid body-like movements were observed for the entire tank during the vertical shaking. Hydrodynamic pressure was close to the calculated value, assuming a rigid tank wall. (The measured maximum value was 0.31 psi compared to a calculated value of



a) Horizontal excitation only.



b) Vertical excitation only.



c) Combined (horizontal & vertical) excitation.

South side axial strain. South side hoop strain.

FIGURE 4-1 Strain Histories at the South Side of Tank No. 2, Sin5hz Excitations

0.26 psi.) The circumferential hoop strain distribution was as expected, with a maximum value 30% higher than if it were assumed that the tank wall was rigid. Axial strain components in the order of 2/5 of those in the circumferential direction were measured, which were primarily due to Poisson effects plus local shell bending. Strains at the bottom of the south side of the tank are presented in Figure 4-1b.

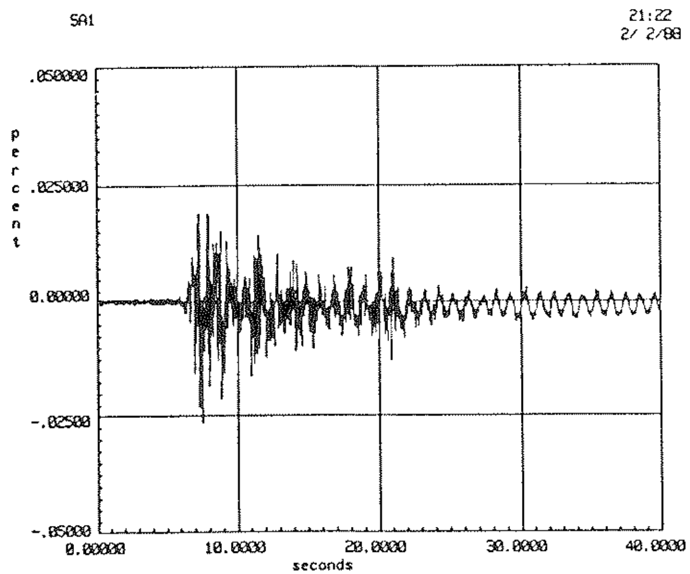
Response to SN5HV excitation: Theoretically, if the material remains elastic and no second-order geometric effects come into play, the response of the tank to the combined horizontal and vertical excitation should be a linear combination of the responses due to the separate excitations. The experimental data demonstrate that such was the case. Strain responses at the bottom of the north side, due to this combined excitation, are shown in Figure 4-1c). By comparing these to the corresponding strains shown in Figures 4-1a, horizontal excitation only, and 4-1b, vertical excitation only, it is clear that linear superposition is valid.

4.3 Scaled El Centro Horizontal and Vertical Excitations

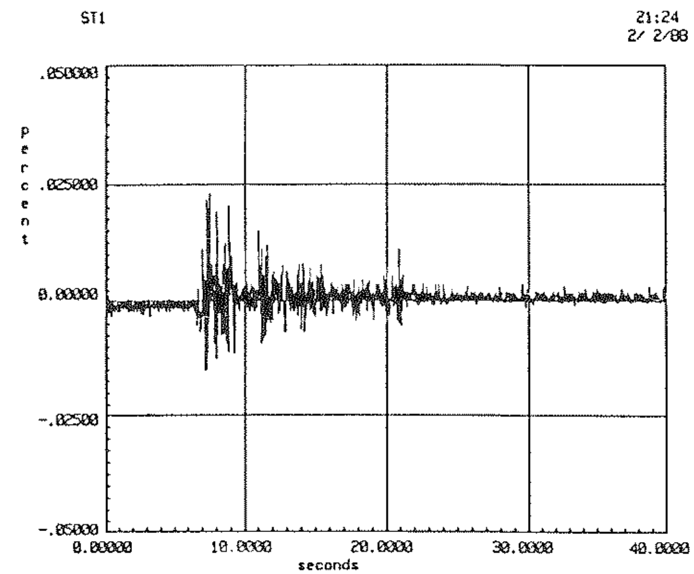
Here, the primary objective was to study the tank response to 'real' seismic excitation which has major energy input in the low frequency range. Buckling due to a resonance type of behavior was to be avoided. El Centro NS and vertical components with a time scale of 1.73 shorter in duration were used to shake the tank model. The intensity was incrementally increased by 50% at each step, starting from 100%. The shaking sequence at a specific intensity was defined as follows: horizontal only, vertical only, and then combined vertical and horizontal. (Interpretation of modal responses to these excitations correspond to three times larger prototype structure steel tanks, as indicated in Table II-2.)

During the 100% excitations, the tank responses were slightly stronger than those exhibited during the sin5hz excitations, with very slight membrane vibration. Physical observation indicated that the response mechanism was similar to the previous ones. All strain gauges indicated linear elastic behavior. For combined vertical and horizontal excitation, a linear combination of the individual responses was observed. Responses to excitations of 150% and 200% of El Centro were more or less the same, but with an increase in scale factor of 1.5 or 2.0. No buckling, or any other type of failures was encountered. Several drops of water spilled from the top, due to an unsatisfactory seal.

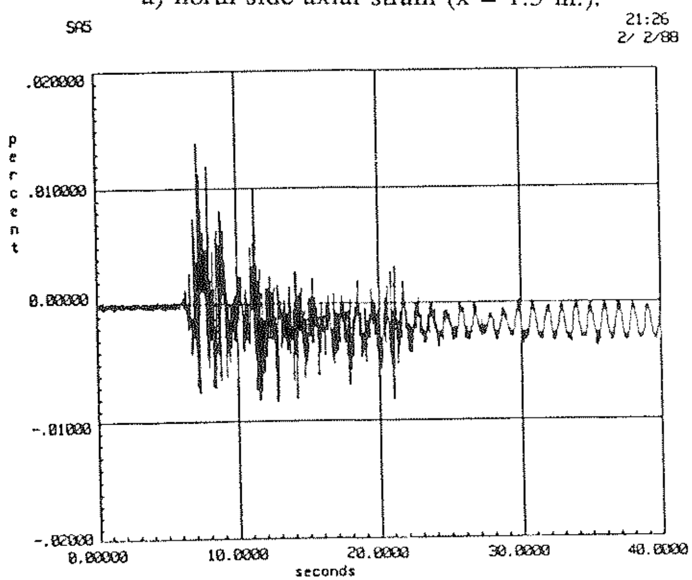
Response to EC1H2.5 (250% horizontal) excitation: Moderate tank response was observed due to this magnitude of excitation. No damage was visible to the tank shell. However, upon checking the strain gauge records, it was found that small amounts of residual strains were accumulated at certain locations at the base of the tank, mainly near the excitation axis. (Representative strain histories due to this excitation are shown in Figure 4-2.) The maximum compressive strain at the north side was 0.026%, while the associated maximum hoop tensile strain was 0.023%. When combining equivalent dynamic stresses with the existing stresses due



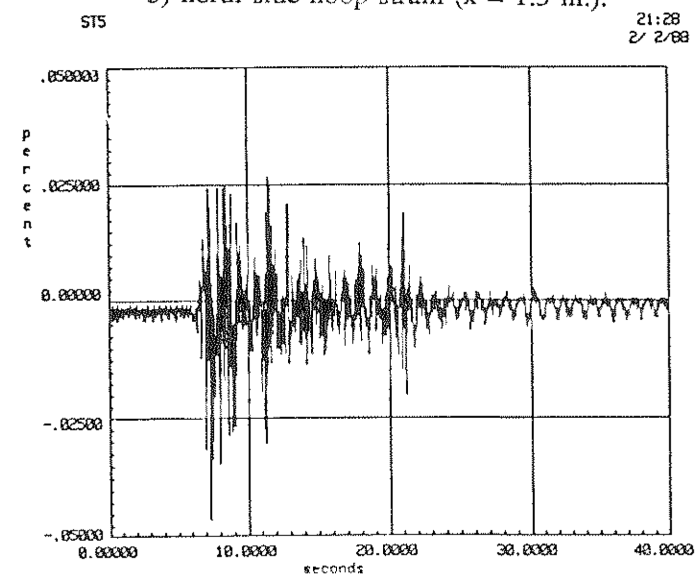
a) north side axial strain (x = 1.5 in.).



b) north side hoop strain (x = 1.5 in.).



c) south side axial strain (x = 1.5 in.).



d) south side hoop strain (x = 1.5 in.).

FIGURE 4-2 Selected Strain Histories to EC1H2.5 Excitation, Tank No. 2

to hydrostatic pressure, the resultant hoop tensile stress ratio σ_θ/σ_y was 0.60. The compressive stress ratio σ_{axial}/σ_y was 0.32. The combined maximum shear stress ratio τ_{max}/σ_y was 0.92, which was very close to the Tresca yield limit. Since the proportional limit of the material was considerably lower than that of the yielding limit (0.2% offset), this stress combination caused a nonlinear, inelastic type of response.

Response to EC1V2.5 (250% vertical) excitation: The maximum vertical acceleration achieved was 0.52 g. A linear elastic type of response was observed from both physical examination and instrumented data reduction. Dynamic strains developed in the tank shell were of a similar magnitude order to those observed for EC1H1 (100% horizontal) excitation.

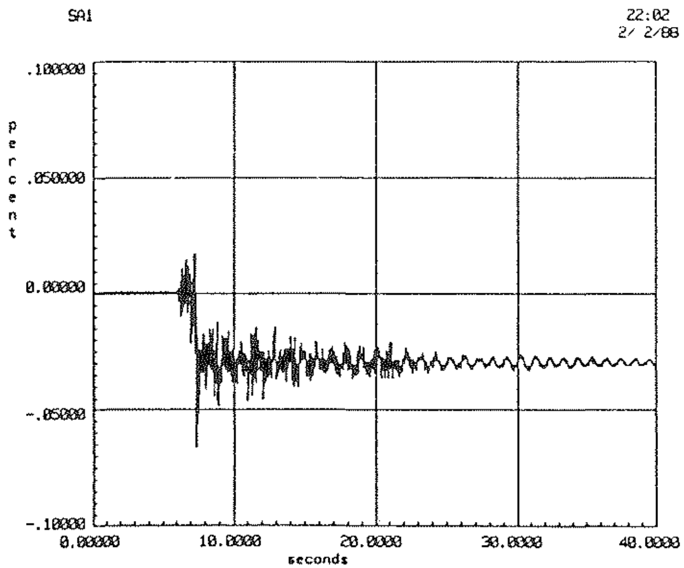
Response to EC1HV2.5 excitation: Here, the achieved table excitation was a combination of the horizontal EC1H2.5 and vertical EC1V2.5. A mode of cantilever bending plus moderate-strong membrane vibration was observed. Physical examination, after the shaking, indicated a slight buckling around a large portion of the north and south sides at the bottom ring. The shape of the buckling was rather flat in the vertical direction, and barely visible, but capable of being felt with the hand. Most of the recorded strain histories at 1.5 inches above the base indicated inelastic deformation, with the north-half larger than the south-half. At the north side , as shown in Figure 4-3, the inelastic deformation extended to at least 6.0 inch above the base, indicating that a relatively large portion of the tank shell had experienced strains into the inelastic range.

The displacement history at selected locations along the north and south sides are shown in Figure 4-4. The sudden jump of the north side bottom displacement indicated very clearly when the buckling occurred. The corresponding displacements at the opposite south side locations show nothing but 'noise'. The relationship between the mid-height and top displacements at the south side suggest that the cantilever bending was the dominant response mode.

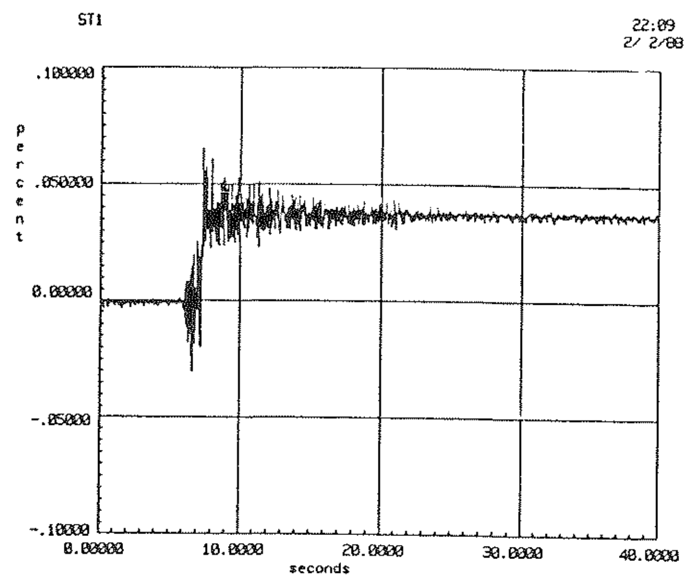
4.4 Apparent 'Bulge' Failure Excitation

El Centro NS excitation at 300% magnitude scale, 1.73 time scale (EC1H3) was used to bring tank model No. 2 to total failure. The obtained peak acceleration was 1.46 g. Due to nonlinearity of the control system, this was larger than the initially desired 1.02 g.

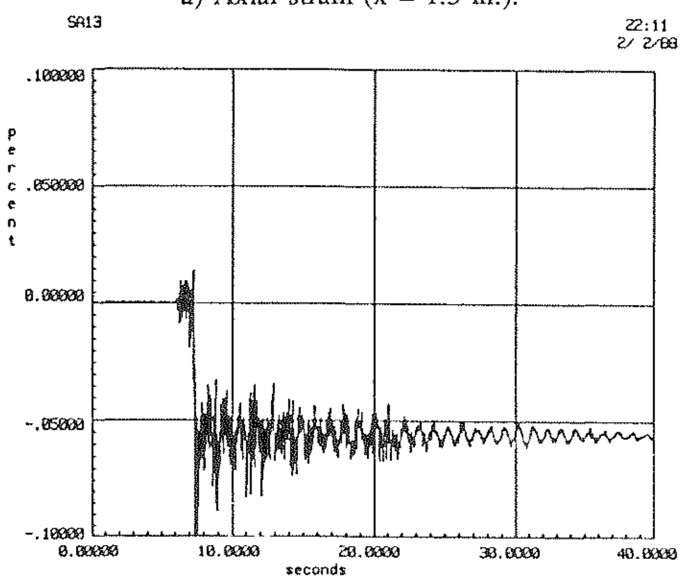
Violent tank responses, involving complete shell 'elephant foot bulge' failure, were observed for this strong shaking. The buckling initiated at the north side of the tank almost immediately after commencement of shaking. This was followed by an instantaneous spreading of the inelastic zones all around the tank shell near the base. Lateral resistance was completely lost, and large rigid body translation and rocking movements were observed. Vertical settlement around the base of the tank shell was observed. Some of the displacement transducer arms were unable to adjust to this large vertical settlement of the tank shell, and ended up malfunctioning. Many strain gauges 'popped off' from the tank shell due to excessive inelastic deformation. Upon



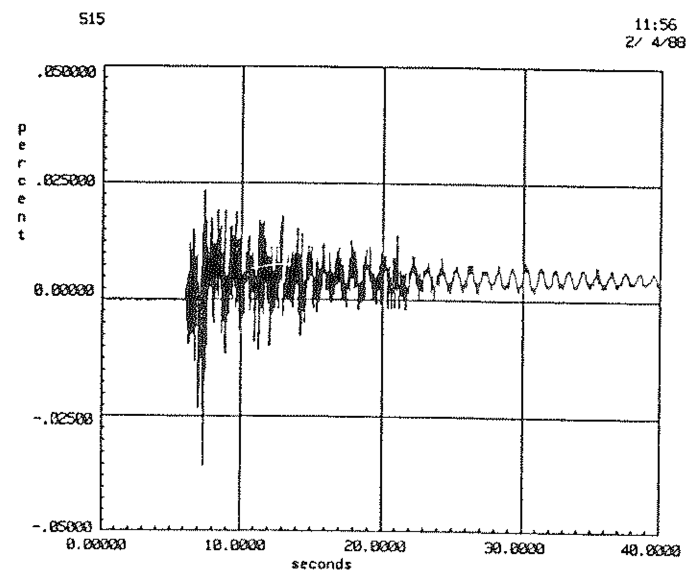
a) Axial strain (x = 1.5 in.).



b) Hoop strain (x = 1.5 in.).

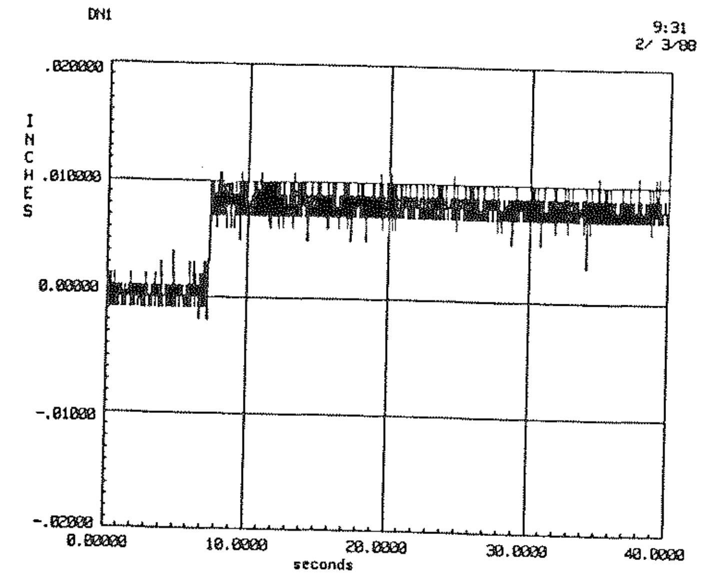


c) Axial strain (x = 3.0 in.).

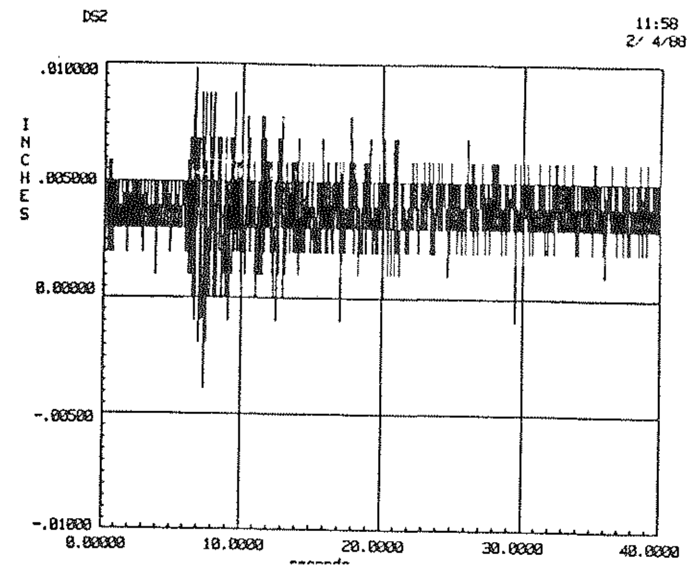


d) Axial strain (x = 6.0 in.).

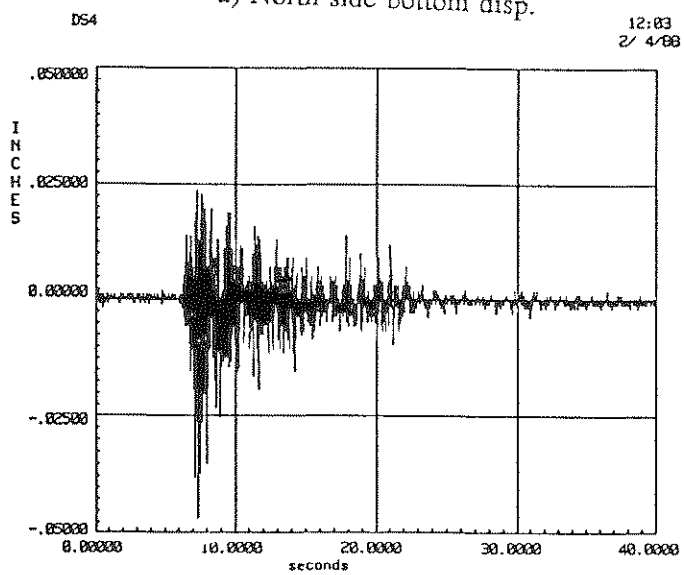
FIGURE 4-3 Strain Histories at the North Side of Tank No. 1, EC1HV2.5 Excitation



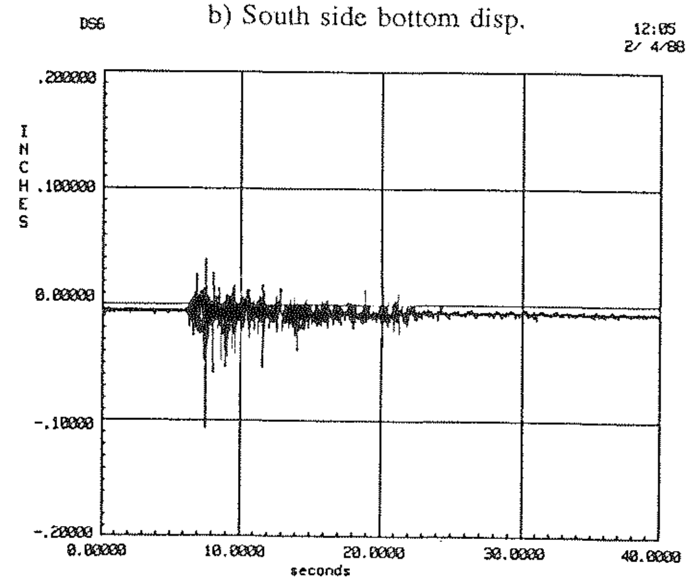
a) North side bottom disp.



b) South side bottom disp.



c) South side mid-height.



d) South side top disp.

FIGURE 4-4 Displacement Responses to EC1HV2.5 Excitation, Tank No. 2

stopping, the tank was permanently tilted toward the south with a maximum lateral top displacement on the order of 1 to 2 inches. Minor leaks were experienced around the buckled area. No leaks were observed along the vertical splice joint, indicating that the epoxy adhesive worked well. (The entire buckling process during the shaking was successfully recorded on videotape. Viewing of the tape can be arranged with the Structure Engineering Lab., Department of Civil Eng., SUNY/Buffalo.)

A group of pictures showing the overall buckling mechanism, and various local views are presented in Figure 4-5 and 4-6. Permanent inelastic deformations were sustained around the entire tank from the base to approximately 4 inches above. In other words, almost 1/10 of the total tank height suffered from plastic deformations. The tank shell at the south side was shortened 1.0 inches due to the compressive settlement caused by the two wrinkles. Elsewhere, the tank shell settled less amounts, but of the same order of magnitude.

Around the north side of the tank wall, one outward going wrinkle of the elephant foot type of bulge was developed at about 2 inches above the base; while two wrinkles were obtained around the south side. Comparing this to the field damage observation, as shown in Figure 2-1, it can be easily recognized that the experimentally obtained two wrinkled plastic deformation shape is in agreement to those observed in field damage of 'real tanks' subjected to 'real earthquakes'.

At the east and west sides, failure was due to excessive shear developed after the north and south sides lost their stability. The post-buckling shape at these locations showed an inward shear type of buckling. This was different from the post-buckling shapes at the north and south sides along the excitation axis.

A considerable number of the electrical resistance type instrument malfunctioned due to excessive inelastic deformations. The residual strains at station 15 indicated that the actual inelastic deformation extended at least 6 inches (3/20 height) above the base. (The relative displacements shown in Figure 4-7 may not reflect true values, because of 'stockings' sometimes observed due to the vertical settlement. However, at least these data give a quantitative indication of the displacement values associated with the failure mechanism, and the real maximum values should be no less than the indicated instrumented values.)

It is interesting to note that the residual lateral displacement of the shell at the north side bottom reached 1.3 inches, while the south side bottom returned almost to zero. This was because the measuring instrument at the south side was located in the valley between two wrinkles. At the top of the north side of the tank, the residual displacement was about 1.5 inches. The acceleration responses remained relatively low due to the inelastic deformation as well as non-resonance of the shell membrane.

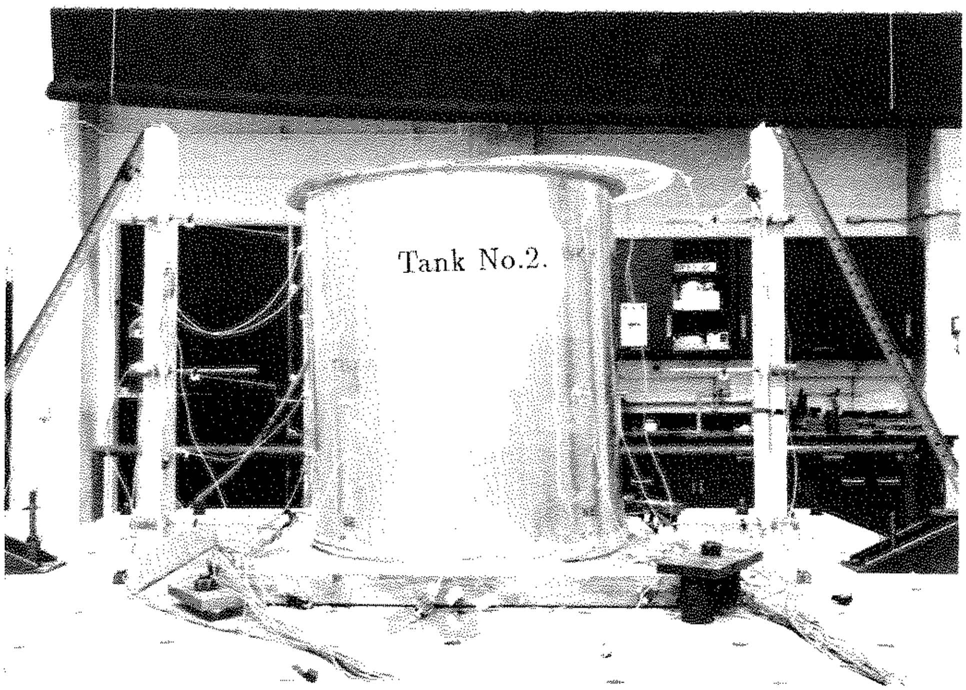


FIGURE 4-5 Overall Buckling Mechanism of Tank No. 2 After EC1H3 Excitation

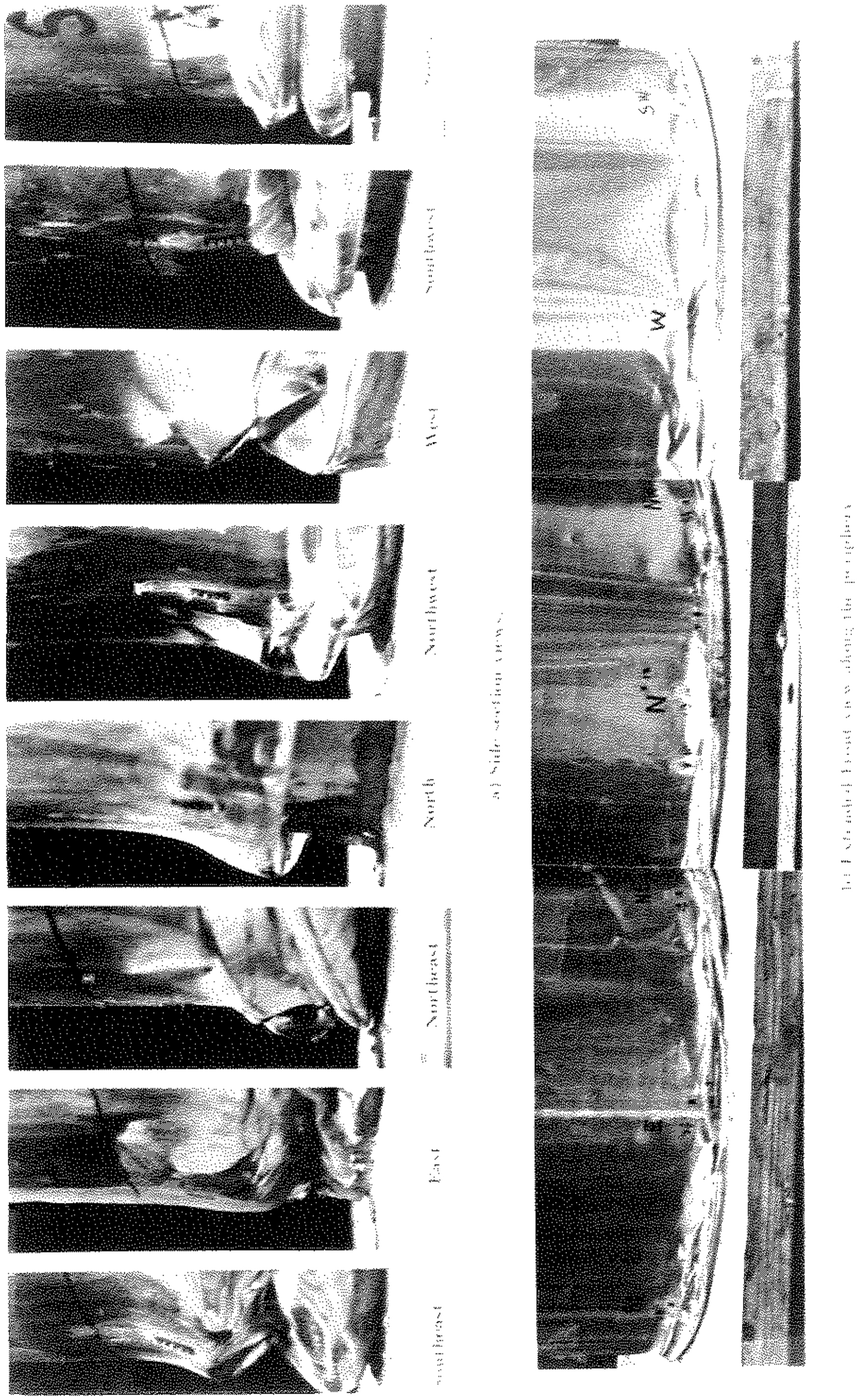
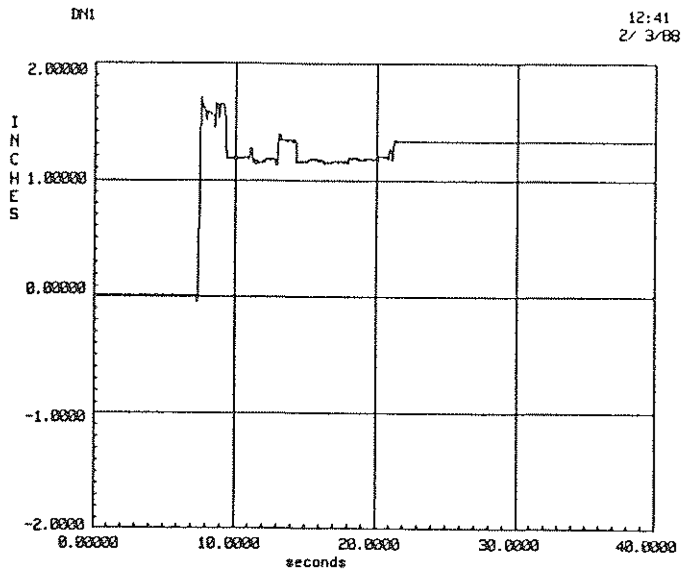
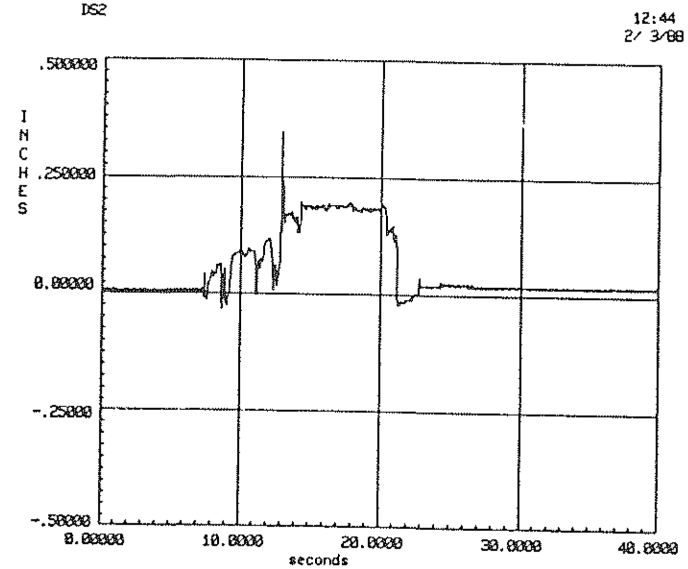


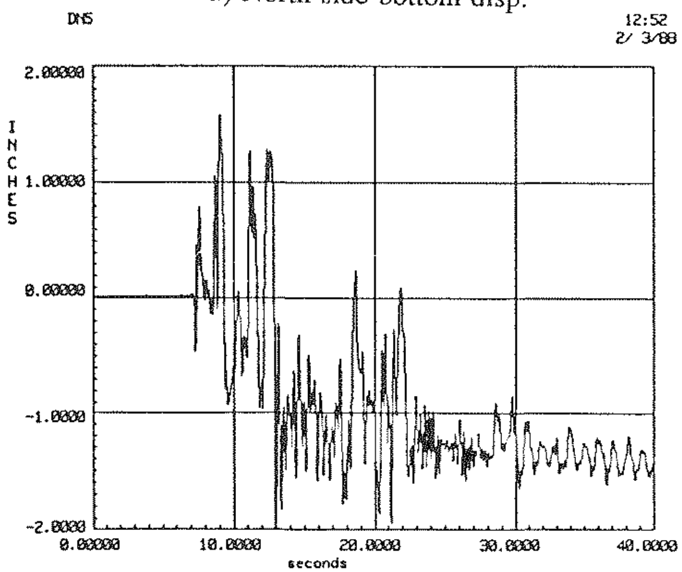
FIGURE 4-6 Buckling Shape of Tank No. 2



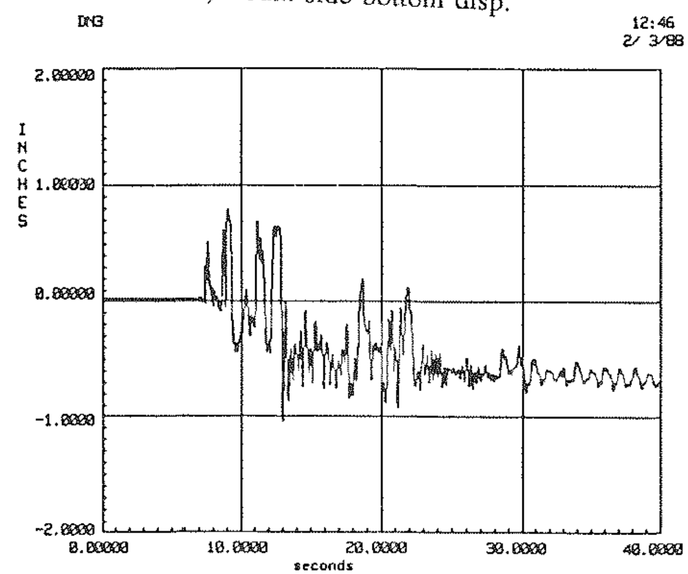
a) North side bottom disp.



b) South side bottom disp.



c) North side mid-height.



d) Norht side top-disp.

FIGURE 4-7 Displacement Responses to EC1H3 Excitation, Tank No. 2

SECTION 5
DISCUSSION OF TEST RESULTS

5.1 General Tank Performances

From these test results and from observations of the performance of full scale structures subjected to strong earthquakes, it is clear that ground supported cylindrical storage tanks are vulnerable to seismic excitations. Unlike other structural systems, due to the special form of the tank shell, ductility does not provide a significant reserve in energy absorbtion. Any inelastic deformation will contribute to failure of the shell, and there may be induced inelastic instability of the 'elephant foot bulge' type. Therefore, in all circumstances, tanks should be designed to respond elastically to strong earthquakes.

5.2 'Elephant Foot Bulge' Mechanism

One of the most significant achievements of this experimental study is the successful reproduction of the buckling mechanism associated with 'elephant foot bulge' type of instability. Many previous uncertainties now can be more clearly defined and understood.

First, concerning the question of whether or not the horizontal component of the excitation is the major cause of the 'elephant foot bulge': it can be concluded from these test results that horizontal excitation alone can/and, at least for the case studied, is the major source to induce the 'elephant foot bulge'. (In current seismic design practice, the horizontal component of the seismic excitation is considered as the main source of seismic loading for most structures. This principle seems to be equally applicable to seismic design of liquid storage tanks, including consideration of the 'elephant foot bulge'.)

Secondly, on the buckling mechanism of the 'elephant foot bulge': the test results agreed well with those observed in field damage reports. The failure patterns of the test model tanks were almost identical to those observed in full scale structures subjected to actual earthquakes. The critical region is concentrated near the base of the shell, where both the axial compression and hoop tension are maximum. The hydrodynamic pressure developed during seismic excitation causes the stresses in the shell to increase into the inelastic range. 'Elephant foot bulge' can be characterized, therefore, as a combined strength and instability problem, rather than one of elastic 'bifurcation' buckling.

The lateral deformation pattern of the shell due to 'elephant foot bulge' may consist of one or several wrinkles, extending a considerable distance around the shell circumference. The deformation shape of the 'bulge' is consistent with the static outward deformation shape due to hydrostatic pressure of the liquid.

For anchored tanks, the response mechanisms associated with the 'elephant foot bulge' of the model tanks appeared to have two forms. One can be characterized as cantilever bending, with circumferential wave n and vertical wave m both equal to one, as demonstrated from the responses of tank No. 2. The other form can be described as a membrane resonance mechanism with a combination of circumferential wave number n not equal to one - as indicated from partial responses of tank No. 1. While the cantilever bending form has been well recognized by the engineering profession, the shell membrane resonance mechanism is seldom considered. (Actually, these require much less intensity of shaking to induce buckling by resonance than by cantilever bending, provided the shaking source contains the resonance frequency content. Since earthquakes do contain high energy input in many different frequency ranges, analysis and design of tanks with lower aspect ratio ($h/R < 3.0$) should presume both possibilities.)

5.3 Critical Buckling Criteria

Current U.S. design practice of liquid storage tanks [7, 8] limits the maximum allowable compressive stress in the tank shell to a certain prescribed value to insure against tank shell instability of both the 'diamond buckling' and 'elephant foot bulge'. There are two problems associated with this practice. First, 'diamond buckling' and 'elephant foot bulge' are fundamentally different processes, and this is not recognized. Secondly, the uni-directional stress criteria now specified does not reflect the true stress state behavior of the tank shell.

'Diamond buckling' and 'elephant foot bulge' represent different mode of instability. They should be treated differently in design. While 'diamond buckling' represents anelastic bifurcation type of problem, the 'elephant foot bulge' represents a combined inelastic strength and stability problem. Although previous tests results [32] suggest that current criteria are overly-conservative for 'diamond buckling', this study has demonstrated that such criteria are not necessarily safe for 'elephant foot bulge'.

The interaction effect between circumferential hoop tension and axial compression should be considered when evaluating allowable axial compressive stress. This is because at the critical sections, where bucking occurs, the shell is subjected to significant bi-axial stresses. The magnitude of the hoop tension may be of non-negligible order, even when considering hydrostatic loading alone. Once hydrodynamic loading due to seismic excitation is imposed, this hoop tension increases considerably at the exact location where the axial compression is greatest. The stress resultant thus obtained tends to reach Tresca's yielding condition and inelastic instability results, as these tests revealed. 'Elephant foot bulge' is more related to this combined stress effect.

From these and other test results, it appears that the interaction curves recently presented in the New Zealand Code [9] give more reasonable buckling predictions. When judged by such a combined maximum shear stress ratio $(\sigma_1 - \sigma_3)/\sigma_y$, the obtained value at initiation of 'bulge' was

0.70 for tank No. 1 and 0.92 for tank No. 2, while the prediction was on the order of 0.72. Since the strengthening effect of the boundary condition to tank No. 1 had less contribution than to tank No. 2, it is considered that the critical value from tank No. 1 is more representative of real tanks. Larger differences, on the other hand, would be obtained if the buckling criteria were judged by uni-directional allowable compressive stress alone. (The obtained critical axial compressive stresses for tanks No. 1 and No. 2 were 36% and 78% of the classical buckling value of uniformly axially loaded cylindrical shells, respectively. The prediction would be on the order of 42% of the classical value.)

5.4 Vertical Excitation Effects

The effect of vertical excitation on tanks is important. It should not be neglected in seismic design. The upward acceleration of the vertical ground motion increases the specific gravity of the contained liquid by a ratio of a/g. The stresses of the tank shell are increased in the same proportion. (Moreover, the dynamic magnification factor of 2(a/g), due to the 'breathing mode' of the tank to vertical excitation, as suggested by several investigators [24, 22], may also be encountered. Such a magnifying effect was not observed during the tests of this study.) By adding to the shell hoop tension a term which is proportional to the ratio a/g, this effect can be reasonably estimated. Such a provision has been included in the recent AWWA standard but not in the API standard.

When horizontal and vertical excitations are combined, the situation is worse. As indicated by test results of tank No. 2, the resulting stress state generally follows the linear superposition rule, when these component motions are combined. It is obvious that the combined effect should be considered in seismic loading combinations.

There has been raised in the literature the question of whether the 'elephant foot bulge' is/can be induced by vertical excitation alone. These test results indicate that there exists the possibility that an 'elephant foot bulge' might be excited by vertical acceleration alone, if the seismic vertical motion is extremely strong. By comparing the peak strain magnitude associated with SN5V vertical excitation of Figure 4-1b to that of SN5H horizontal excitation of Figure 4-1a, such a possibility should be recognized. Further experimental studies on this problem need to be carried out, and have been proposed for inclusion in the next phase of this investigation.

5.5 Frequencies, Sloshing Waves, Anchorage, etc.

The natural frequencies and mode shapes of the tank system tested are clustered very closely, which made structural identification difficult. This is due to the fact that many combinations of circumferential n and vertical m waves exist. To separate these frequencies and mode shapes, analytical methods combined with experimental results were required.

The sloshing wave forms observed at the top of the cylinder during the tests were irregular. Only during the excitation of SN5H to tank No. 2, did the wave move along the excitation axis. During most of the other tests, the waves moved both along the excitation axis and perpendicular to it. (This was probably due to the fact that when the wave peaks reached the top lid, they split into perpendicular motions. Large components of circular wave motion were also observed, suggesting an imperfect bias.)

Since the tank models buckled at the fixed boundary condition, which is the 'best end condition' that can be achieved, good anchorage does not prevent tanks from buckling. However, the critical buckling stress associated with the better anchored tank No. 2 was considerably higher than that associated with the 'less rigidly' anchored tank No. 1.

Although a number of initial fabrication imperfections were observed away from the base of the model tanks, the test result indicates that those imperfections had little influence on the buckling of the 'elephant foot' type. This is for two reasons. First, the existence of high hoop tension, due to hydrostatic pressure, considerably reduces the effect of these types of imperfections. Secondly, since the mechanism is characterized by progressive instability rather than bifurcation, as loads increase the relative magnitude of the imperfection decreases.

SECTION 6
SUMMARY AND CONCLUSIONS

Based upon the investigation carried out in this shaking table study of cylindrical liquid storage tanks, the following summary and conclusions can be made:

1. The most significant result is that the phenomena of 'elephant foot bulge' was for the first time successfully reproduced in the laboratory on relatively large scale aluminum tank models under seismic base excitations. The obtained post buckling shapes of the tank shells were almost identical to those observed in field damage reports. The failure mechanism of the model tanks appeared to have been of the combined cantilever bending and circumferential membrane resonance form.

2. The critical compressive stresses observed in the tests, at initiation of the 'elephant foot bulge', were considerably lower than those reported by others for 'diamond buckling'. This suggests that tank shell instability failures due to 'diamond buckling' and 'elephant foot bulge' represent different phenomena and should be treated differently in design. Current U.S. design criteria do not recognize this difference and therefore may be unsafe for certain types or sizes of tanks. An interaction curve similar to the recently developed New Zealand provision is one possible way to handle these two distinctively different problems.

3. For the type and size of tanks tested, the primary loading source to cause the 'elephant foot bulge' was horizontal seismic excitation. This failure mechanism was observed for tank No. 1 under pure horizontal excitation. (Nevertheless, vertical excitation effects also should be considered in design, because significant stresses may be generated in the shell due to vertical excitation, as the test results indicate.)

4. Post buckling behavior of the tank shell was satisfactory. The functional performance and lateral stiffness of the tank was not significantly altered after the tank shell experienced slight 'elephant foot bulge'. It may therefore be speculated that slightly buckled tanks may be reused after careful inspection. Moreover, good anchorage design of the tank does not prevent the shell from buckling. 'Elephant foot bulge' may occur not only for unanchored tanks, but also for anchored ones.

SECTION 7
REFERENCES

1. Davidson, B.J., "Steel and Stainless Steel Tanks," in Edgecombe Earthquake Reconnaissance Report edited by Pender, M.J. et al., *Bull. New Zealand Nat. Soc. Earthq. Eng.*, Vol. 20, No. 3, Sept. 1987, pp. 234-238.

2. Hanson, R.D., "Behavior of Liquid Storage Tanks," *The Great Alaska Earthquake of 1964, Engineering*, National Academy of Sciences, Washington D.C., 1973, pp. 331-339.

3. Jennings, P.C., "Engineering Features of the San Fernando Earthquake, February 9, 1971," *Report No. EERL 71-02,* Earthq. Eng. Res. Lab., Pasadena, Calif., Jun., 1971.

4. Manos, G.C. and Clough R.W., "Tank Damage During the May 1983 Coalinga Earthquake," *Earthq. Eng. and Struct. Dyn.*, Vol. 13, 1985, pp. 449-466.

5. Rinne, J.E., "The Prince Williams Sounds, Alaska Earthquake of 1964," Vol.2-A, *Coast and Geodetic Survey*, U.S. Dept. of Commerce, Washington D.C., 1969, pp. 245-252.

6. Shih, C.F. and Babcock, D.D., "Buckling of Oil Storage Tanks in SPPl Tank Farm During the 1979 Imperial Valley Earthquake," *Proc. PVP Conf.*, ASME, San Antonio, Texas, 1984.

7. API Standard 650, "Welded Steel Tanks of Oil Storage," American Petroleum Institute, Washington D.C., Eighth Edition, November, 1988.

8. AWWA-D100, "AWWA Standard for Welded Steel Tanks for Water Storage," American Water Works Association, Denver, Colorado, 1987.

9. "Seismic Design of Storage Tanks," Recommendations of a study group of NZSEE, Edited by Priestley, J.N., *New Zealand National Society for Earthquake Engineering*, Dec. 1986.

10. Jia, Z.H., "Buckling Resistance of Cylindrical Liquid Storage Tanks Under Earthquake Excitation," *Ph.D. Dissertation*, State University of New York at Buffalo, Buffalo, New York, 1988.

11. Haroun, M.A. and Housner, G.W., "Dynamic Characteristics of Liquid Storage Tanks," *Journal Eng. Mech. Div.*, ASCE, Vol. 108, No. EM5, Oct., 1982, pp. 783-800.

12. Housner, G.W., "Dynamic Pressure on Accelerating Fluid Containers," *Bull. Seism. Soc. Am.*, Vol. 47, 1957, pp. 15-35.

13. Natsiavas, S., "Response and Failure of Liquid-Filled Tanks under Base Excitation," *Ph.D. Thesis*, CalTech, Pasadena, California, 1987.

14. Veletsos, A.S., "Seismic Effects in Flexible Liquid Storage Tanks," *Proc. 5th WCEE*, Vol. 1, Rome, Italy, 1974, pp. 630-639.

15. Veletsos, A.S. and Yang, J.Y., "Earthquake Response of Liquid Storage Tanks," *Adv. Civil Eng. through Eng. Mech.*, ASCE, Releigh, N.C., 1977, pp. 1-24.

16. Tso, W.K., Ghobarah, A. and Yee, S.K., "Seismic Design Forces for Cylindrical Tanks on Ground," *Can. Journal Civil Eng.*, Vol.12, 1985, pp. 12-23.

17. Wozmiak, T.S. and Mitchell, W.W., "Basis of Seismic Design of Storage Tanks," *Advances in Storage Tank Design*, API, 43rd Midyear Meeting, Toronto, Canada, May, 1978, pp. 1-34.

18. Clough, D.P., "Experimental Evaluation of Seismic Design Method of Broad Cylindrical Tanks," *Report No. EERC 77-10*, Earthq. Eng. Res. Ctr., Univ. of Cal., Berkeley, CA., 1977.

19. Kana, D.D., "Status and Research Needs for Prediction of Seismic Response in Liquid Containers," *Nuclear Eng. and Design*, Vol. 69, 1982, pp. 205-221.

20. Niwa, A., "Seismic Behavior of Tall Liquid Storage Tanks," *Report No.EERC 78-04*, Earthq. Eng. Res. Ctr., Univ. of Cal., Berkeley, C.A., Feb., 1978.

21. Reinhorn, A.M., Park, Y.J., Saldana, W.L. and Wang, C.T., "Evaluation of Seismic Response of Plastic Fluid Storage Tanks," Technical Report NCEER-88- (Scheduled for publication), Buffalo, New York, 1988.

22. Chen, G.Q., "Elephant Foot Phenomenon in Liquid Storage Tanks," Proc. PVP Conf., ASME, 1983, pp.107-112.

23. Haroun, M.A. AND Tayel, M.A., "Response of Tanks to Vertical Seismic Excitation," *Earthq. Eng. & Stru. Dyn.*, Vol. 13, 1985, pp. 587-596.

24. Marchaj, T.J., "Importance of Vertical Accelerations in the Design of Liquid Storage Tanks," *Proc. 2nd U.S. Conf. Earthq. Eng.*, Stanford, 1979, pp. 146-155.

25. Ketter, R.L., "The UB Seismic Simulator," *Engineering Progress of Western New York*, Vol. 3, No. 2, SUNY/Buffalo, Buffalo, New York, 1983-1984.

26. Baker, E.H. et al., *Structural Analysis of Shells*, McGraw-Hill Book Co., New York, 1972.

27. Gerard, G. and Becker, H., "Hand Book of Structural Stability, part 3 - Buckling of Curved Plates and Shells," NACA TN3783, Aug., 1957.

28. Ketter, R.L., *Lecture Note on Structural Stability*, SUNY/Buffalo, 1985.

29. Timoshenko, S.P. and Gere, J.M., *Theory of Elastic Stability*, 2nd Ed., McGraw-Hill, New York, 1961.

30. Timoshenko, S.P. and Voinowsky-Krieger, S., *Plates and Shells*, 2nd Ed., McGraw-Hill, New York, 1959.

31. Choi, H.S., Tanami, T and Hanger, Y., "Failure Tests of Cantilevered Cylindrical Shells under a Transverse Load," *Shells, Memberances and Space Frames*, Proc. IASS Sym., Osaka, Vol.1, pp. 265-272, 1986.

32. Manos, G.C. and Clough R.W., "Dynamic Response Correlation of Cylindrical Tanks," *Lifeline Eq. Eng.: Performance Design and Construction*, edited by Cooper, J.D., 1985, pp. 190-211.

33. Niwa, A. and Clough, R.W., "Buckling of Cylindrical Liquid Storage Tanks under Earthquake Loading," *Journal of Earthq. Eng. and Struct. Dyn.*, V. 10, 1982, pp. 107-122.

34. Shih, C.F., "Failure of Cylindrical Liquid Storage Tanks," *Ph.D. Thesis*, California Institute of Technology, 1982.

35. Nagashima H., Kokubo, K., Takayanagi, M., Saitoh, K. and Imaoka, T., "Experimental Study on the Dynamic Buckling of Cylindrical Tanks," *JSME Int. J.*, Vol. 30, No. 263, May, 1987, pp. 737-746.

36. Yamaki, N., Naito, K. and Sato, T., "Buckling of Circular Cylindrical Shells Under Combined Action of a Transverse Edge Load and Hydrostatic Pressure," *Thin-Walled Structure*, edited by Rhodes, J. and Walker, A.E., pp. 286-298, 1980.

37. Zui, H., Shinke, T. and Nishimura, A., "Experimental Studies on Earthquake Response Behavior of Cylindrical Tanks," *Journal Pressure Vessel Technology*, Vol. 109, pp. 50-57, Feb. 1987.

NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
LIST OF PUBLISHED TECHNICAL REPORTS

The National Center for Earthquake Engineering Research (NCEER) publishes technical reports on a variety of subjects related to earthquake engineering written by authors funded through NCEER. These reports are available from both NCEER's Publications Department and the National Technical Information Service (NTIS). Requests for reports should be directed to the Publications Department, National Center for Earthquake Engineering Research, State University of New York at Buffalo, Red Jacket Quadrangle, Buffalo, New York 14261. Reports can also be requested through NTIS, 5285 Port Royal Road, Springfield, Virginia 22161. NTIS accession numbers are shown in parenthesis, if available.

NCEER-87-0001 "First-Year Program in Research, Education and Technology Transfer," 3/5/87, (PB88-134275/AS).

NCEER-87-0002 "Experimental Evaluation of Instantaneous Optimal Algorithms for Structural Control," by R.C. Lin, T.T. Soong and A.M. Reinhorn, 4/20/87, (PB88-134341/AS).

NCEER-87-0003 "Experimentation Using the Earthquake Simulation Facilities at University at Buffalo," by A.M. Reinhorn and R.L. Ketter, to be published.

NCEER-87-0004 "The System Characteristics and Performance of a Shaking Table," by J.S. Hwang, K.C. Chang and G.C. Lee, 6/1/87, (PB88-134259/AS).

NCEER-87-0005 "A Finite Element Formulation for Nonlinear Viscoplastic Material Using a Q Model," by O. Gyebi and G. Dasgupta, 11/2/87, (PB88-213764/AS).

NCEER-87-0006 "Symbolic Manipulation Program (SMP) - Algebraic Codes for Two and Three Dimensional Finite Element Formulations," by X. Lee and G. Dasgupta, 11/9/87, (PB88-219522/AS).

NCEER-87-0007 "Instantaneous Optimal Control Laws for Tall Buildings Under Seismic Excitations," by J.N. Yang, A. Akbarpour and P. Ghaemmaghami, 6/10/87, (PB88-134333/AS).

NCEER-87-0008 "IDARC: Inelastic Damage Analysis of Reinforced Concrete Frame - Shear-Wall Structures," by Y.J. Park, A.M. Reinhorn and S.K. Kunnath, 7/20/87, (PB88-134325/AS).

NCEER-87-0009 "Liquefaction Potential for New York State: A Preliminary Report on Sites in Manhattan and Buffalo," by M. Budhu, V. Vijayakumar, R.F. Giese and L. Baumgras, 8/31/87, (PB88-163704/AS). This report is available only through NTIS (see address given above).

NCEER-87-0010 "Vertical and Torsional Vibration of Foundations in Inhomogeneous Media," by A.S. Veletsos and K.W. Dotson, 6/1/87, (PB88-134291/AS).

NCEER-87-0011 "Seismic Probabilistic Risk Assessment and Seismic Margins Studies for Nuclear Power Plants," by Howard H.M. Hwang, 6/15/87, (PB88-134267/AS). This report is available only through NTIS (see address given above).

NCEER-87-0012 "Parametric Studies of Frequency Response of Secondary Systems Under Ground-Acceleration Excitations," by Y. Yong and Y.K. Lin, 6/10/87, (PB88-134309/AS).

NCEER-87-0013 "Frequency Response of Secondary Systems Under Seismic Excitation," by J.A. HoLung, J. Cai and Y.K. Lin, 7/31/87, (PB88-134317/AS).

NCEER-87-0014 "Modelling Earthquake Ground Motions in Seismically Active Regions Using Parametric Time Series Methods," by G.W. Ellis and A.S. Cakmak, 8/25/87, (PB88-134283/AS).

NCEER-87-0015 "Detection and Assessment of Seismic Structural Damage," by E. DiPasquale and A.S. Cakmak, 8/25/87, (PB88-163712/AS).

NCEER-87-0016 "Pipeline Experiment at Parkfield, California," by J. Isenberg and E. Richardson, 9/15/87, (PB88-163720/AS).

NCEER-87-0017	"Digital Simulation of Seismic Ground Motion," by M. Shinozuka, G. Deodatis and T. Harada, 8/31/87, (PB88-155197/AS). This report is available only through NTIS (see address given above).
NCEER-87-0018	"Practical Considerations for Structural Control: System Uncertainty, System Time Delay and Truncation of Small Control Forces," J.N. Yang and A. Akbarpour, 8/10/87, (PB88-163738/AS).
NCEER-87-0019	"Modal Analysis of Nonclassically Damped Structural Systems Using Canonical Transformation," by J.N. Yang, S. Sarkani and F.X. Long, 9/27/87, (PB88-187851/AS).
NCEER-87-0020	"A Nonstationary Solution in Random Vibration Theory," by J.R. Red-Horse and P.D. Spanos, 11/3/87, (PB88-163746/AS).
NCEER-87-0021	"Horizontal Impedances for Radially Inhomogeneous Viscoelastic Soil Layers," by A.S. Veletsos and K.W. Dotson, 10/15/87, (PB88-150859/AS).
NCEER-87-0022	"Seismic Damage Assessment of Reinforced Concrete Members," by Y.S. Chung, C. Meyer and M. Shinozuka, 10/9/87, (PB88-150867/AS). This report is available only through NTIS (see address given above).
NCEER-87-0023	"Active Structural Control in Civil Engineering," by T.T. Soong, 11/11/87, (PB88-187778/AS).
NCEER-87-0024	Vertical and Torsional Impedances for Radially Inhomogeneous Viscoelastic Soil Layers," by K.W. Dotson and A.S. Veletsos, 12/87, (PB88-187786/AS).
NCEER-87-0025	"Proceedings from the Symposium on Seismic Hazards, Ground Motions, Soil-Liquefaction and Engineering Practice in Eastern North America," October 20-22, 1987, edited by K.H. Jacob, 12/87, (PB88-188115/AS).
NCEER-87-0026	"Report on the Whittier-Narrows, California, Earthquake of October 1, 1987," by J. Pantelic and A. Reinhorn, 11/87, (PB88-187752/AS). This report is available only through NTIS (see address given above).
NCEER-87-0027	"Design of a Modular Program for Transient Nonlinear Analysis of Large 3-D Building Structures," by S. Srivastav and J.F. Abel, 12/30/87, (PB88-187950/AS).
NCEER-87-0028	"Second-Year Program in Research, Education and Technology Transfer," 3/8/88, (PB88-219480/AS).
NCEER-88-0001	"Workshop on Seismic Computer Analysis and Design of Buildings With Interactive Graphics," by W. McGuire, J.F. Abel and C.H. Conley, 1/18/88, (PB88-187760/AS).
NCEER-88-0002	"Optimal Control of Nonlinear Flexible Structures," by J.N. Yang, F.X. Long and D. Wong, 1/22/88, (PB88-213772/AS).
NCEER-88-0003	"Substructuring Techniques in the Time Domain for Primary-Secondary Structural Systems," by G.D. Manolis and G. Juhn, 2/10/88, (PB88-213780/AS).
NCEER-88-0004	"Iterative Seismic Analysis of Primary-Secondary Systems," by A. Singhal, L.D. Lutes and P.D. Spanos, 2/23/88, (PB88-213798/AS).
NCEER-88-0005	"Stochastic Finite Element Expansion for Random Media," by P.D. Spanos and R. Ghanem, 3/14/88, (PB88-213806/AS).
NCEER-88-0006	"Combining Structural Optimization and Structural Control," by F.Y. Cheng and C.P. Pantelides, 1/10/88, (PB88-213814/AS).
NCEER-88-0007	"Seismic Performance Assessment of Code-Designed Structures," by H.H-M. Hwang, J-W. Jaw and H-J. Shau, 3/20/88, (PB88-219423/AS).

NCEER-88-0008 "Reliability Analysis of Code-Designed Structures Under Natural Hazards," by H.H-M. Hwang, H. Ushiba and M. Shinozuka, 2/29/88, (PB88-229471/AS).

NCEER-88-0009 "Seismic Fragility Analysis of Shear Wall Structures," by J-W Jaw and H.H-M. Hwang, 4/30/88, (PB89-102867/AS).

NCEER-88-0010 "Base Isolation of a Multi-Story Building Under a Harmonic Ground Motion - A Comparison of Performances of Various Systems," by F-G Fan, G. Ahmadi and I.G. Tadjbakhsh, 5/18/88, (PB89-122238/AS).

NCEER-88-0011 "Seismic Floor Response Spectra for a Combined System by Green's Functions," by F.M. Lavelle, L.A. Bergman and P.D. Spanos, 5/1/88, (PB89-102875/AS).

NCEER-88-0012 "A New Solution Technique for Randomly Excited Hysteretic Structures," by G.Q. Cai and Y.K. Lin, 5/16/88, (PB89-102883/AS).

NCEER-88-0013 "A Study of Radiation Damping and Soil-Structure Interaction Effects in the Centrifuge," by K. Weissman, supervised by J.H. Prevost, 5/24/88, (PB89-144703/AS).

NCEER-88-0014 "Parameter Identification and Implementation of a Kinematic Plasticity Model for Frictional Soils," by J.H. Prevost and D.V. Griffiths, to be published.

NCEER-88-0015 "Two- and Three- Dimensional Dynamic Finite Element Analyses of the Long Valley Dam," by D.V. Griffiths and J.H. Prevost, 6/17/88, (PB89-144711/AS).

NCEER-88-0016 "Damage Assessment of Reinforced Concrete Structures in Eastern United States," by A.M. Reinhorn, M.J. Seidel, S.K. Kunnath and Y.J. Park, 6/15/88, (PB89-122220/AS).

NCEER-88-0017 "Dynamic Compliance of Vertically Loaded Strip Foundations in Multilayered Viscoelastic Soils," by S. Ahmad and A.S.M. Israil, 6/17/88, (PB89-102891/AS).

NCEER-88-0018 "An Experimental Study of Seismic Structural Response With Added Viscoelastic Dampers," by R.C. Lin, Z. Liang, T.T. Soong and R.H. Zhang, 6/30/88, (PB89-122212/AS).

NCEER-88-0019 "Experimental Investigation of Primary - Secondary System Interaction," by G.D. Manolis, G. Juhn and A.M. Reinhorn, 5/27/88, (PB89-122204/AS).

NCEER-88-0020 "A Response Spectrum Approach For Analysis of Nonclassically Damped Structures," by J.N. Yang, S. Sarkani and F.X. Long, 4/22/88, (PB89-102909/AS).

NCEER-88-0021 "Seismic Interaction of Structures and Soils: Stochastic Approach," by A.S. Veletsos and A.M. Prasad, 7/21/88, (PB89-122196/AS).

NCEER-88-0022 "Identification of the Serviceability Limit State and Detection of Seismic Structural Damage," by E. DiPasquale and A.S. Cakmak, 6/15/88, (PB89-122188/AS).

NCEER-88-0023 "Multi-Hazard Risk Analysis: Case of a Simple Offshore Structure," by B.K. Bhartia and E.H. Vanmarcke, 7/21/88, (PB89-145213/AS).

NCEER-88-0024 "Automated Seismic Design of Reinforced Concrete Buildings," by Y.S. Chung, C. Meyer and M. Shinozuka, 7/5/88, (PB89-122170/AS).

NCEER-88-0025 "Experimental Study of Active Control of MDOF Structures Under Seismic Excitations," by L.L. Chung, R.C. Lin, T.T. Soong and A.M. Reinhorn, 7/10/88, (PB89-122600/AS).

NCEER-88-0026 "Earthquake Simulation Tests of a Low-Rise Metal Structure," by J.S. Hwang, K.C. Chang, G.C. Lee and R.L. Ketter, 8/1/88, (PB89-102917/AS).

NCEER-88-0027 "Systems Study of Urban Response and Reconstruction Due to Catastrophic Earthquakes," by F. Kozin and H.K. Zhou, 9/22/88, to be published.

NCEER-88-0028	"Seismic Fragility Analysis of Plane Frame Structures," by H.H-M. Hwang and Y.K. Low, 7/31/88, (PB89-131445/AS).
NCEER-88-0029	"Response Analysis of Stochastic Structures," by A. Kardara, C. Bucher and M. Shinozuka, 9/22/88, (PB89-174429/AS).
NCEER-88-0030	"Nonnormal Accelerations Due to Yielding in a Primary Structure," by D.C.K. Chen and L.D. Lutes, 9/19/88, (PB89-131437/AS).
NCEER-88-0031	"Design Approaches for Soil-Structure Interaction," by A.S. Veletsos, A.M. Prasad and Y. Tang, 12/30/88, (PB89-174437/AS).
NCEER-88-0032	"A Re-evaluation of Design Spectra for Seismic Damage Control," by C.J. Turkstra and A.G. Tallin, 11/7/88, (PB89-145221/AS).
NCEER-88-0033	"The Behavior and Design of Noncontact Lap Splices Subjected to Repeated Inelastic Tensile Loading," by V.E. Sagan, P. Gergely and R.N. White, 12/8/88.
NCEER-88-0034	"Seismic Response of Pile Foundations," by S.M. Mamoon, P.K. Banerjee and S. Ahmad, 11/1/88, (PB89-145239/AS).
NCEER-88-0035	"Modeling of R/C Building Structures With Flexible Floor Diaphragms (IDARC2)," by A.M. Reinhorn, S.K. Kunnath and N. Panahshahi, 9/7/88.
NCEER-88-0036	"Solution of the Dam-Reservoir Interaction Problem Using a Combination of FEM, BEM with Particular Integrals, Modal Analysis, and Substructuring," by C-S. Tsai, G.C. Lee and R.L. Ketter, 12/31/88.
NCEER-88-0037	"Optimal Placement of Actuators for Structural Control," by F.Y. Cheng and C.P. Pantelides, 8/15/88.
NCEER-88-0038	"Teflon Bearings in Aseismic Base Isolation: Experimental Studies and Mathematical Modeling," by A. Mokha, M.C. Constantinou and A.M. Reinhorn, 12/5/88.
NCEER-88-0039	"Seismic Behavior of Flat Slab High-Rise Buildings in the New York City Area," by P. Weidlinger and M. Ettouney, 10/15/88, to be published.
NCEER-88-0040	"Evaluation of the Earthquake Resistance of Existing Buildings in New York City," by P. Weidlinger and M. Ettouney, 10/15/88, to be published.
NCEER-88-0041	"Small-Scale Modeling Techniques for Reinforced Concrete Structures Subjected to Seismic Loads," by W. Kim, A. El-Attar and R.N. White, 11/22/88.
NCEER-88-0042	"Modeling Strong Ground Motion from Multiple Event Earthquakes," by G.W. Ellis and A.S. Cakmak, 10/15/88, (PB89-174445/AS).
NCEER-88-0043	"Nonstationary Models of Seismic Ground Acceleration," by M. Grigoriu, S.E. Ruiz and E. Rosenblueth, 7/15/88.
NCEER-88-0044	"SARCF User's Guide: Seismic Analysis of Reinforced Concrete Frames," by Y.S. Chung, C. Meyer and M. Shinozuka, 11/9/88, (PB89-174452/AS).
NCEER-88-0045	"First Expert Panel Meeting on Disaster Research and Planning," edited by J. Pantelic and J. Stoyle, 9/15/88, (PB89-174460/AS).
NCEER-88-0046	"Preliminary Studies of the Effect of Degrading Infill Walls on the Nonlinear Seismic Response of Steel Frames," by C.Z. Chrysostomou, P. Gergely and J.F. Abel, 12/19/88.
NCEER-88-0047	"Reinforced Concrete Frame Component Testing Facility - Design, Construction, Instrumentation and Operation," by S.P. Pessiki, C. Conley, T. Bond, P. Gergely and R.N. White, 12/16/88, (PB89-174478/AS).

NCEER-89-0001 "Effects of Protective Cushion and Soil Compliancy on the Response of Equipment Within a Seismically Excited Building," by J.A. HoLung, 2/16/89.

NCEER-89-0002 "Statistical Evaluation of Response Modification Factors for Reinforced Concrete Structures," by H.H-M. Hwang and J-W. Jaw, 2/17/89.

NCEER-89-0003 "Hysteretic Columns Under Random Excitation," by G-Q. Cai and Y.K. Lin, 1/9/89.

NCEER-89-0004 "Experimental Study of 'Elephant Foot Bulge' Instability of Thin-Walled Metal Tanks," by Z-H. Jia and R.L. Ketter, 2/22/89.

National Center for Earthquake Engineering Research
State University of New York at Buffalo