To the Graduate Council:

I am submitting herewith a thesis written by Sagar Ramesh Kulkarni entitled "Verification of the Correlation between Peak Particle Velocity (PPV) and the Response of a Structure." I have examined the final paper copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

Dr. Earl E. Ingram, Major Professor

We have read this thesis
and recommend its acceptance:

Accepted for the Council:

Vice Chancellor and Dean of
Graduate Studies

VERIFICATION OF THE CORRELATION BETWEEN PEAK PARTICLE VELOCITY (PPV) AND THE RESPONSE OF A STRUCTURE

A Thesis
Presented for the
Master of Science
Degree
The University of Tennessee, Knoxville

Sagar Ramesh Kulkarni
August 2004

This thesis is

dedicated to my mother

Smt. Smita Ramesh Kulkarni

Acknowledgements

I shall take the opportunity to express my gratitude to the members of my committee for their support during this endeavor. I am especially thankful to my major professor, Dr. Earl E. Ingram, for his guidance, constructive criticism with a superb rapport all the way. I would also like to thank Dr. Edwin G. Burdette whose guidance, financial support and confidence in me made everything work out. I am certainly grateful to Dr. David. W. Goodpasture, Dr. Richard M. Bennett, Dr. Eric C. Drumm and Dr. J Harold Deatherage for the knowledge I gathered from them inside, as well as outside, the classroom. Thanks to all for making my learning process a pleasure.

This Master's Degree would not have been possible without the endless prayers and blessings, for me, of my uncles Prakash Gupte, Kiran Gupte, my aunt Rajani Gupte, my beloved grandmother Malati Gupte and rest of my family. My special thanks to Potdar and Parulekar family for turning the partings into a cheerful encouragement.

Finally, I would like to share my success with my friend Dr. Milind Gharpure; with whom I could always take things for granted. His calm, thoughtful and continuous encouragement throughout my highs and lows nourished me until I got here.

Thanks everybody for being there. Cheers!!!

Abstract

The purpose of this study is to verify the relation between the peak particle velocity (PPV) and the dynamic response of the structure. Based on the study of blast vibrations, the U.S Bureau of Mines correlated the PPV as the reliable parameter for gauging the strength of a blast wave. The prevalent practice of quantifying the ground wave motion is in terms of PPV. The bureau has published the threshold value of PPV=1/2 in/sec as the maximum safe value for a blast wave. This study simulates two structures having two different natural periods resembling the period of short buildings and medium height buildings. The simulation is done using finite element models in ADINA to study the response of the two structures for various ground accelerations while keeping the value of PPV constant. The process is repeated for a constant acceleration while varying the magnitude of PPV. Based on the results of these finite element models it is observed that the response is closely proportional to the PPV rather than the ground acceleration. Hence PPV appears to be the correct term to represent the strength of ground motion.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION AND OVERVIEW OF SOLUTION

Introduction

The repeated explosions that occur in quarries and mines have historically been a nuisance for people residing in the vicinity. The nuisance is in the form of vibrations that potentially cause structural damage. In 1930, the U.S Bureau of Mines initiated an extensive study to determine the various parameters influencing structural damage from blasting. There were two major types of damaging sources associated with the blasts; one was through the ground vibrations and another was the air blast. This thesis discusses damage due to the ground vibrations and does not address the air blast or any other source of structural damage. In this study by the Bureau, structural damage threshold values were identified which would be irrespective of any blast. Structural damages over and above these threshold values were inspected and were correlated with the "strength" of the blast.

The Bureau reports introduced a concept called "Peak Particle Velocity" (PPV) to quantify the "strength" of a blast. The magnitude of structural damage was then correlated to the PPV of the ground excitation. Guidelines were then developed to regulate blast strengths such that a PPV threshold is not exceeded.

Regulating blast strength based solely on PPV does not take into consideration the peak particle acceleration of the motion wave. It may happen that the peak particle velocity regulated by the miners is well within the safe permissible limit but the peak particle acceleration may cause a response of the structure above the safety limits. Hence the question arises as to whether the response is related to the peak particle velocity or peak particle acceleration. The finite element simulation research conducted at the University of Tennessee using the permissible value of PPV investigates the validity of the popular correlation between the response and peak particle velocity.

Overview of solution

In order to understand the exact correlation between the PPV and the response of the structure, it is important to develop and answer a set of questions which would act as the road map for this thesis. A key question is; for varying ground accelerations, do both structures react the same way to a given PPV. Keeping all the other parameters constant, the correlation between the PPV and response can be determined. The correlation can be easily represented by plotting the response vs. ground acceleration under the constant PPV. According to the current theory, if we have constant PPV and constant structural parameters the response should also be constant, but if we obtain changing response for changing ground acceleration under constant PPV then the validity of this term "PPV" will be disproved.

'*Dynamics of structures*' by Anil K. Chopra The response of a structure is a broad term. The structural engineer uses this term to describe how much internal force is induced in the structure due to the external dynamic force. A relationship is developed between the ground motion frequency, particle acceleration and particle velocity to the magnitude of shear force and bending moment induced in the structure. To calculate the bending moment at the base of a column, the horizontal force acting on the structure must be determined. Thus, summation of all the horizontal forces times their respective moment arms, depending on the degrees of freedom, is the total moment at the base. It is to be noted that the horizontal force acting on the structure is determined by the following equation of motion.

$$ku + c\dot{u} + m\ddot{u} = p(t) \text{ -------------------------------- Fundamental equation of motion}$$

$$p(t) = -ma \qquad \text{-------------------------------- External dynamic force (seismic)}$$

$$ku + c\dot{u} = -m\left(\ddot{u} + a\right)$$

also,

$$ku = m(pSA)$$

where,

k is Stiffness of structure, u is structural displacement

c is damping, $\dot{u}$ is structural velocity

m is mass, $\ddot{u}$ is structural acceleration

$p(t)$ is external dynamic force

pSA is Pseudo spectral acceleration

In case of typical civil engineering structures, the damping is generally below 5%. Hence the product of damping and velocity is generally of negligible contribution. Thus, spring constant (k) times displacement (u) forms the predominant magnitude of the internal force induced called a spring force. Spring force can also be represented in terms of mass times pseudo spectral acceleration (pSA). Hence internal force can either be represented in terms of spring constant or mass by multiplying with displacement or pSA respectively. The later expression is adopted in this thesis to represent the internal force.

The input data collected near the foundation of the structure included the ground displacement (Δ), ground acceleration or peak particle acceleration (ppa), ground motion frequency (f_g) and the peak particle velocity (PPV); and output was the structural acceleration ($\ddot{u}$). To study the response modification, it is important to determine the parameters of the equations of motion required for the analysis (mass (m) and stiffness (k) of the structure).

A damping coefficient was calculated by performing a lab test explained in chapter 3. The contribution of this parameter was found to be negligible; hence it was neglected in the actual simulation in ADINA. The research was confined to single degree of freedom cases catering to two classes of natural periods. The first case represented the stiffer class of structure having a period range between 0.1 to 0.3 seconds and the second prototype represents a more flexible class ranging from 0.3 to 0.6 seconds.

From the available sample quarry data as well as the literature review, the safe permissible value of PPV was noted and, keeping the peak velocity of the input wave motion equal to this PPV, a set of different magnitudes of ground acceleration was applied. The structural displacement and the structural acceleration were monitored according to the varying ground acceleration. The results were categorized as to be increasing, decreasing or constant due to constant PPV, and the comprehensive plots were obtained. The plots were then analyzed and the reason for the variation in the structural acceleration, if any, was deduced. The charts correlating each parameter with the response were obtained. These charts illustrated correlation between the peak particle velocity and the structural response.

CHAPTER 2

REVIEW OF LITERATURE

History

The first study of blast vibrations in U.S was reported in 1927 by Rockwell. He stated that normal blasting operations at a distance of 200 ft. to 300 ft. from the residential structure may be considered safe for the structures. The U.S Bureau of Mines conducted an extensive investigation on blast vibrations between 1930 and 1940. The book '*Vibrations from Blasting*' by David E. Siskind reports that Thoenen and Windes performed the first review of safe blasting in 1930. The results were reported in Bulletin 442 (1942). This study was based on the acceleration criteria ranging from 0.1g to 1.0g.

U.S. Bureau of Mines - (Bulletin 656) reviews the investigation of blast vibrations by Theonen and Windes in 1930. In the 1930 study, acceleration = 1.0g was considered as the index for the damage. Accelerations below 0.1g did not cause any damages, while the acceleration ranging between 0.1g to 1.0g affected the structures slightly. The wide ranges of safe values presented in this report were not able to convince the bureau on any definite term, which further necessitated a deeper study. A propagation law was set in this report which limited blasters to a specified separation distance, charge weight, and displacement amplitude. After World War II, rapid construction necessitated blasting operations, and the simultaneous urban sprawl brought the residential zones physically closer to the blasting quarries. Blasting operations started receiving more complaints from the residents in the vicinity about the damages. Thus, the technological data involved in the blasting operation was evaluated, and this issue became one of the important concerns amongst federal, state and local governments, blasting industries, explosives manufacturers, insurance companies, and scientists.

Bulletin 656 reports that in 1949 Crandell launched a concept of Energy Ratio (E.R) as the square of acceleration in ft/sec^2 over square of frequency in cycles per second. He found that the E.R. limits below 3.0 are safe for the structures, 3.0 to 6.0 indicates caution, while values above 6.0 were classified as danger for the structures. In 1950 Sutherland set up a new dimension to the prevalent study. He found that human beings are sensitive to the vibrations which do not have damaging potency. The study established two thresholds as the discomfort level and damage level waves. E.R. was still the basis of measuring the ground wave. Many states started defining the damage limits in terms of E.R. as low as 1.0. Pennsylvania adopted a limit in terms of displacement as low as 0.03 inches. Further, in 1958, based on a series of blast vibration studies, Langefors, Kihlstrom and Westerberg correlated damage with particle velocity for the first time. They defined the threshold of 2.8in/sec as the damaging velocity. Later in 1960 Edwards and Northwood studied the response of six structures against the blasting vibrations. They included the effects of all the kinematic properties like displacement, velocity, and acceleration in their study. This study stated that the particle velocity is the most reliable correlation with the damage and set the threshold value as 2.0 inch/sec.

'*Vibrations from Blasting*' by David E. Siskind reports that in 1962 Duvall and Fogelson repeated the vibration study reported in RI 5968. They introduced a measuring parameter "Peak Particle Velocity (PPV)", with the safety level again as 2 in/sec. This 2-in/sec value was adopted by many states, yet some places continued to receive complaints. Addressing these complaints, Illinois and Pennsylvania conducted an additional study in 1974 and 1975. In the following 5 years RI 8507 (Siskind et al. 1980b) recommended frequency based controls for prevention of threshold hairline or cosmetic cracks. Some industries responded adversely to these recommendations and emphasized that a reduction factor of up to 3 to 4 should be applied to the above recommended values in the low frequency range.

Evolution of PPV

U.S Bureau of Mines - (Bulletin 656) generated the safe blasting criterion based on the study of blast vibration and the monitored damage. The limiting displacement causing the structural damage was plotted against structural frequency on a logarithmic scale. A regression line with a slope of -1 was drawn on this plot pooling all the damaging displacements. The bureau determined that the slope of line corresponded to the constant particle velocity. Three such lines were established indicating the major, minor and safe blasting criteria. The bureau then started defining the safety criteria in terms of particle velocity. Velocity is the slope of displacement curve plotted against time and not against the structural frequency. Hence, the regression line generated by the bureau depicts the failure envelope in terms of displacement alone and should not be related to the velocity.

In 1949, Crandell used his parameter of energy ratio (E.R.) for measuring the blast vibration. The energy ratio is the ratio of the square of acceleration over square of frequency. Although not discussed by Crandell, the bureau stated that as acceleration is proportional to velocity keeping the frequency constant, the safe limits of E.R. were related to the safe limit of particle velocity. The E.R. limits of 3.0 corresponded with the bureau's result of 3.3 in/sec, and an upper limit of 6.0 corresponded with 4.7 in/sec velocity, which was concluded as the fair match between these two approaches.

Based on the extensive data made available by Edwards, Northwood, and Langefors, the bureau plotted the limiting particle velocity for various natural frequencies. Unlike the displacement vs. frequency plot, the regression line pooled all the damaging velocities into a perfect horizontal line which means that, irrespective of natural frequency of the structure, the given particle velocity will result in a more or less identical response.

YAN Zhixin and JIANG Ping (2002) highlighted the reasons for PPV to be taken as the criterion for the measurement and describing the vibrating strength. They also brought into notice that PPV eliminates the influence of soil factors as it does not vary greatly with change in soils. Based on the wave is having simple harmonic motion, $V=2\pi A/T$, where V is velocity, A is amplitude, and T is time. A and T fluctuate correspondingly, therefore, the ratio A/T is not affected significantly with changing soils. This keeps the velocity uninfluenced with changing soils. The ratio of vibrating V and the transmission speed C is the strain; i.e. $\varepsilon = V/C$. Also, in a structured object affected by vibration, strain is the ratio of stress to modulus of elasticity; $\varepsilon = \sigma / E$. Thus, both are correlated to get $\sigma_m = V_m E/C$. This is the correlation established between the structural stress and the velocity where suffix 'm' stands for maximum.

Monitoring of PPV

U.S. Bureau of Mines - (Bulletin 656). The bureau spent almost five years from 1930 to 1935 in deducing the precise instrumentation for the study of blast vibration.

Finally, the bureau recommended the use of seismic transducers for monitoring the blast vibrations. These seismic transducers converted the ground motion into a varying voltage or a spot of light which recorded on the moving strip of light sensitive paper. These transducers were designed to record particle displacement, velocity, or acceleration. The typical displacement seismograph was a portable type made up of a rigid case with leveling screws. The rigid case had a timing mechanism, recording mechanism, and three inertial pendulums in it having three axes mutually perpendicular. Motion was indicated by the deflection of a light beam on the photographic paper. The light beams were deflected by the mirrors, and the recording was magnified optically and mechanically up to 150 times greater than the actual motion. Thus, the seismograph with trace deflection of 0.1 inch can measure the displacement from 0.000667 inches to 0.0133 inches at frequencies ranging from 5 to 40 cps. The typical portable velocity transducers type had two units. One contained the gages while the other housed the amplifiers, batteries, light source, timing device, galvanometer, and recording camera. This transducer was designed as per the soil density; hence it was more advanced than the displacement transducers. The resonant frequency was between 2 and 5 cps. The light traces were recorded in a similar fashion on the moving strip by the camera with the time on the x-axis. The portable acceleration seismograph contained three external gages. The outputs of these gages were proportional to that of the displacement transducers. Two types of indicating and recording systems were used. Electronic circuits were used to deflect the meter, and peak acceleration was recorded by it while another system had a similar concept of galvanometer and light sensitive paper to record the wave form. According to the variables included in the study of blast vibrations, the bureau concluded that the magnitude of damage to the structures was closely related to the particle velocity rather than displacement or ground acceleration.

Anon (1992) has reviewed the procedures for monitoring the potential for structural damage. The input ground motion frequency is monitored by locating transducers next to the foundation of a structure, and the kinematic properties of the motion waves are recorded. The output or the response is measured at the critical location in the structure by measuring the strain at that location. Further, the response is typically correlated with ground motion frequency and the particle velocity of the ground motion. From the above records, the damage probabilities vs. the particle velocity curves are plotted, and thus damage limit is defined. This limitation enables one to determine the magnitude of peak particle velocity of the motion wave in order to be within the limiting values.

R.A.Farnfield (1996) examined the role of geophone frequency response over the recorded level of vibrations. PPV is the universally quoted term as a measurement of blast vibration and thus the degree of accuracy of measuring PPV is important. The blasting seismograph assembly consists of geophones, amplifiers, A/D converters and a computer. Geophones produce a voltage output proportional to vibration velocity, which is amplified by the amplifiers. The A/D converter converts the analogue signal into digital signal, which in turn gets recorded in the computer. A shaking table test is performed to obtain the comparative tests between various transducer types. In this test two geophones having different resonant frequencies are axially attached with an accelerometer. An example of results are as follows: 5Hz frequency wave with peak level of 10 mm/sec (0.4 in/sec) is recorded as 10 mm/sec (0.4 in/sec) on the accelerometer, 10.2m/sec (0.402 in/sec) on the geophone SM6 (with 4.5Hz resonant frequency) and as 0.2m/sec (0.008 in/sec) on geophone SM4 (with 10 Hz resonant frequency). A total of 138 blasts were used for this comparative study. Geophone SM4 showed about 40% error in reading the PPV whereas geophone SM6 was in the range of 10%. These errors are minimized by geophone linearization techniques using the transfer function between the geophone as input and accelerometer as output.

Currently used safe values of PPV

YAN Zhixin and JIANG Ping (2002) have related the safe PPV values with the type of structure. The safety radius $R_d = K_d \beta^3\sqrt{Q}$, where K_d is the coefficient related to the foundation medium of buildings, β is coefficient related to the blast features and Q is the quantity of explosive in (kg). Thus, in China more emphasis is given on the safe values with regard to the type of structure in order to attain rationality between the owner of the affected structure and the miner. The paper states the accepted safe PPV values adopted in China and some parts of the world, such as in USA it is 2in/s. France specifies the safe PPV in densely populated area to be 10cm/sec (3.9 in/sec). Germany, Switzerland and some other countries of Europe strictly adopt the earthquake criteria. In China it is as follows:

1) Earth-cave dwellings, earth adobe construction, stone houses: 1cm/sec (0.4in/sec)
2) Ordinary brick houses, blocks of materials: 2~3 cm/sec (0.8~1.2 in/sec)
3) Reinforced concrete frame houses: 5cm/sec (2in/sec)

4) Draining tunnels: 10cm/sec (3.9in/sec), Transport tunnels: 15cm/sec (5.9in/sec)
5) Mining stony head with unstable rocks and good protection of stands: 10cm/sec (3.9in/sec)
6) Medium stable rocks with protection of stands: 20cm/sec (7.8in/sec)
7) Hard rocks no protection of stands: 30cm/s (11.8in/sec)

Thus, as per the surrounding types of buildings, the quantity of the explosive is regulated.

Division of Mineral Mining (DMM) Surface Blaster's Certification Study Guide defines the categories of damages like,

Threshold: New cracks in plaster, opening of old cracks and dislodging of loose objects fall under this category.

Minor: Fallen plaster, broken windows, significant cracks in plaster, hair cracks in masonry. All cracks that do not affect the structural strength are noted as Minor.

Major: Cracks that weaken the structure, shifts of the foundation, permanent displacement of bearing walls, settlements which cause distortion of structure or the walls getting out of plumb.

The crack threshold is found in the range from 0.8 to 11.8 in/sec. The correlation established in this study is that higher frequencies of maximum particle velocity warrant higher thresholds. The construction blasts generate the highest threshold; then comes the quarry blast; whereas surface mine produces the lowest thresholds among all. Safety values stated in this study guide are such as, when PPV exceeds 2 in/sec, the threshold of cosmetic damages start. Minor damages are observed at 5.4in/sec and major at about 7.6 in/sec. About 100 exceptional observations are noted where PPV in the range of 2~6 in/sec did not caused any damage. This paper also expresses a consensus with the value of 2-in/sec as the safest PPV, which will not cause any appreciable damage to the structure.

Frequency influence and response spectra

According to the Table-2-1 shown below, DMM Surface Blaster's Certification Study Guide illustrates that the natural frequency of the residential structure is somewhere near 7hz. Hence in spite of the high frequency wave, the structure critically behaves with that wave which matches its natural frequency. This phenomenon of increased response also varies inversely with the damping of the structure. This paper restates that when the PPV is maintained below 2 in/sec, the response effect of critical frequency is still under the tolerable limits.

Table-2-1 Type of structure and its empirical natural frequency

Structure or Element	Natural Frequency
Multi Storey Building	F = 0.1N (N= number of stories)
Radio Tower 100ft tall	3.8
Petroleum Distillation Tower 65ft tall	1.2
Coal Silo, 200 ft tall	0.6
Building walls	12 – 20
Wood Frame Residences (1 to 2 Story)	7.0 Standard Deviation = 2.2

(DMM Surface Blaster's Certification Study Guide Table 5.2 – Page 62)

Response spectra analysis

'*Dynamics of structures*' by Chopra explains the detailed concept of Response Spectra Analysis. Peak response quantity when plotted as a function of natural time period for a given damping ratio is called response spectrum. For a given structural displacement, D, corresponding to the ground displacement, the peak relative pseudo velocity V can be determined as,

$$V = \omega_n D = 2\pi D/T_n$$

The strain energy related to this pseudo velocity as,

$$E_{so} = mV^2/2$$

The prefix pseudo is used for V to clarify that it is not equal to the peak ground velocity PPV, even though it has the same units. This book illustrates, with comparative plots shown below as Figure-2-1 (6.12.1 in Chopra) and Figure-2-2 (6.12.2 in Chopra), the relation between the relative ground velocities and the pseudo velocity and relative ground acceleration and pseudo spectral acceleration respectively as a function of time period, for the El Centro earthquake. It is evident from this plot that for a lower time period, i.e. $T_n \rightarrow 0$, the relative pseudo velocity converges with the ground velocity, $V \rightarrow \dot{u}_0$ and when $T_n \rightarrow \infty$ then $V \rightarrow 0$. The plot does not show any fixed proportionality between the relative ground velocity and the pseudo velocity other than the divergence stated above. On the other hand relative ground acceleration holds a consistent relationship with the pseudo spectral acceleration irrespective of the natural period. However for higher damping ratios, slight divergence in the response is observed.

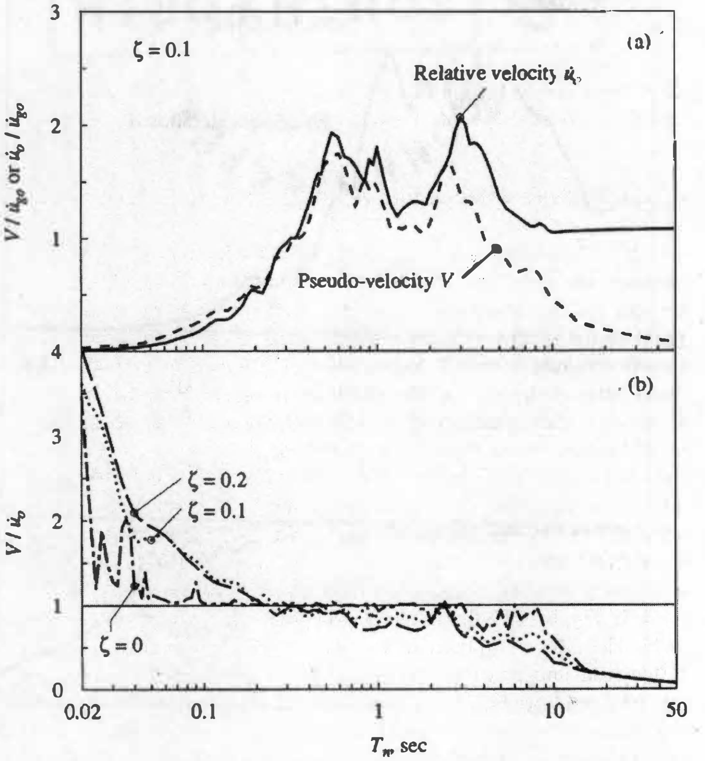


Figure 6.12.1 (a) Comparison between pseudo-velocity and relative-velocity response spectra; ζ = 10%; (b) ratio $V/\dot{u}_o$ for ζ = 0, 10, and 20%.

Figure– 2-1 Comparison Between Ground Velocity And Struc. Velocity (Chopra, *DYNAMICS OF STRUCTURES*, 2/E, © 2001, Pp.241 Reprinted By Permission Of Pearson Education, Inc., Upper Saddle River, New Jersey.)

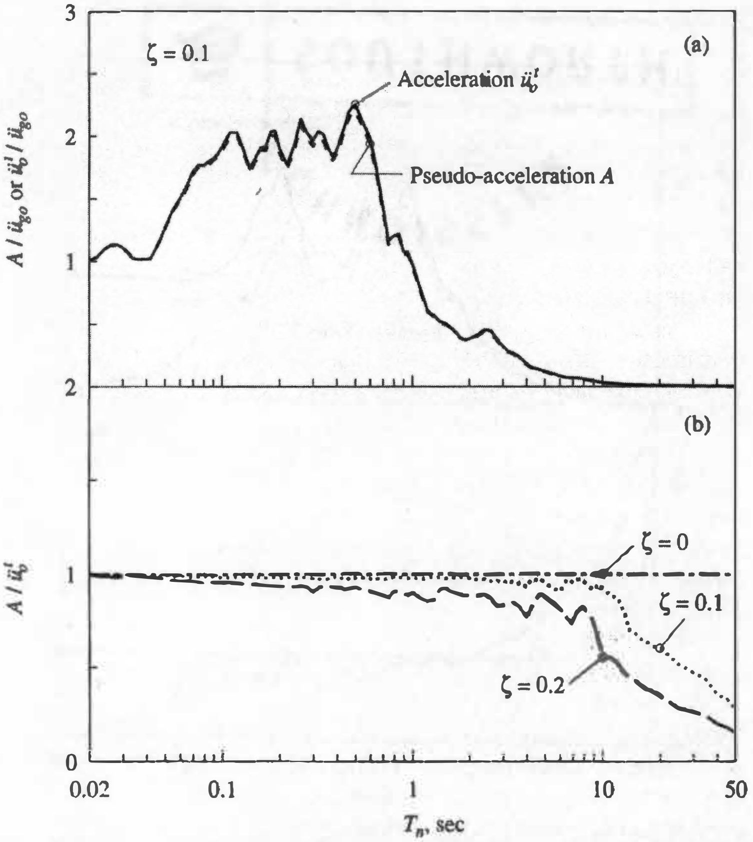


Figure 6.12.2 (a) Comparison between pseudo-acceleration and acceleration response spectra; $\zeta = 10\%$; (b) ratio $A/\ddot{u}_o^t$ for $\zeta = 0$, 10, and 20%.

Figure–2-2 Comparison Between Ground Acceleration And Struc. Acceleration (Chopra, *DYNAMICS OF STRUCTURES*, 2/E, © 2001, Pp.243 Reprinted By Permission Of Pearson Education, Inc., Upper Saddle River, New Jersey.)

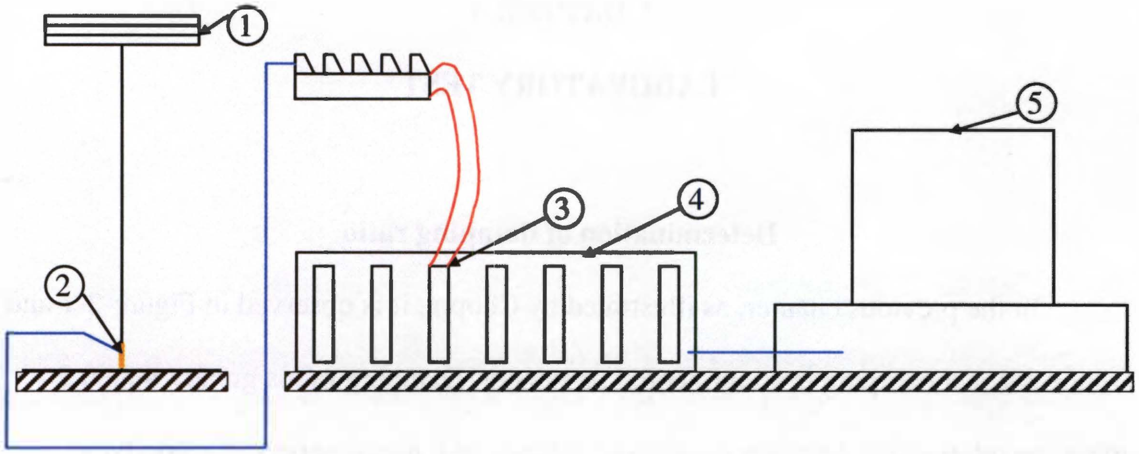
CHAPTER 3

LABORATORY TEST

Determination of damping ratio

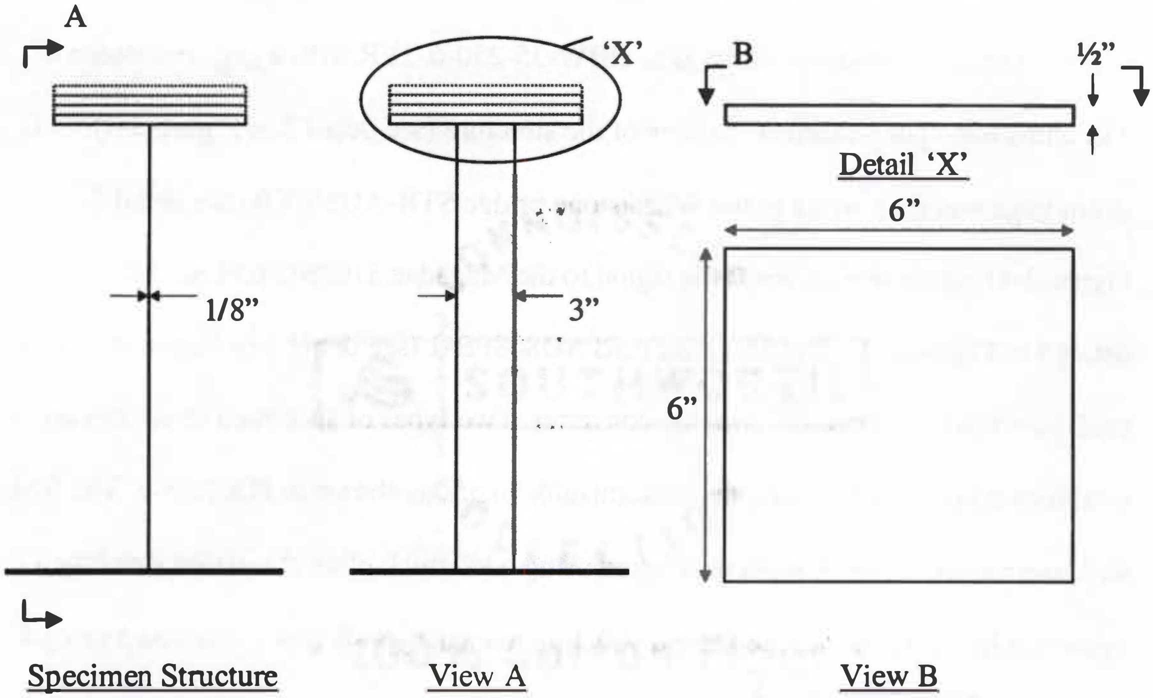
In the previous chapter, as illustrated by Chopra, it is observed in Figure-2-1 and Figure-2-2 that damping is an influential parameter. Damping ratios greater than 0.2 affect the relationship between the ground velocity and pseudo-spectral velocity, or ground acceleration and pseudo-spectral acceleration, significantly; whereas, lower damping ratios do not affect the relationship considerably. A prototype model was therefore constructed to test the actual damping ratio of the structure used in the computer simulation.

The structure specimen as shown in Figure-3-1 was clamped to a fixed base using four C-clamps. A weldable strain gage HBW-35-250-6-3VR with a gage resistance of 350 ohms was spot welded at the base of the structure (see detail 2 in Figure-3-1). This strain gage was then wired to the Wheatstone bridge STB-AD808FB (see detail 3 in Figure-3-1) which in turn sends the signal to the Megadac 3108AC part no. ML1064 (see detail 5 in Figure-3-1). The Megadac card AD808FB-1 (see detail 4 in Figure-3-1) was configured to be compatible with the converter. Two types of specimen structures are established in this study using the customizable plates as shown in Figure-3-2. The first structure named Type–A was constructed using a ½" thick plate at top thus exerting a mass of 5 lbs at top, while the second structure named Type-B was constructed using 4 such ½" thick plates summing up to 20 lbs of mass at top. Each case was studied independently for determining the damping ratio.



1 . Specimen Structure with customizable plates at top.
2. Strain Gage HBW–35–250–6–3VR (Gage resistance : 350 ohms + or – 1%)
3. Megadac Card AD 808FB - 1
4. Megadac Model 3108AC – Part No. ML1064
5. Computer (Dos Based)

Figure–3-1 Assembly For Determining Coefficient For Damping



Figure–3-2 Details Of Specimen Structure

After the assembly of all the equipment, the structure was then pulled to one side and was released to vibrate freely. The steady state vibrations were then recorded in the computer in the form of strain vs. time plot. The data were collected at the rate of 400hz. As the strains were directly proportional to the displacements, the damping coefficient was determined from these plots using the log decrement method as follows.

$$\zeta = \frac{1}{2\pi j} \ln\left[\frac{u_i}{u_{i+1}}\right]$$

The strains vs. time plots are shown in Figures-3-3 and 3-4.

Damping Ratio for ½" Plate:

$$\zeta = \frac{1}{2\pi(21.51 - 20.19)} \ln\left[\frac{213.64}{199.68}\right]$$

$$\zeta = 0.0082$$

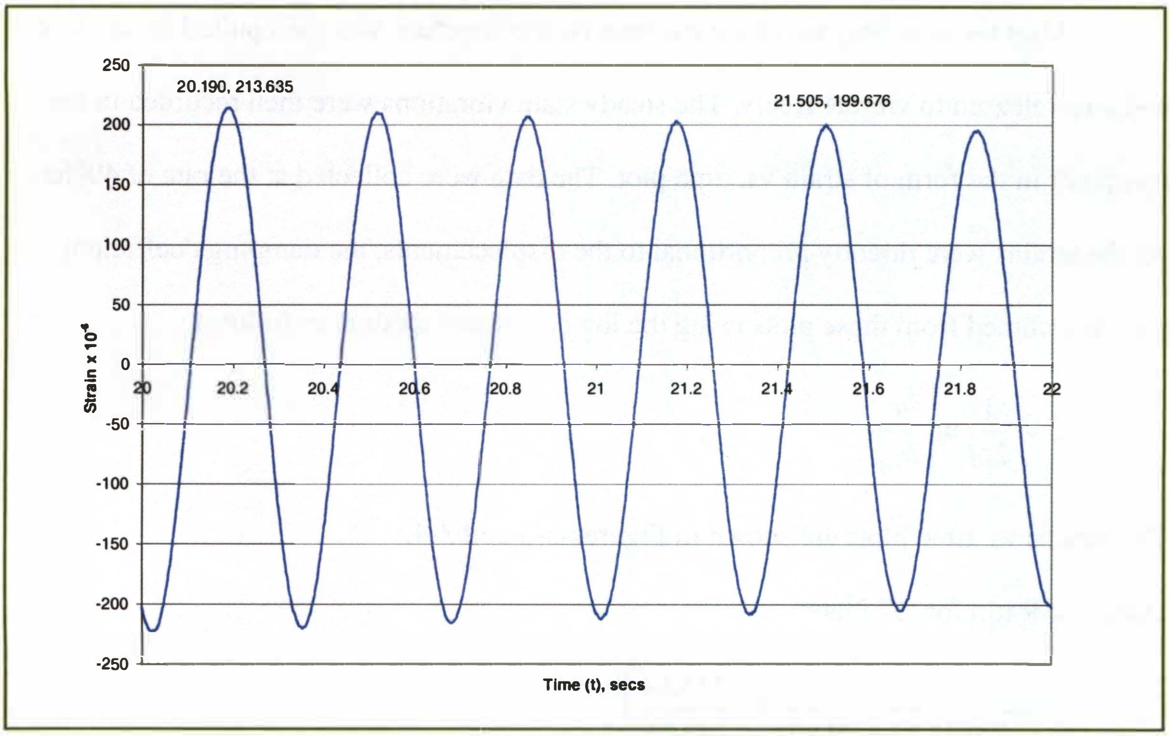
Damping Ratio for 2" Plate: Refer Figure 3-4

$$\zeta = \frac{1}{2\pi(20.14 - 15.24)} \ln\left[\frac{256.73}{213.03}\right]$$

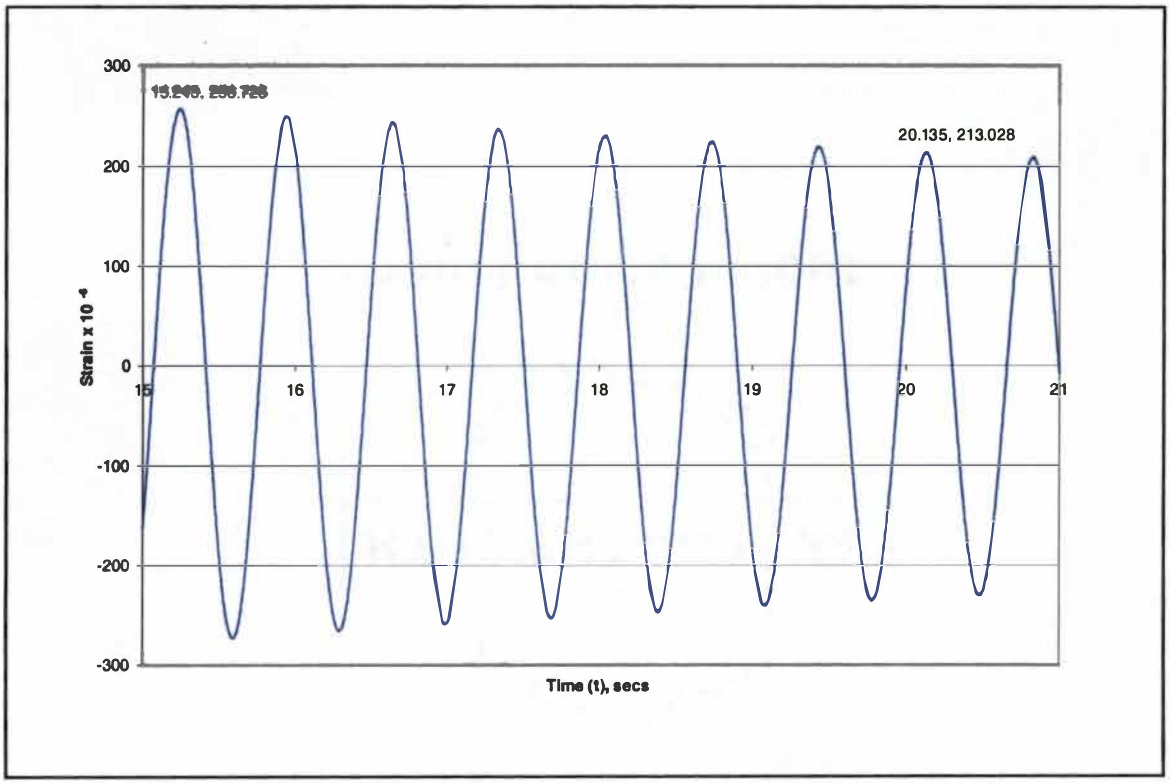
$$\zeta = 0.0061$$

Conclusion

The damping ratio of both the structures was reasonably low and would not affect the relationship between ground and structural motion to a significant extent. Hence the effect of damping ratio in the finite element analysis was neglected.



Figure–3-3 Strain Vs. Time (For 1/2" Plate At Top)



Figure–3-4 Strain Vs. Time (For 2" Plate At Top)

CHAPTER 4

FINITE ELEMENT MODEL

Introduction

The objective of this finite element simulation was to study the response acceleration of the structure and determine the correlation with the ground motion parameters. In order to study which parameter amongst the ground velocity or ground acceleration correlates best with the response acceleration, various permutation and combination case models were generated. The flexibility to generate a variety of models could be achieved easily and cheaply through computer models rather than laboratory experimentation. Thus the finite element models for the structures, type A and type B were generated in ADINA 900 Nodes Version 8.0.

Model specifics

To avoid the hysteric response due to non-linearity of the material, the structure was modeled as 2-D linear elastic beam model. The material property used for steel, as shown in Figure-4-1, was E = 29,000,000 psi and density of steel = 0.283 lb/in^3. The dynamic properties used were as shown in Figure-4-2 and Figure-4-3. To achieve smoothed mode shapes, the stem of 18 inch high structure was divided in 100 elements as shown in Figure-4-1. The top plate however was modeled as a single beam element. Damping ratio, being very low, was neglected in the test. The base of the structure was required to be excited for the test, hence the base was assigned as a roller support to translate horizontally while the top of the structure was set free to deflect.

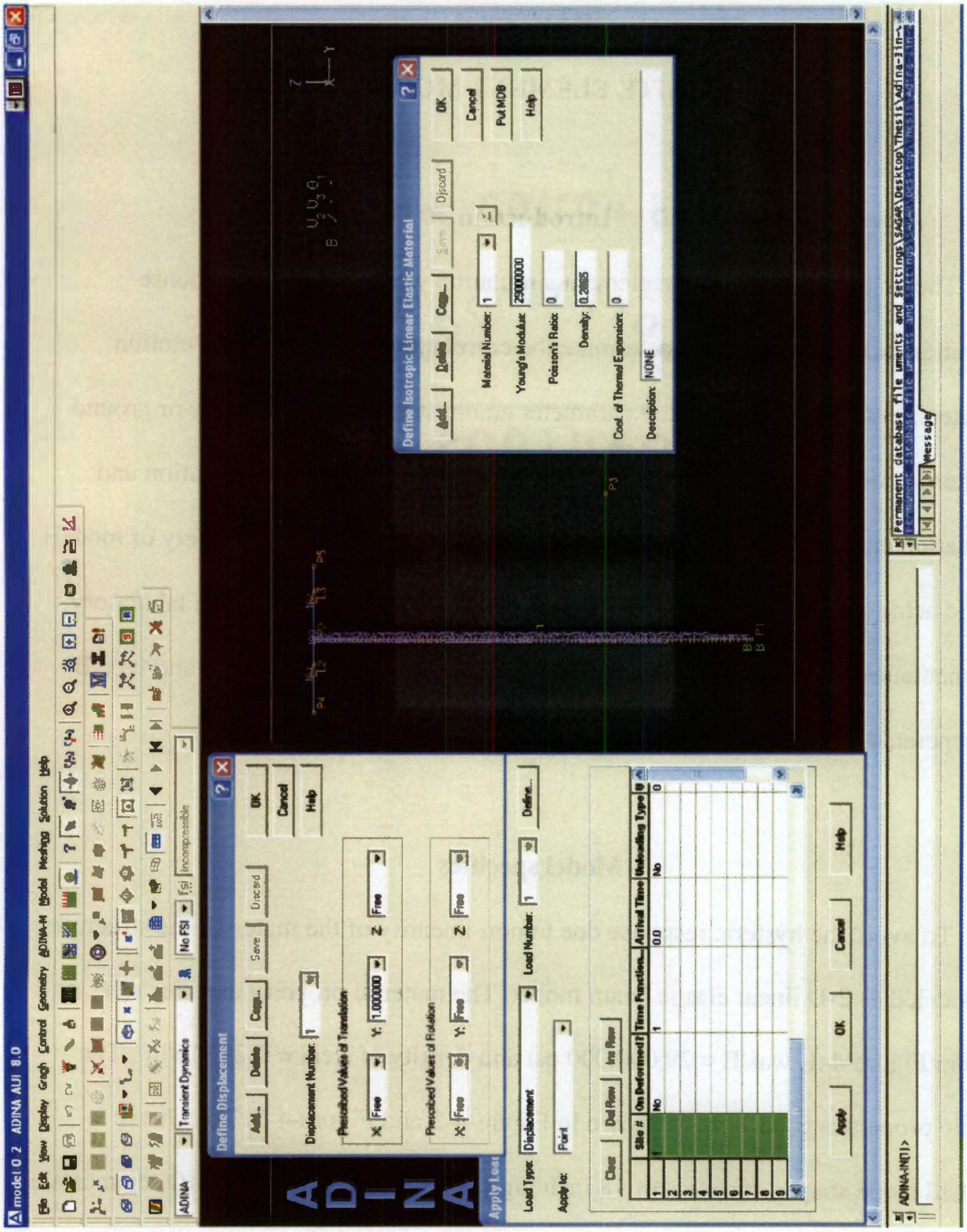


Figure-4-1 Typical Adina Model

1/2" in Thick Plate at Top		
Time Period Of The Structure		
Height Of Structure (L) :	18	in.
Weight At Top (W) :	5	lbs
Width Of Tower (b) :	3	in.
Thickness Of Tower (t) :	0.125	in.
Youngs Modulus (E):	29000000	psi
Moment of Inertia (I) :		
I =	$b*t^3/12$	
I =	0.000488281	in^4
Stiff. Of Structure (k)		
k =	$3 EI / L^3$	
k =	7.28	lb/in
Natural Frequency		
ω =	$\sqrt{k/m}$	
ω =	23.73	rad/sec
Cyclic Frequency		
f=	ω / 2p	
f=	3.78	hz
Time Period :		
T =	1/ f	
T =	0.26	sec

Figure-4-2 Calculation Of Dynamic Properties For ½" Thick Plate Structure

2" in Thick Plate at Top		
Time Period Of The Structure		
Height Of Structure (L) :	18	in.
Weight At Top (W) :	20	lbs
Width Of Tower (b) :	3	in.
Thickness Of Tower (t) :	0.125	in.
Youngs Modulus (E):	29000000	psi
Moment of Inertia (I) :		
I =	$b*t^3/12$	
I =	0.000488281	in^4
Stiff. Of Structure (k)		
k =	$3 EI / L^3$	
k =	7.28	lb/in
Natural Frequency		
ω =	$\sqrt{k/m}$	
ω =	11.86	rad/sec
Cyclic Frequency		
f=	ω / 2p	
f=	1.89	hz
Time Period :		
T =	1/ f	
T =	0.53	sec

Figure-4-3 Calculation Of Dynamic Properties For 2" Thick Plate Structure

The structure was statically unstable for any horizontal load, which entailed the excitation to be in terms of controlled displacement. A half sine curve function of time as shown in the equation below was adopted to generate the excitation displacement.

$$u = \pi(u_{max})\sin\left\{\frac{2t}{T}\right\}$$

Where,

u = Displacement at time, t

u_{max} = Maximum displacement

t = time interval

T = Excitation period

The analysis type was set to Transient Dynamics in Adina (Plimpton, Attaway, Hendrickson, Swegle,Vaughan, Gardner). Transient dynamics is generally used as the best solution method for any impact or collision simulation tests where the results are desired for a very minute time interval. Modeling of material deformation is possible using the transient dynamic codes for discrete time intervals. Lagrangian meshes, unlike the Eulerian meshes, are used for the formulation of transient dynamics models. These meshes are able to befit any complex shapes and deform in a more realistic manner during the simulation. The boundary deformations of an element can be then calculated using the stresses and strains of adjacent elements which is called the transient dynamics solution. The test with the time function (sine curve function) as stated above was made to run for 10 cycles while each cycle was divided into 100 time steps further. Therefore, the results were obtained at extremely small time intervals which required a transient dynamics analysis.

Model cases generated

Each type of structure was tested for three different values of peak ground acceleration as 20in/sec^2, 10in/sec^2, and 6in/sec^2 keeping the peak ground velocity constant. Then keeping the same set of ground acceleration values, the test was run for three different values of peak ground velocity 1in/sec, 2in/sec and 3in/sec. Each test was assigned with a separate model number. The permutation combination and their respective model numbers are illustrated in Figure-4-4 and Figure-4-5. The velocity is directly proportional to acceleration under a constant excitation frequency. Hence, to excite the structure at the same acceleration at higher velocity was not possible without reducing the excitation frequency. Therefore, each model was excited at a different frequency ratio within one set.

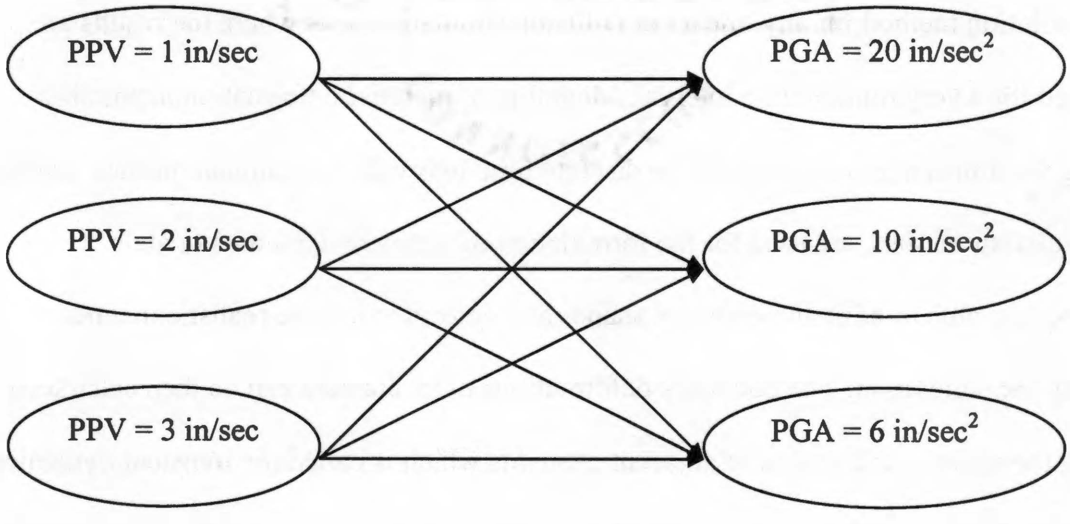


Figure-4-4 Permutation Combination Models Generated In FEA

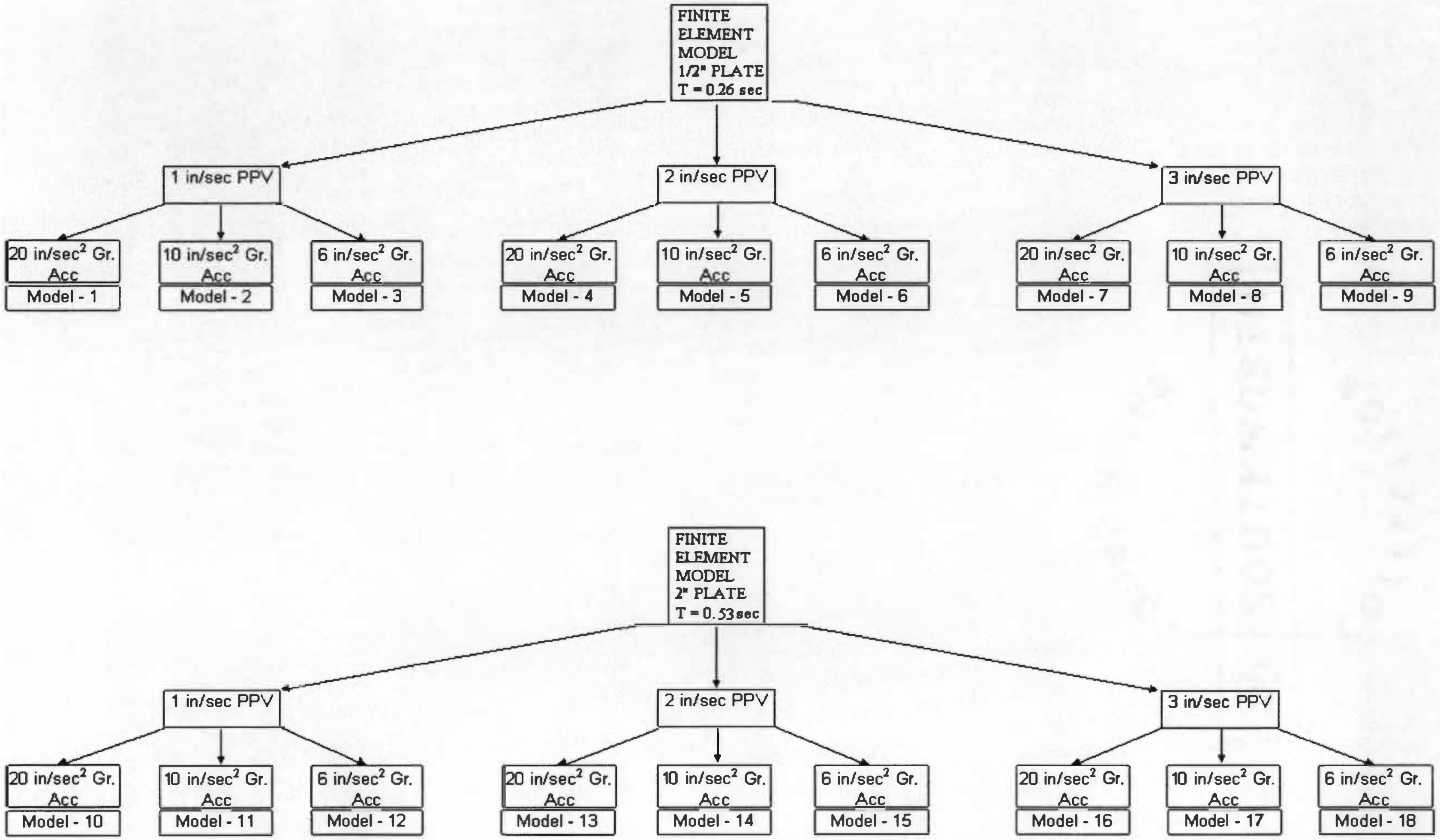


Figure-4-5 Schematic Plan For Type Of Models Generated In Adina

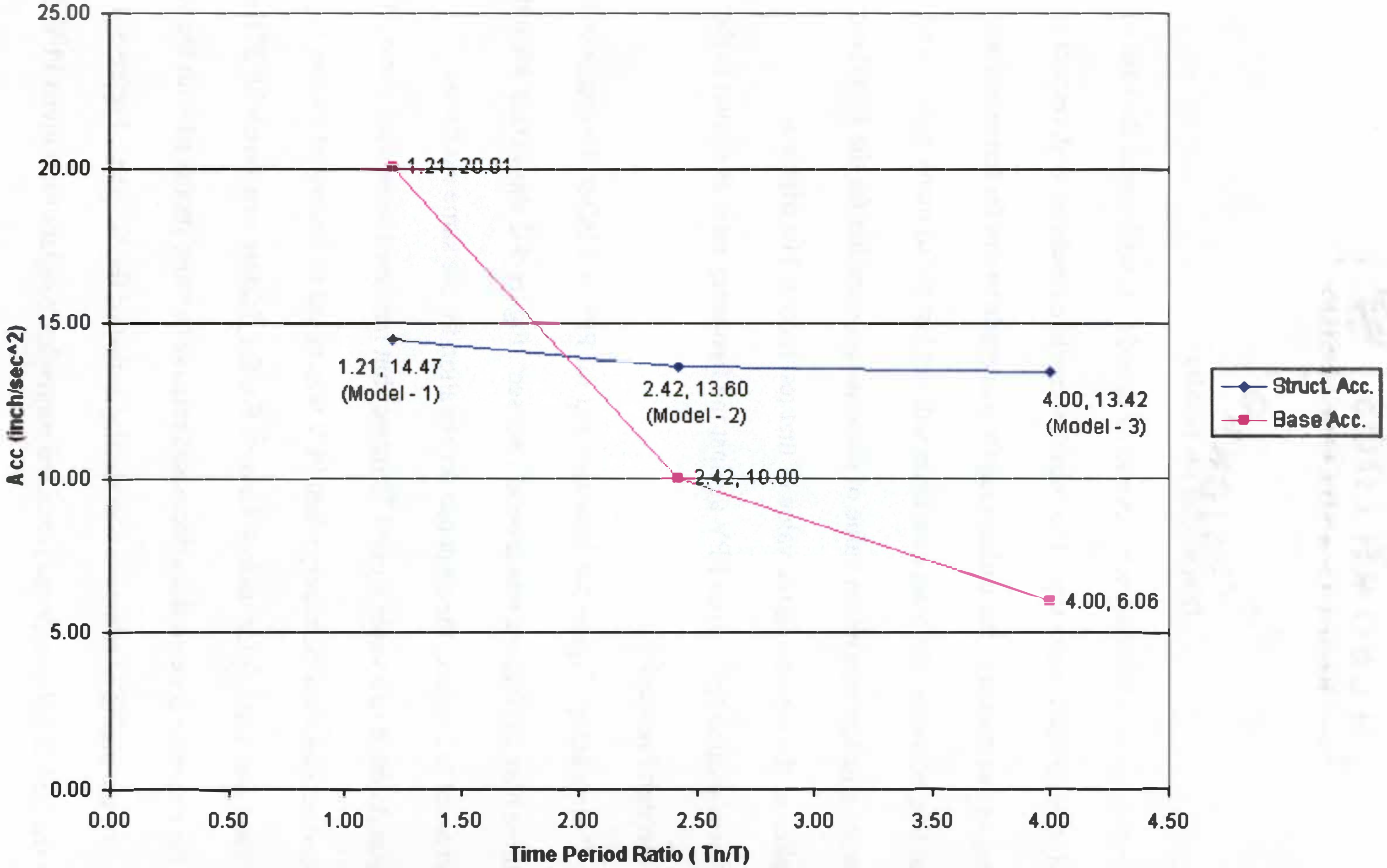
CHAPTER 5

RESULTS AND CONCLUSION

Discussion on results

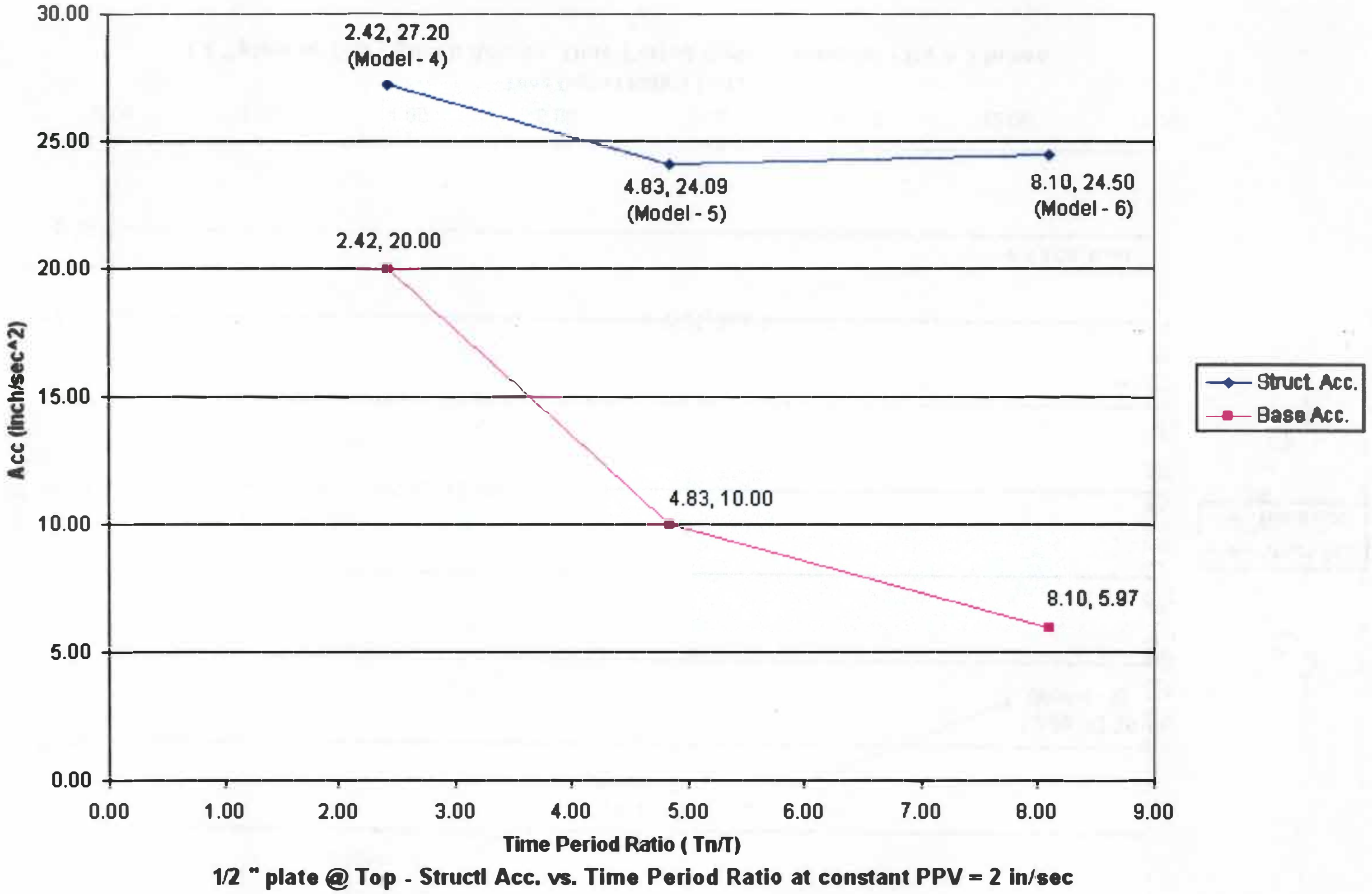
The horizontal accelerations were extracted at node #1, which was the base of structure, and at node #101 at the top. The results were then tabulated with respect to time. Each test as mentioned in the earlier chapter was made to run for ten excitation cycles. In real life, however, the blast excitation will not last for so many numbers of cycles; hence the absolute maximum value of response acceleration for the first three cycles was taken as the representative value of that test model. The triplet of accelerations was plotted for a given PPV against the frequency ratio as shown in the Figures-5-1 through Figure-5-6.

For ½" plate at top, Figure-5-1 indicates that for PPV = 1 in/sec the response for each of the three base excitations was around 13in/sec^2. Figure-5-2 shows that when the PPV was increased to 2 in/sec, the response acceleration for the same excitation magnitude was doubled to a value around 25 in/sec^2 but remained somewhat constant for varying ground acceleration. Similarly when PPV was raised to 3in/sec, shown in Figure-5-3, the response line was tripled up to 40in/sec^2. For the flexible structure with 2" thick plate at top the response accelerations decreased relative to those produced with the ½" plate. However, the nature of response was similar to that of the ½" plate. Figure-5-4 through 5-6 showed that the response remained somewhat constant for a given PPV. For PPV =1in/sec the response accelerations for all of the base accelerations were about 3 in/sec^2. This structure also showed proportionality with PPV.

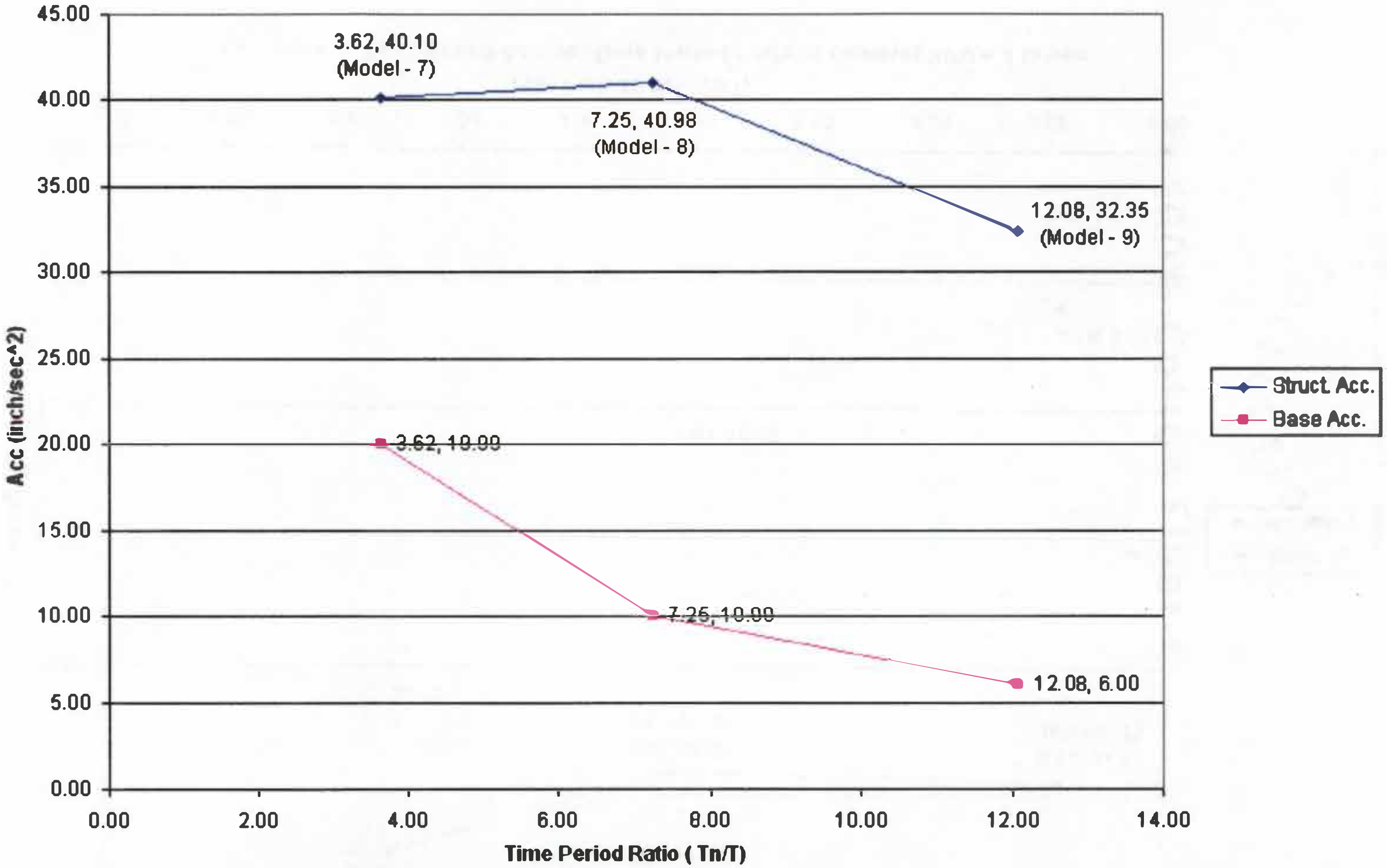


1/2 " plate @ Top - Structl Acc. vs. Time Period Ratio at constant PPV = 1 in/sec

Figure-5-1 Model-1, Model-2, Model-3

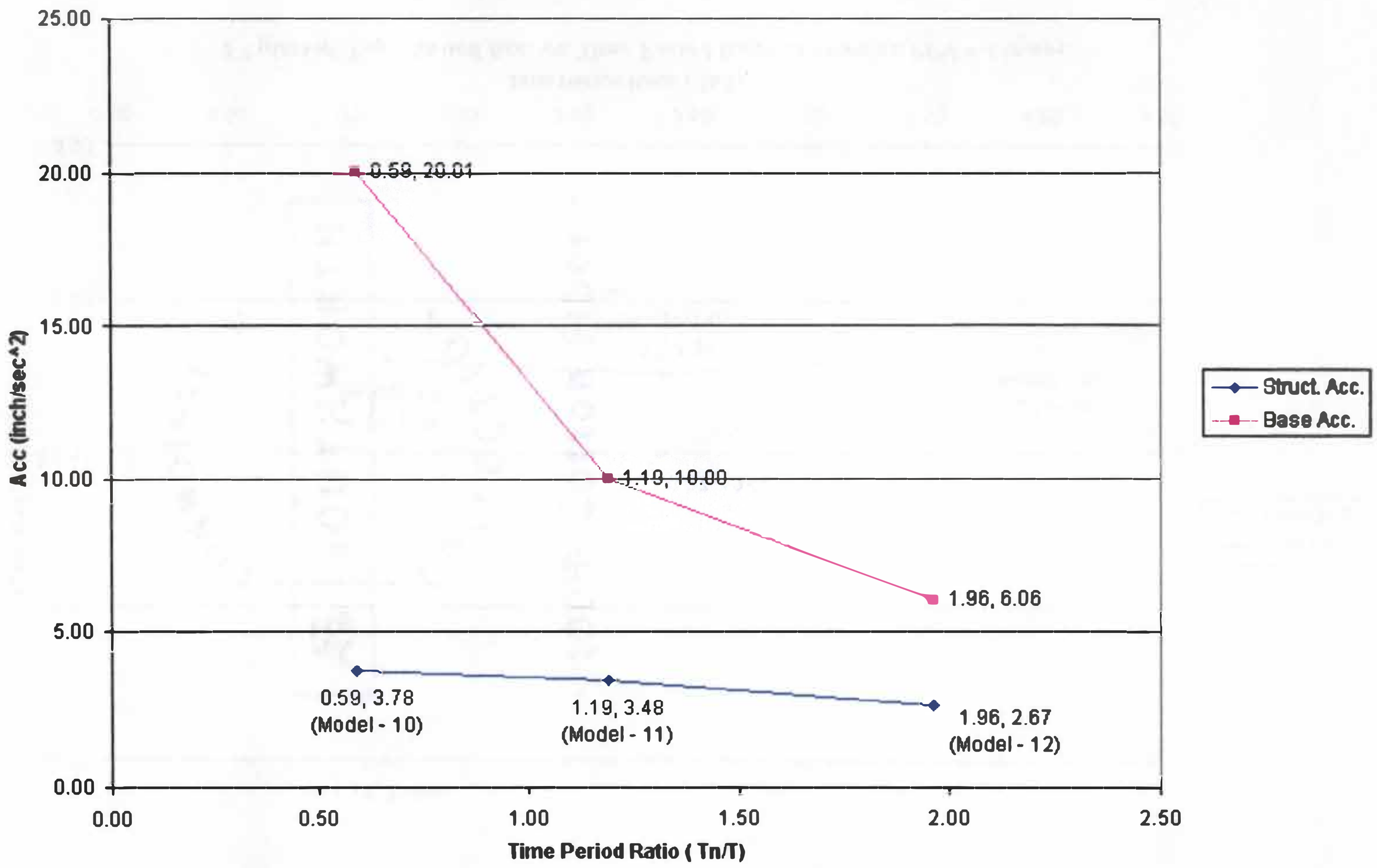


Figure– 5-2 Model -4, Model-5, Model-6



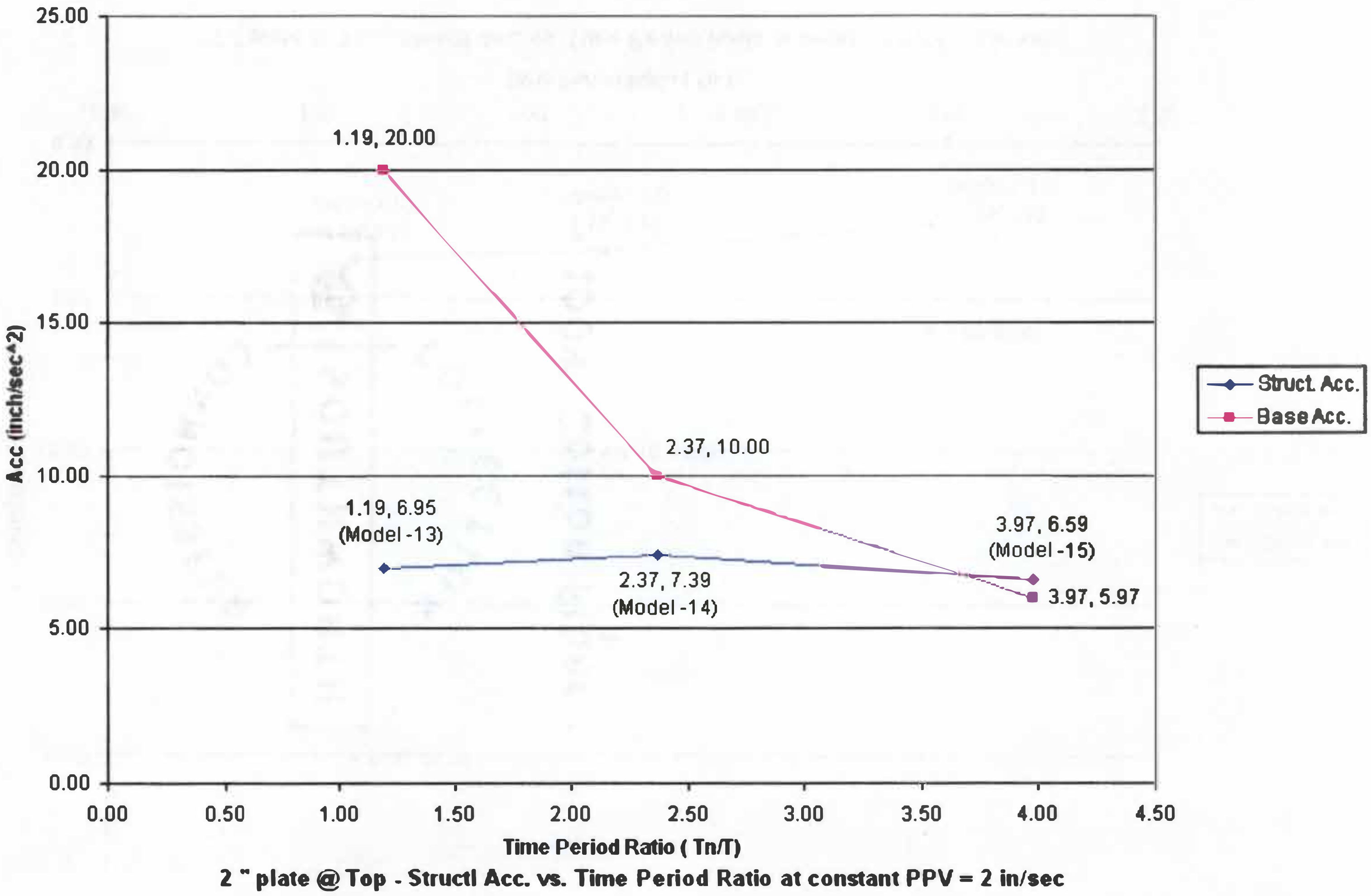
1/2 " plate @ Top - Structl Acc. vs. Time Period Ratio at constant PPV = 3 in/sec

Figure–5-3 Model-7, Model-8, Model-9

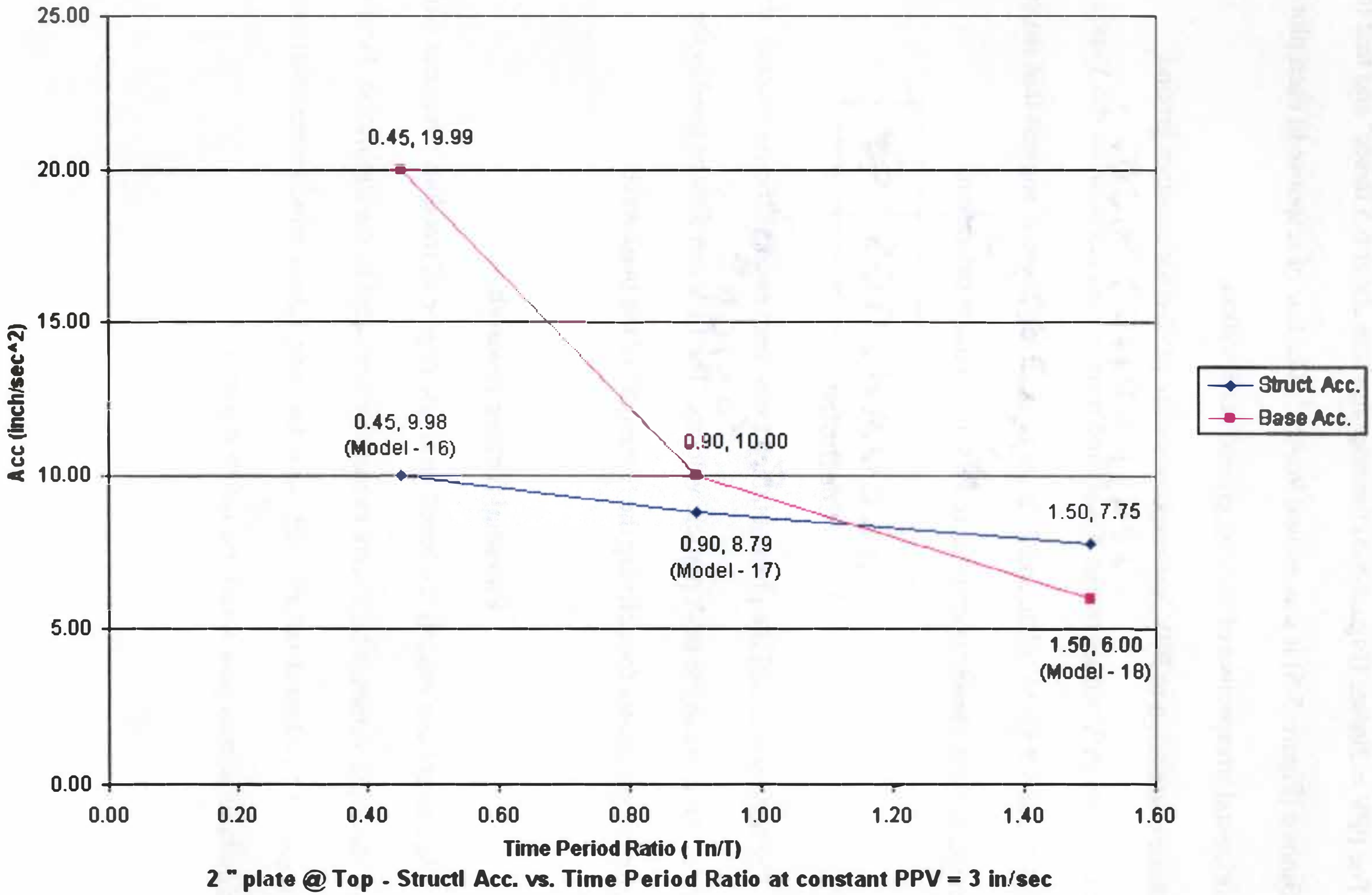


2 " plate @ Top - Structl Acc. vs. Time Period Ratio at constant PPV = 1 in/sec

Figure–5-4 Model 10, Model-11, Model-12



Figure–5-5 Model-13, Model-14, Model-15



Figure–5-6 Model-16, Model-17, Model-18

For PPV = 2in/sec (Figure-5-5) the response was about 6.5 in/sec^2 and that for PPV = 3in/sec (Figure-5-6) it was around 9in/sec^2. The line of response in each plot was fairly horizontal irrespective of varying ground acceleration.

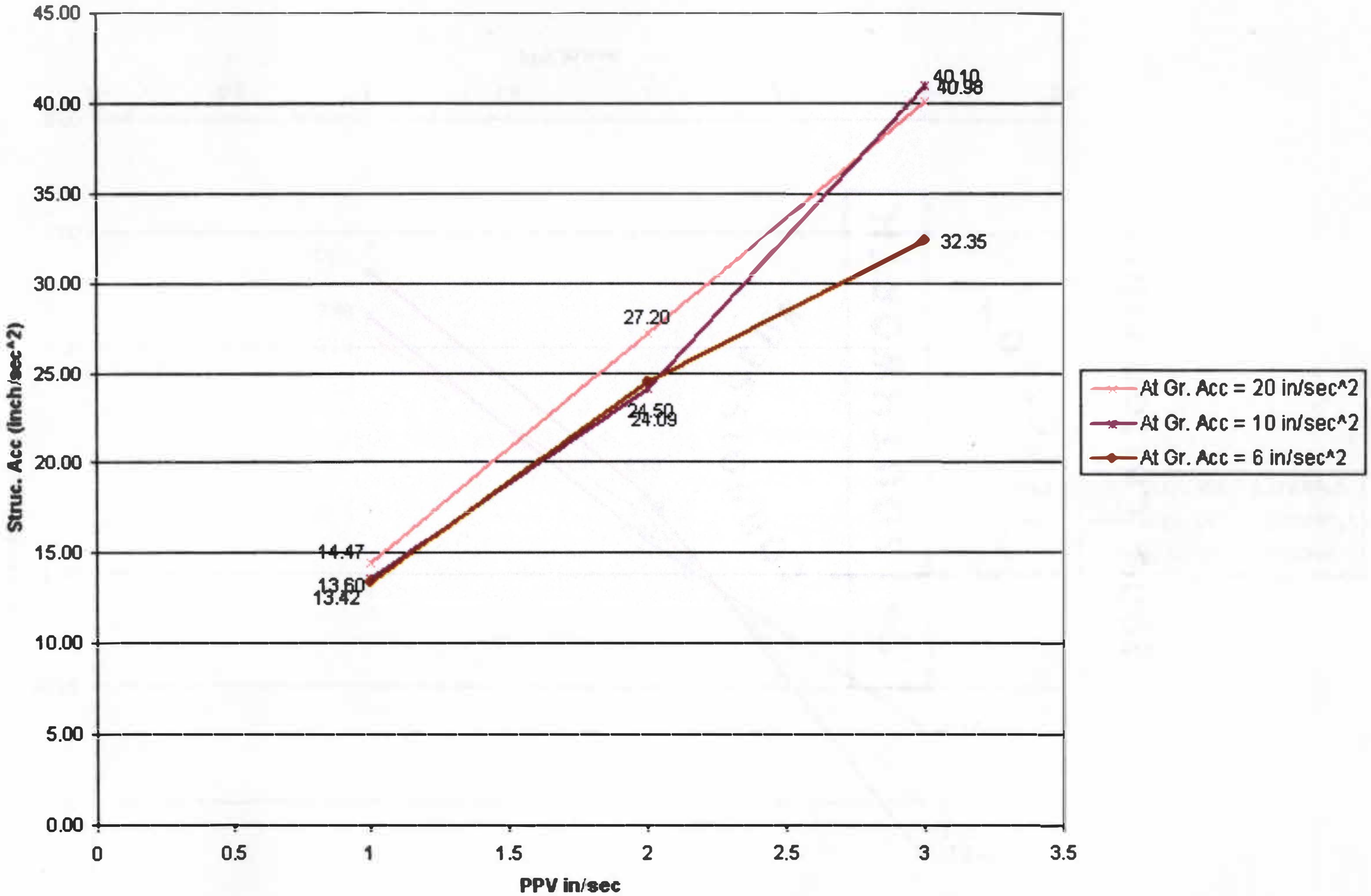
To illustrate correlation to PPV, the responses were plotted for constant ground acceleration with PPV values on the X-axis and response acceleration on the Y-axis. Figures-5-7 and 5-8 for ½" plate and 2" plate respectively thereby suggest that structural acceleration is more closely correlated to PPV than base acceleration.

Conclusion

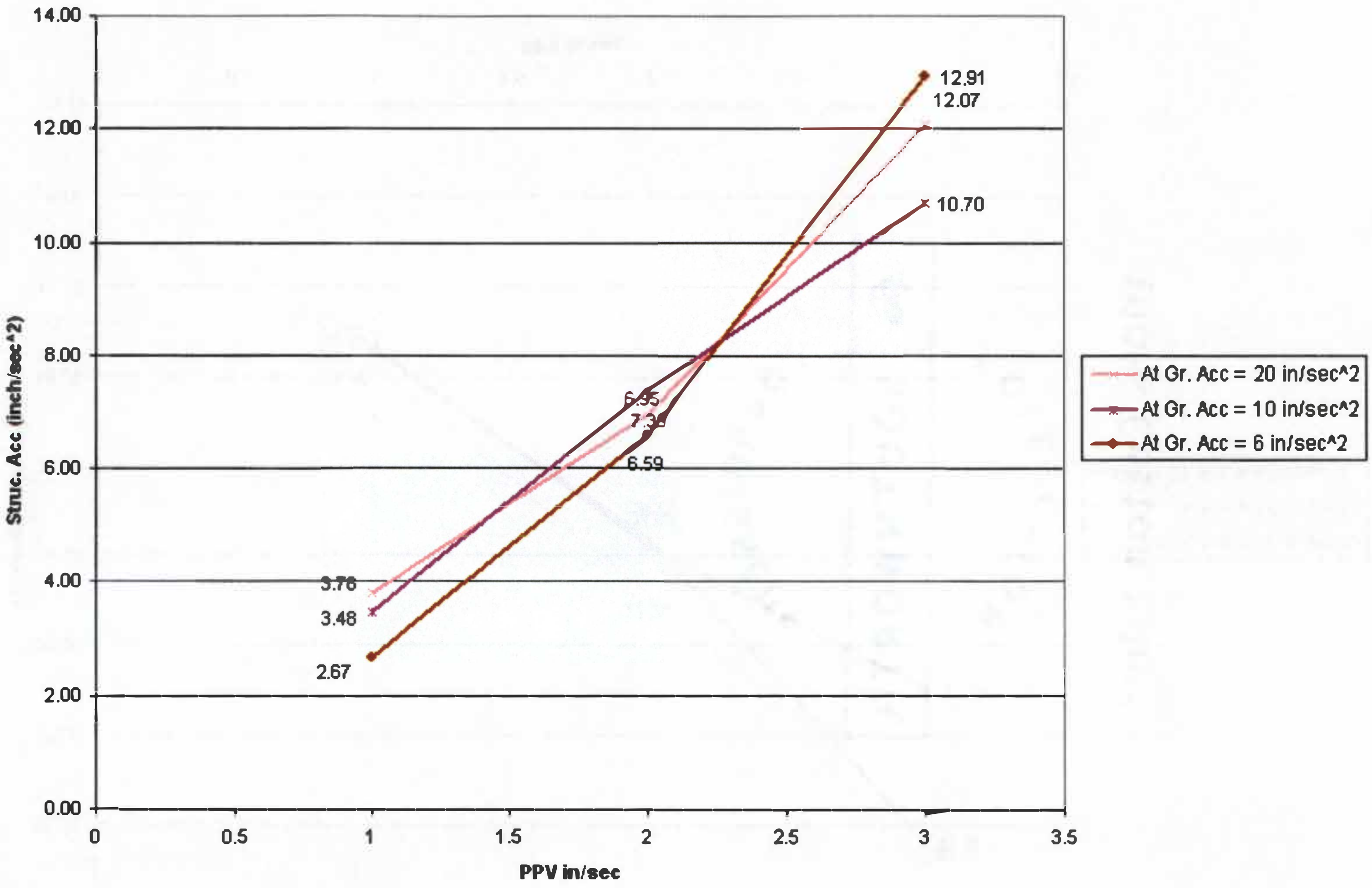
The test models and the plots obtained from them endorse the correlation of the response of structure to the peak particle velocity. The PPV can thus be used as the representative parameter for defining the "strength" of the blast wave.

Potential further research

The current test models are based on single degree of freedom structures with very low damping values, which were eventually neglected in the simulation. Hence the multi-degree of freedom structures with high damping values, which were unattended in these simulations, may pose a task for future research.



Figure–5-7 1/2 " Plate At Top - Ground Acceleration And Struct. Response For Various PPV



Figure–5-8 2 " Plate At Top - Ground Acceleration And Struct. Response For Various PPV

WORK CONSULTED

Anon, “Seismic design and detailing of steel structures,” *Journal of the Institution of Engineers-(India): Civil Engineering Division*, Vol. 83, n AUG, August 2002, p 105-110

Barsom John M, “Fatigue and fracture behavior of moment frame connections under seismic loading (Northridge earthquake),” *ASTM Special Technical Publication*, n 1417, 2002, p 57-72

Stinemates Daniel W, Bennett Joel G, “Finite Element modeling for validation of structural damage identification experimentation,” *Proceedings of SPIE The International Society for Optical Engineering*, Vol 4753, I 2002, p 611-616

Pothisiri T, Hjelmstad K D, “Structural damage detection and assessment from modal response,” *Journal of Engineering Mechanics*, Vol 129, n 2, February 2003, p 135-145

Johnson Timothy, Adams Douglas E, Schiefer Mark, “An analytical and experimental study to access structural damage and integrity using dynamic transmissibility,” *Proceedings of SPIE The International Society for Optical Engineering*, Vol 4753, I 2002, p 472-476

Yan Zhixin, Jiang Ping, Yan Zhixin, Wu Delun, “A study of blast vibration effect and the construction safety,” *Process in safety science and Technology Part A* , Vol 3, 2002, p 569-572

Tripathy G R, Gupta I D, "Prediction of ground vibrations due to construction blasts in different types of rock," *Rock Mechanics and Rock Engineering*, Vol 35, n 3, August 2002, p 195-204

Farnfield R A, "So you think you are monitoring peak particle velocity," *Proceedings of the Annual Symposium on Explosives and Blasting Research 1996, International Society of Explosives Engineers, Cleveland, OH, USA*, p 13-20

Siskind David E, "Frequency analysis and the use of response spectra for blast vibration assessment in mining," *Proceedings of the Annual Symposium on Explosives and Blasting Research 1996, International Society of Explosives Engineers, Cleveland, OH, USA*, p 1-11

Armer GST, Moore D B, "Full scale testing on complete multistory structures," *Structural Engineer*, Vol 72, n 2, Jan 1994, p 30-31

Pal Roy Pijush, "Putting ground vibration predictions into practice," *Colliery Guardian*, Vol 241, n 2, Mar 1993, p 63-67

Anon, "Suggested method for blast vibration monitoring," *Int J Rock Mech Min Sci Geomech Abstr.*, Vol 29, n 2, Mar 1992, p 143-156

Ghosh A, Daemen J J K, "Validation of exponential decay blast vibration predictor with case studies," *Society of Minig Engineers of AIME, Littleton*, CO, USA, 91-87 p 9

Tripathy G R, Gupta I D, "Prediction of ground vibrations due to construction blasts in different types of rock," *Rock Mechanics and Rock Engineering*, Vol 35, n 3, August 2002, p 195-204

Adhikari G R, Singh R B, Gupta R N, "Structural response to ground vibration from blasting in opencast coal mines," *J Mines Met fuels*, Vol 37, n 4, April 1989, p 135-138

Gupta V K, Nielsen Soren R K and Kirkegaard Poul H, "A preliminary prediction of seismic damage based degradation in RC structures," *EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS*, Vol 30, October 2000, p 981-993

Kunnath S K, Reinhorn A M, "Analytical modeling of inelastic seismic response of RC structures," *ASCE Journal of Structural Engineering* , Vol 116(4), 1990, p 996-1017

Park Y J, Ang AH-S "Mechanistic seismic damage model for reinforced concrete," *ASCE Journal of Structural Engineering*, Vol 111(4), 1985, p 772-739

Nicholls H R, Johnson C F, Duvall W I, "Blasting vibrations and their effects on structures," *U S Bureau of Mines Bulletin*, (656), 1971

Siskind D E, Stagg M S, Kopp J W, Dowding C H, "Structure response and damage produced by ground vibration from surface mine blasting" *U S Department of the Interior- Bureau of Mines,* Report of Investigation 8507

Chopra A K, "Theory and application of earthquake engineering – DYNAMICS OF STRUCTURE", Prentice Hall, Upper Saddle River, NJ, 2001

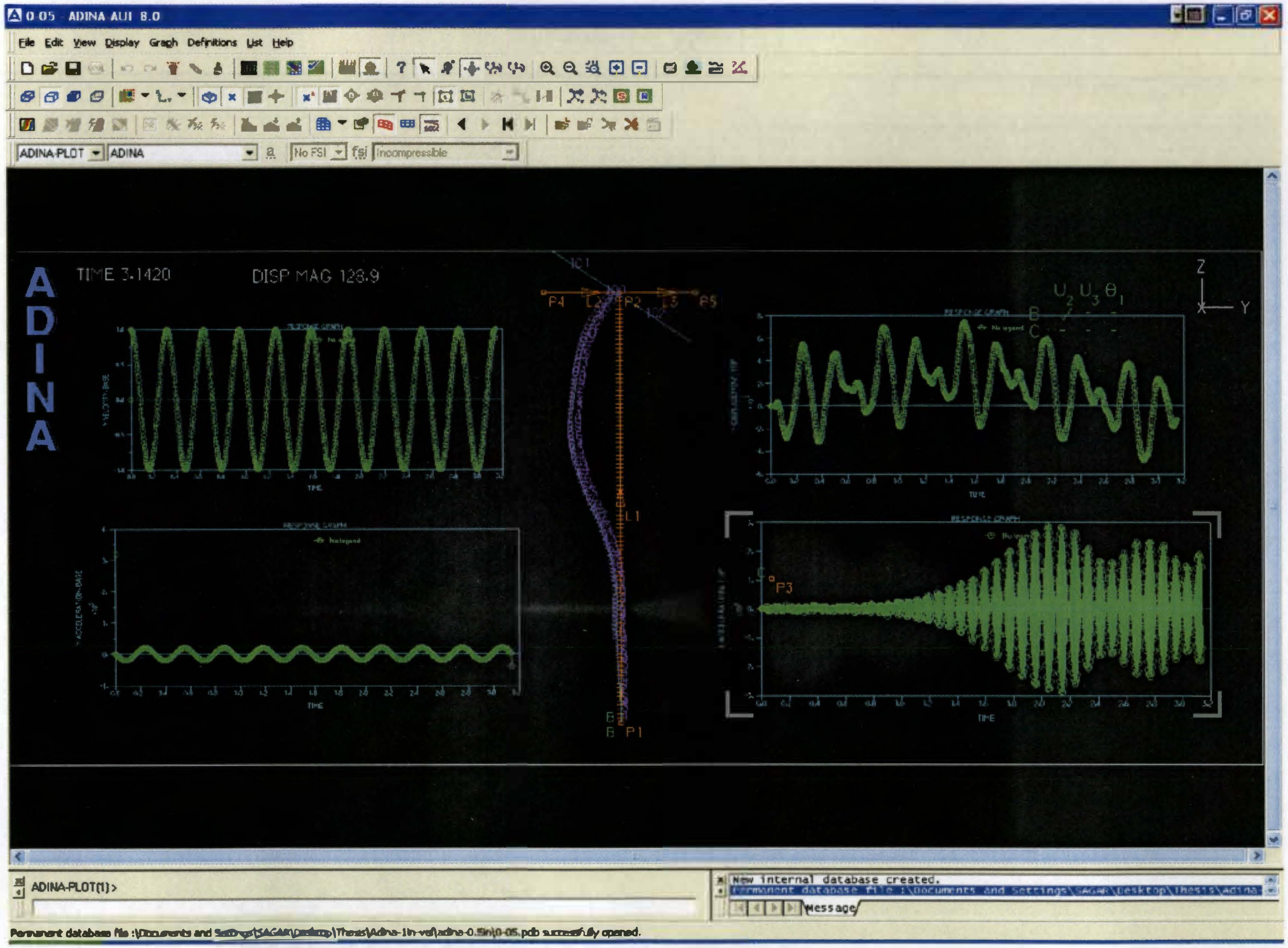
Siskind D. E, "VIBRATIONS FROM BLASTING", Intl Society of Explosives, December 2001

Steve Plimpton, Bruce Hendrickson, Steve Attaway, Jeff Swegle, Courtenay Vaughan, Dave Gardner, "Transient dynamics simulation: Parallel algorithms for contact detection and smoothed particle hydrodynamics," *Super Computing '96 Conference proceedings*, 1996

www.supercomp.org/sc96/proceedings/SC96PROC/PLIMPTON/INDEX.HTM

APPENDIX A

FINITE ELEMENT SIMULATION MODELS



Figure–A-1 Model-1

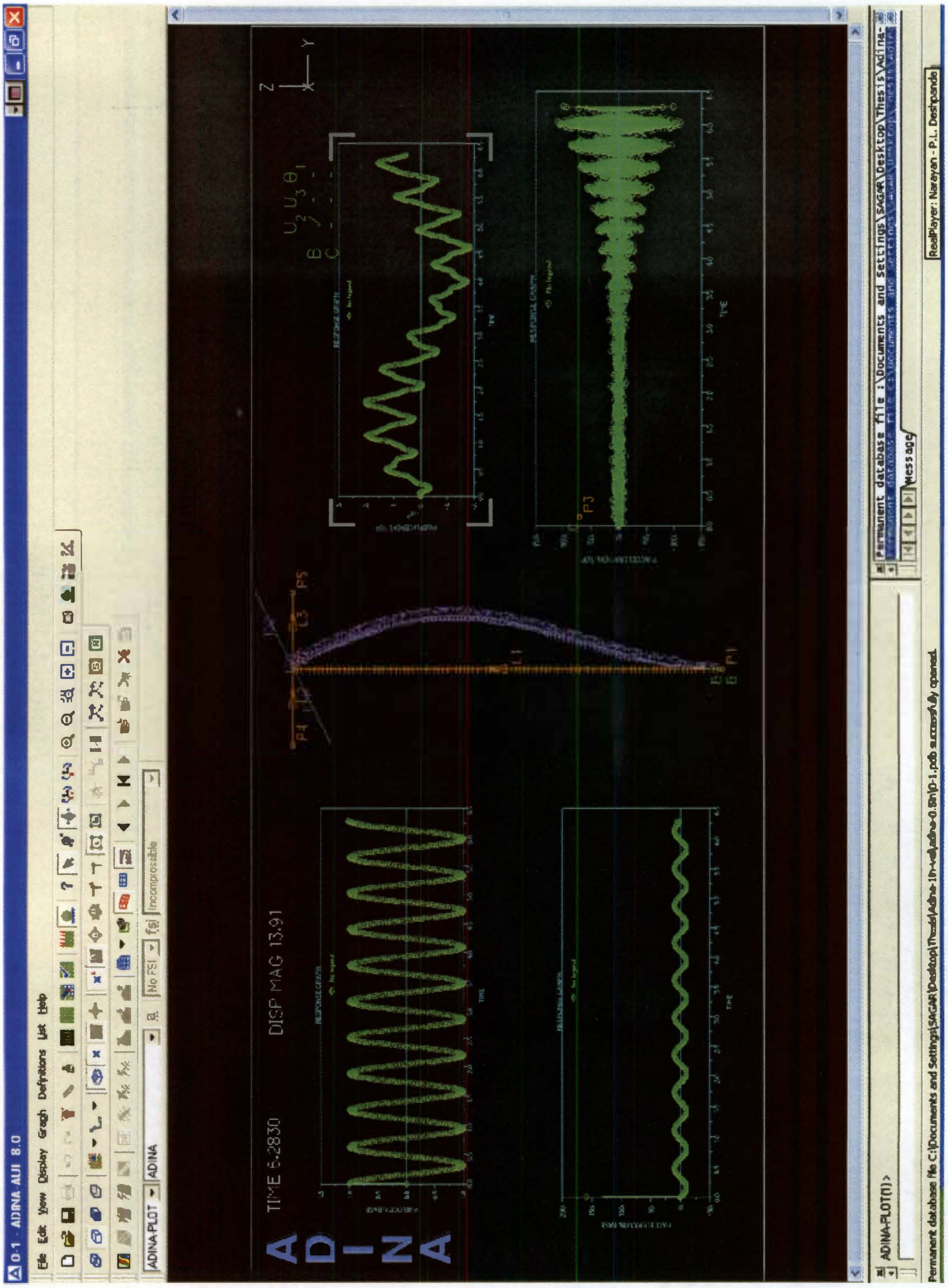


Figure-A-2 Model-2

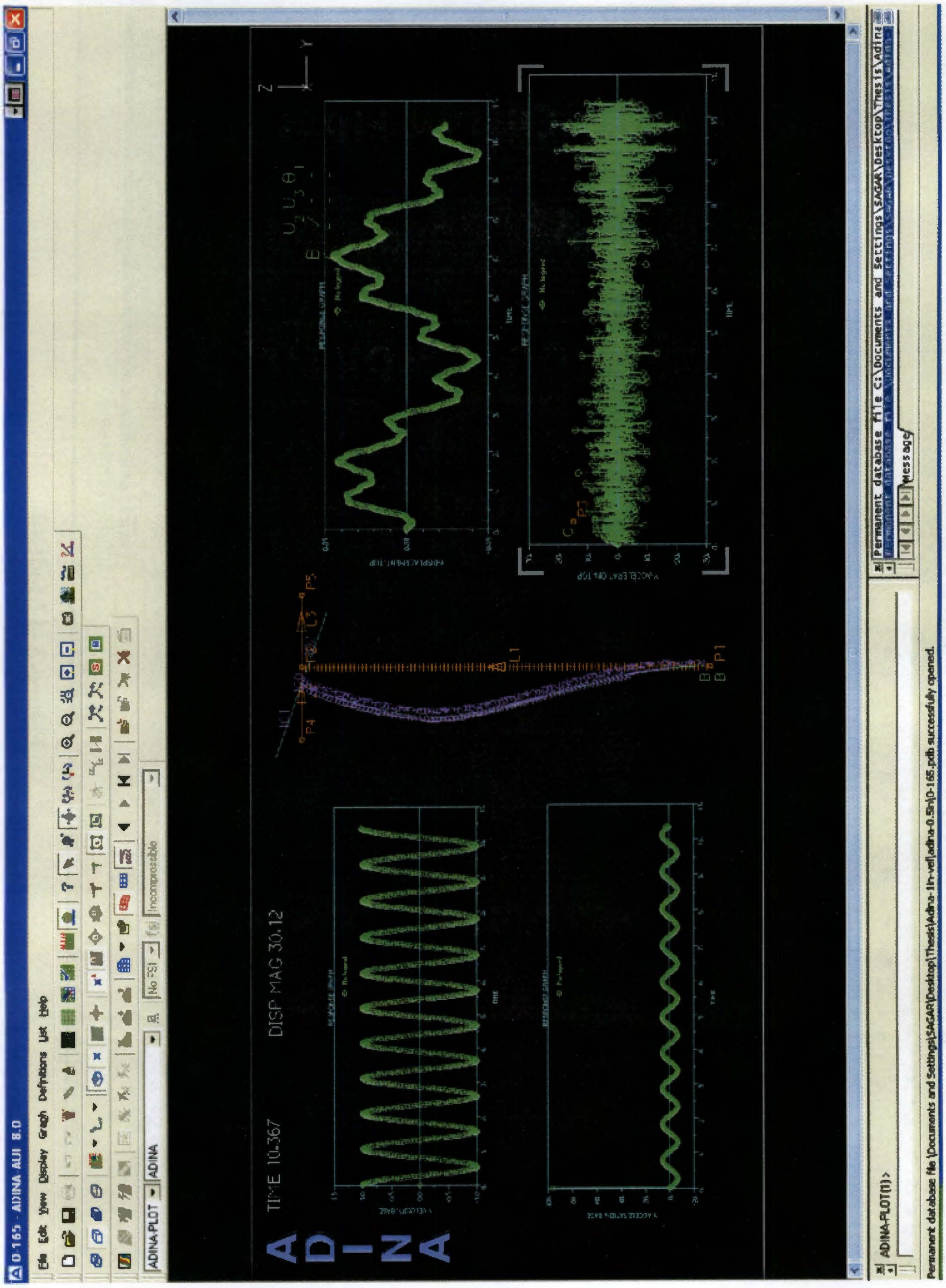


Figure-A-3 Model-3

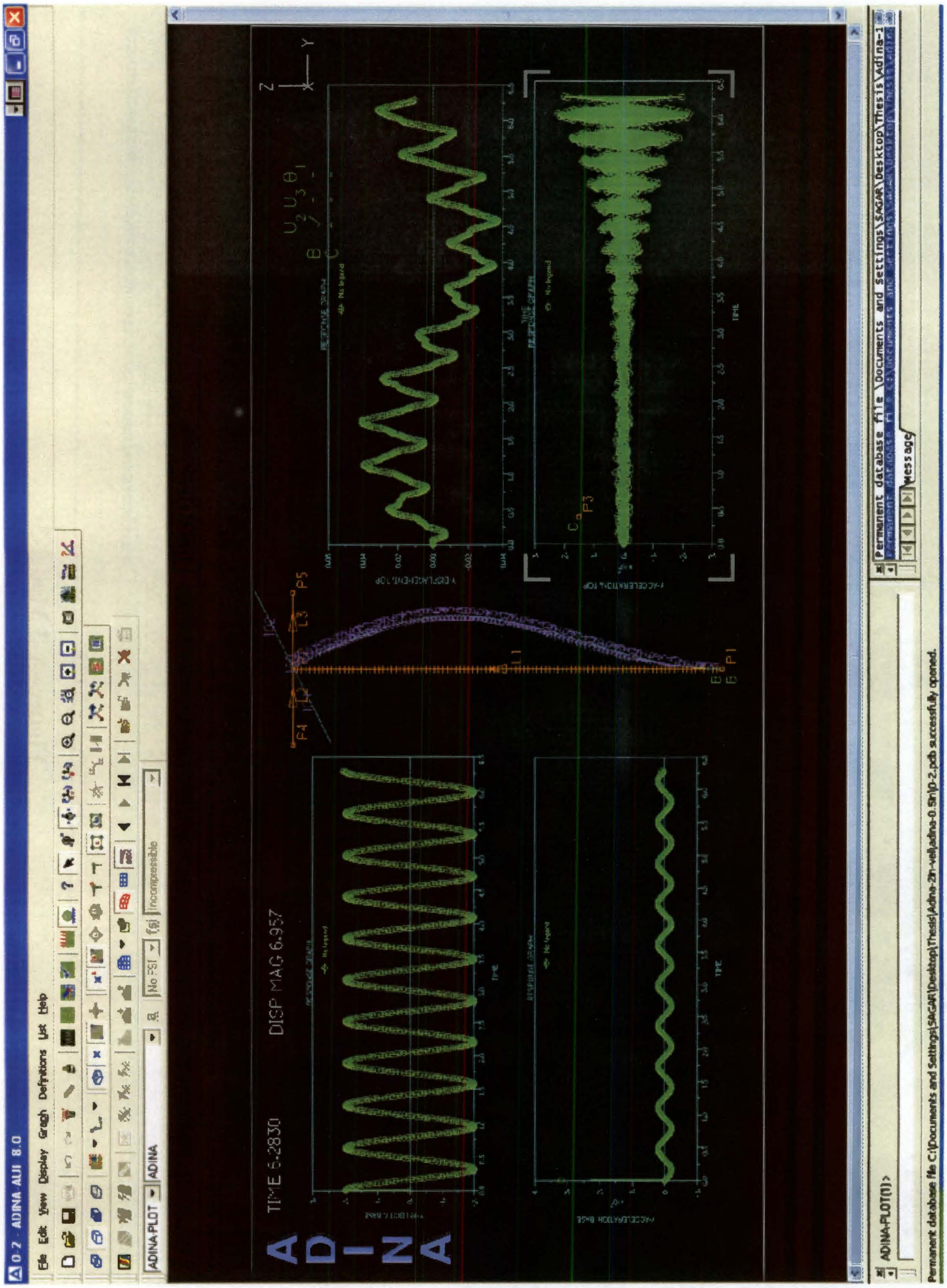


Figure-A-4 Model-4

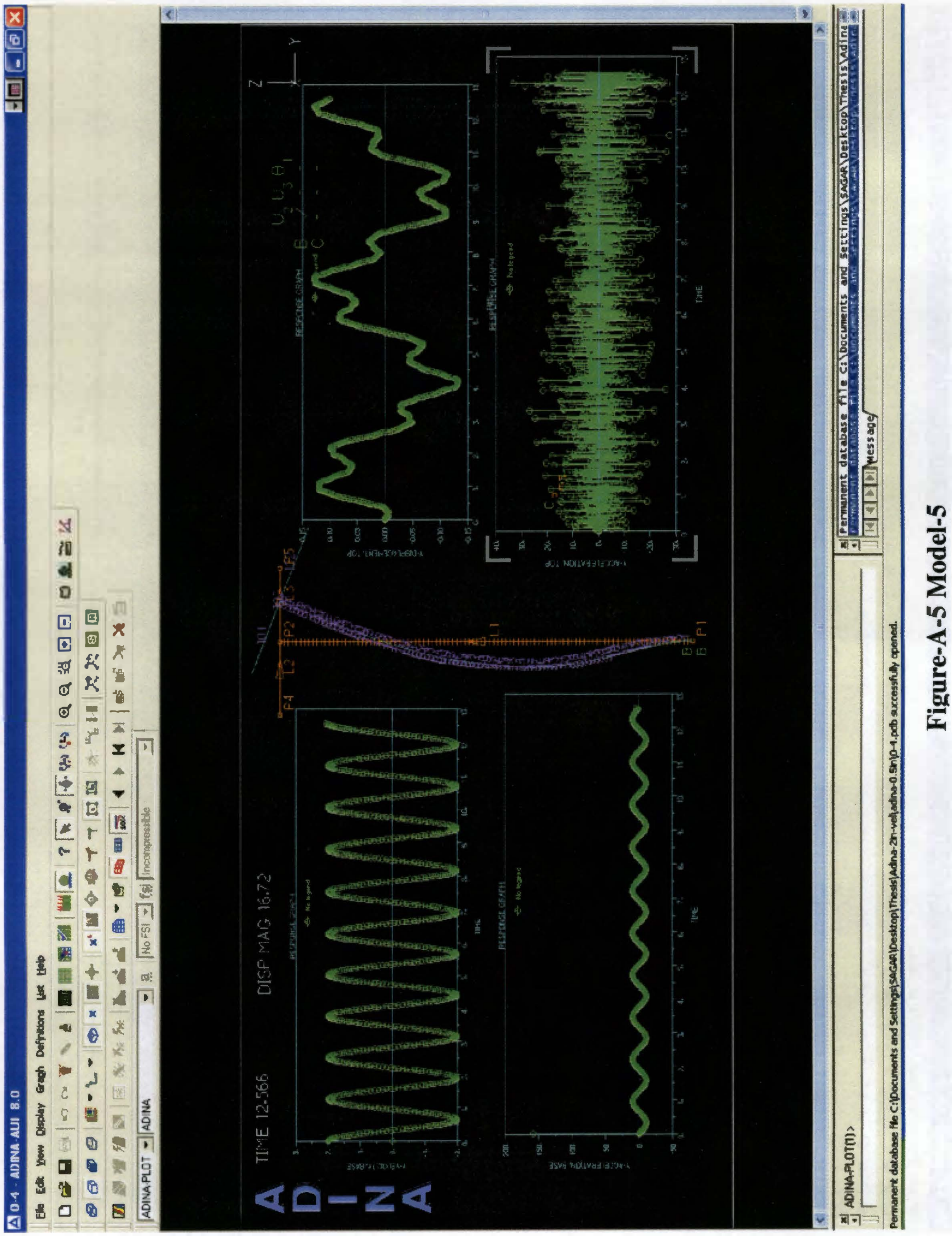


Figure-A-5 Model-5

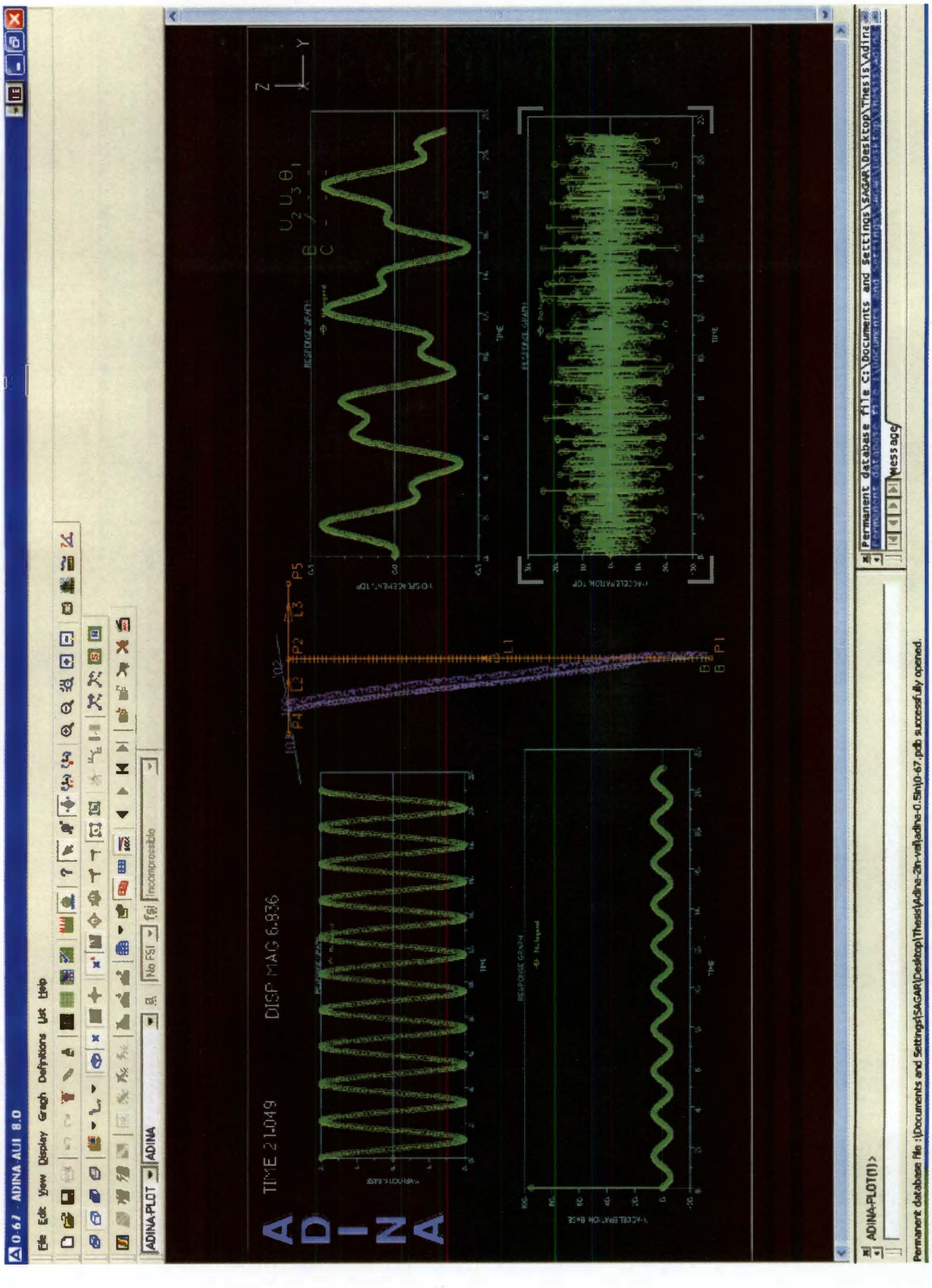


Figure-A-6 Model-6

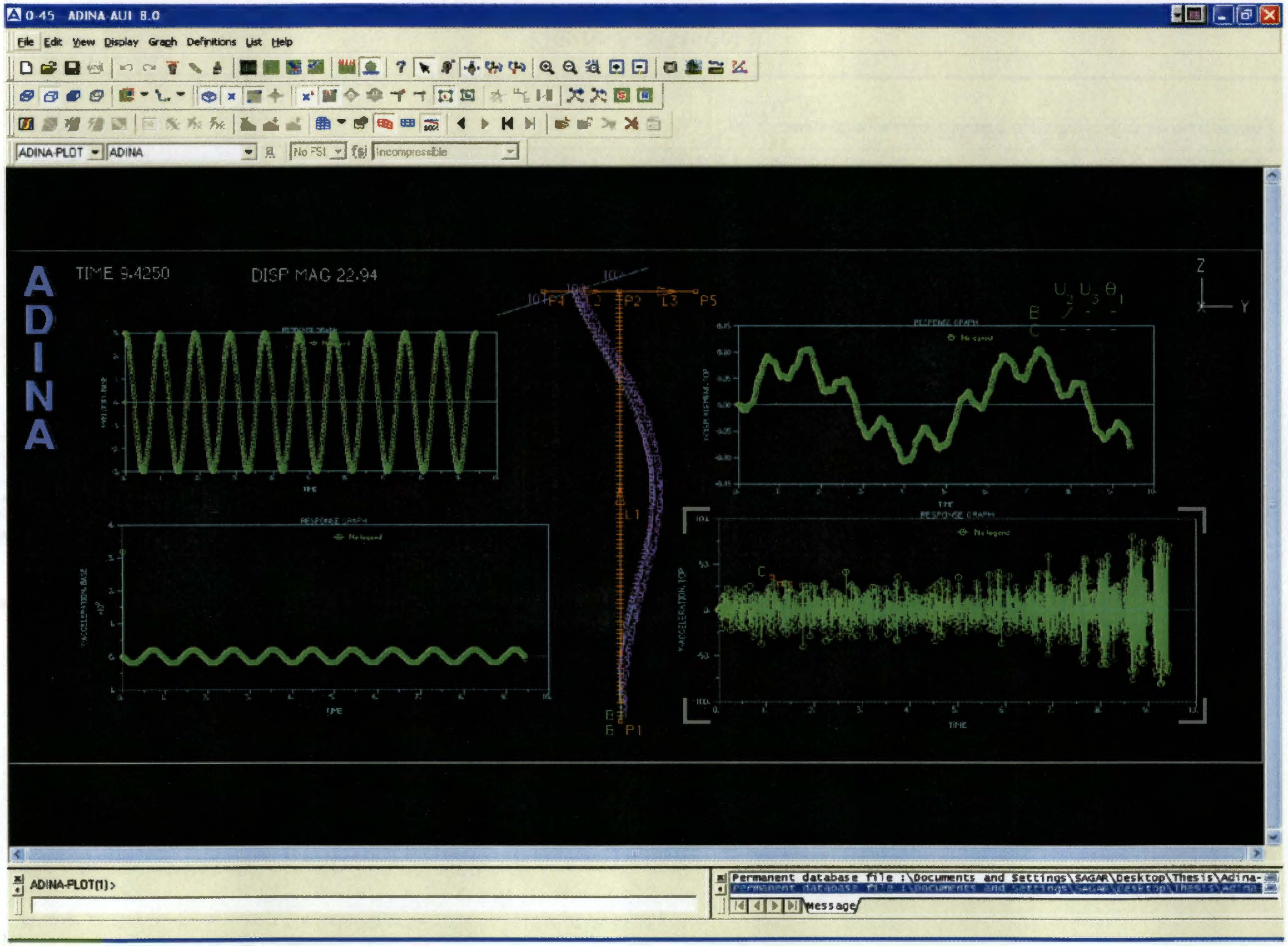


Figure-A-7 Model-7

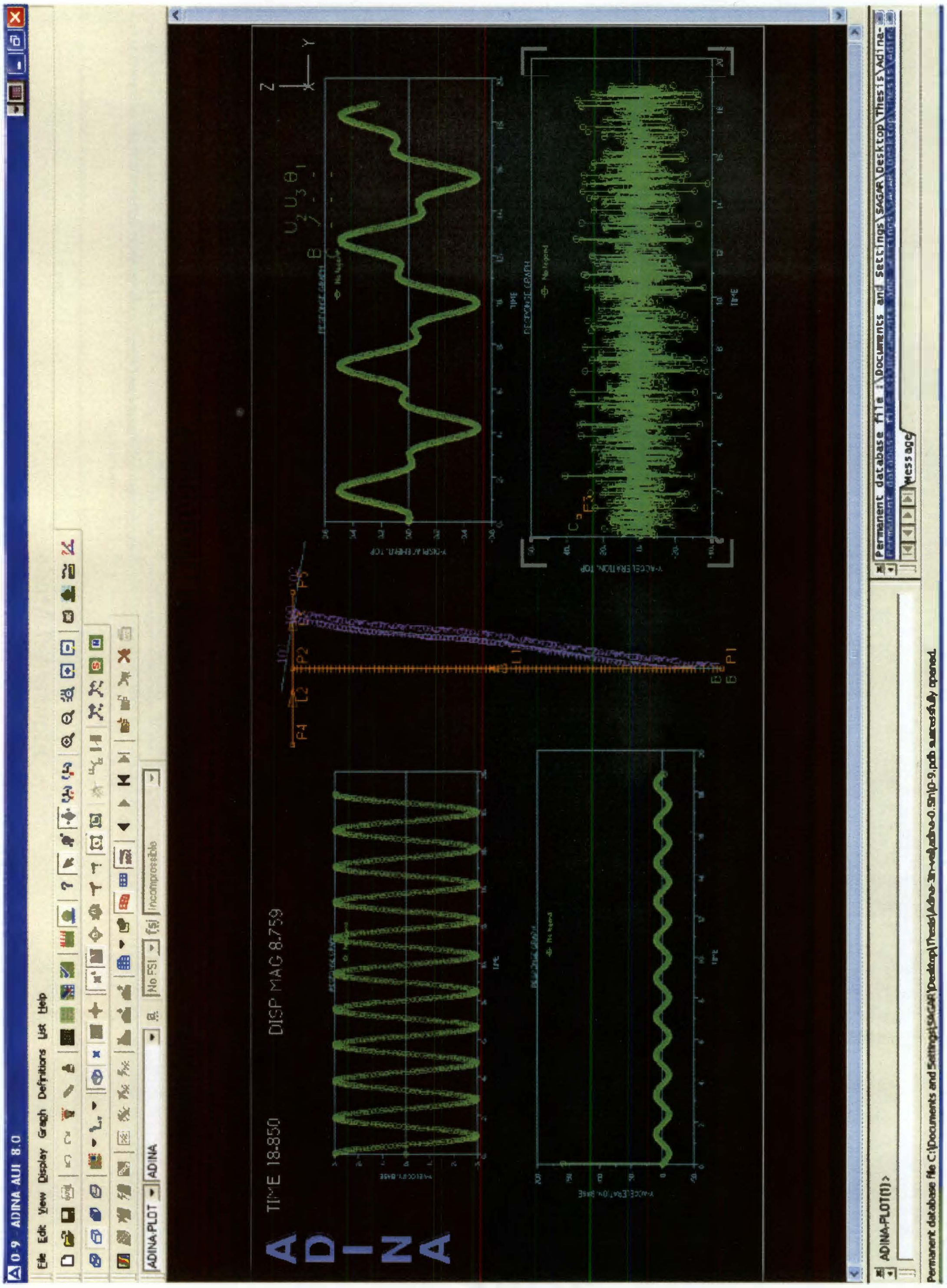


Figure-A-8 Model-8

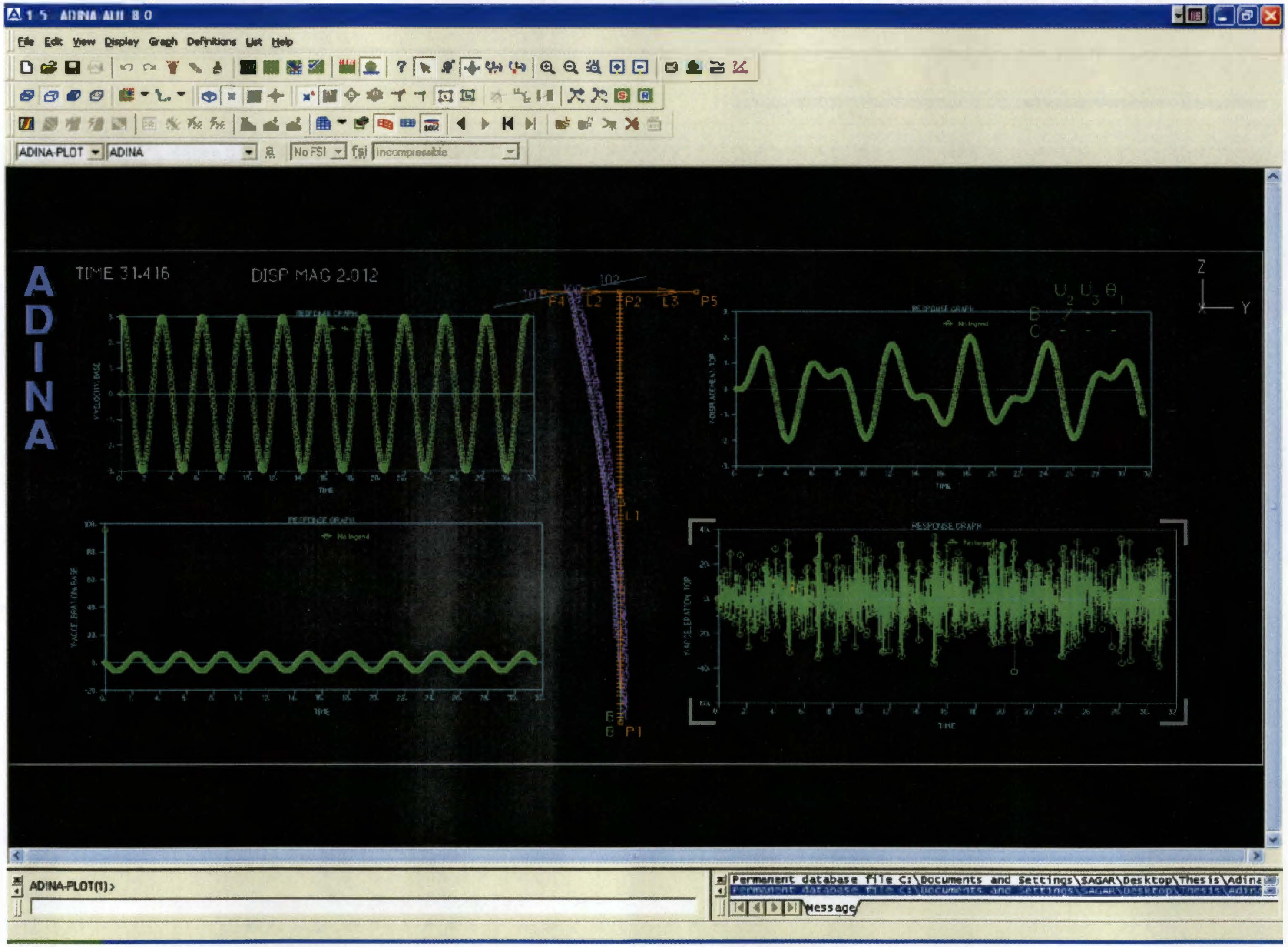


Figure-A-9 Model-9

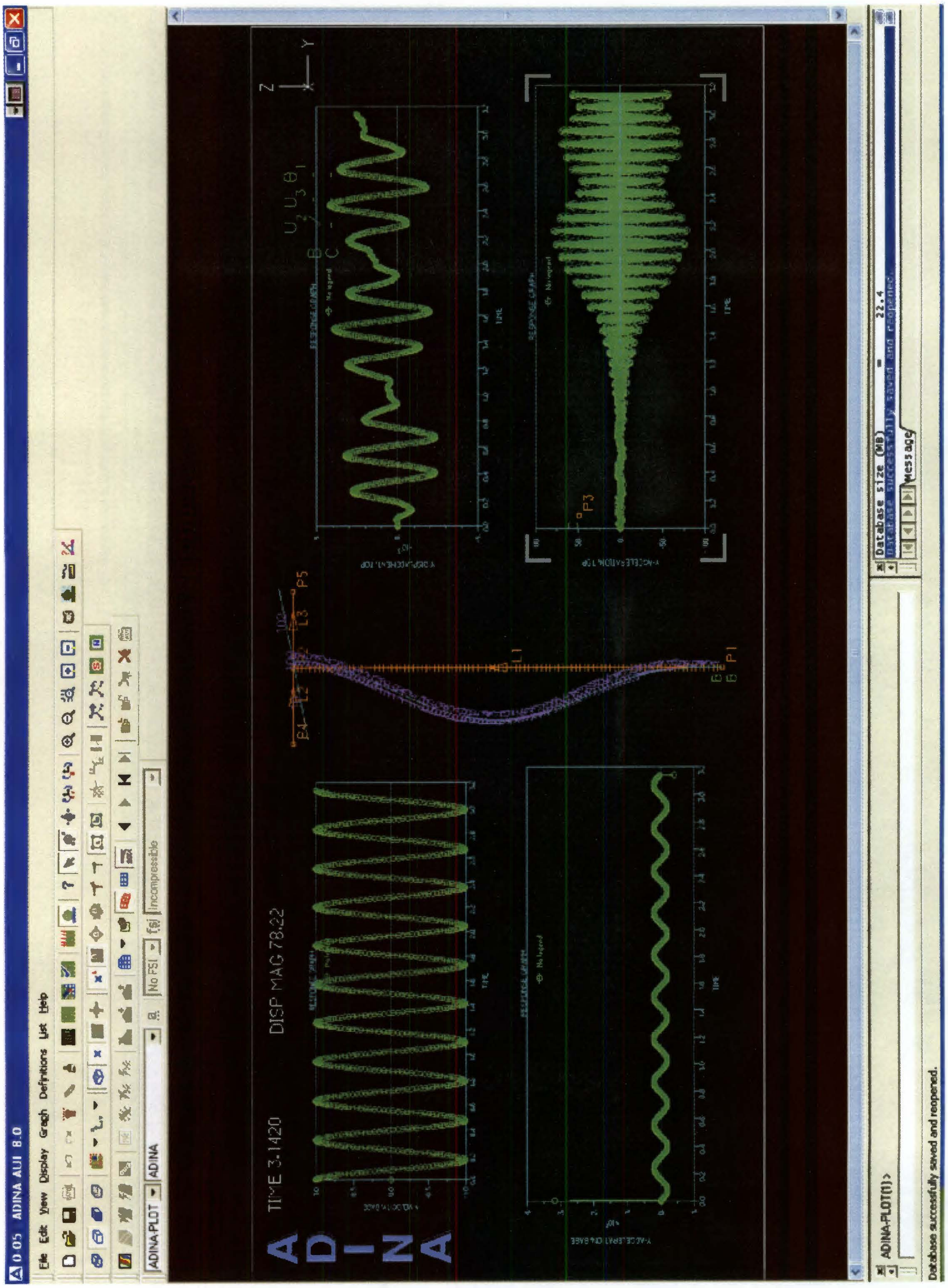


Figure-A-10 Model-10

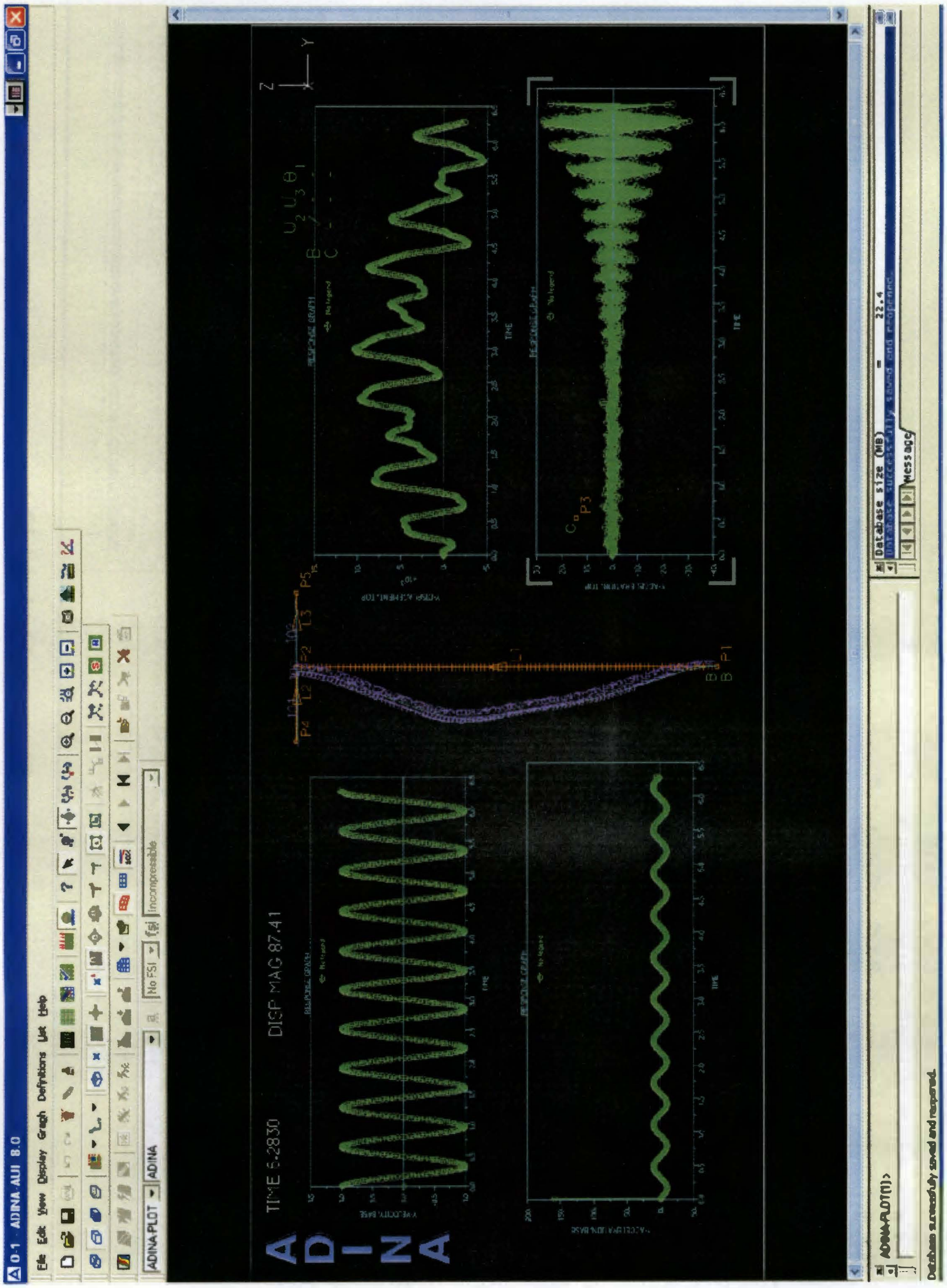


Figure-A-11 Model-11

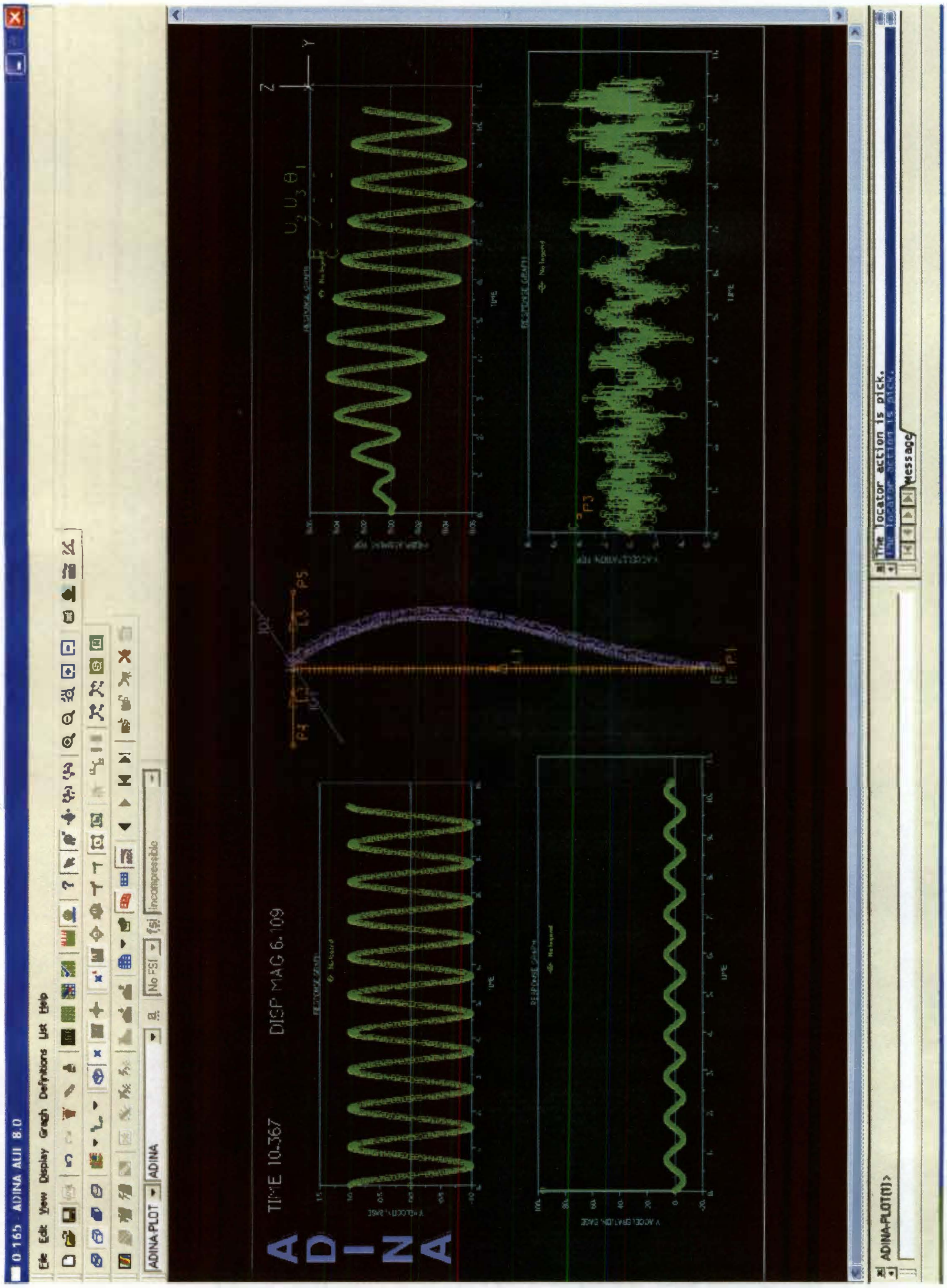


Figure-A-12 Model-12

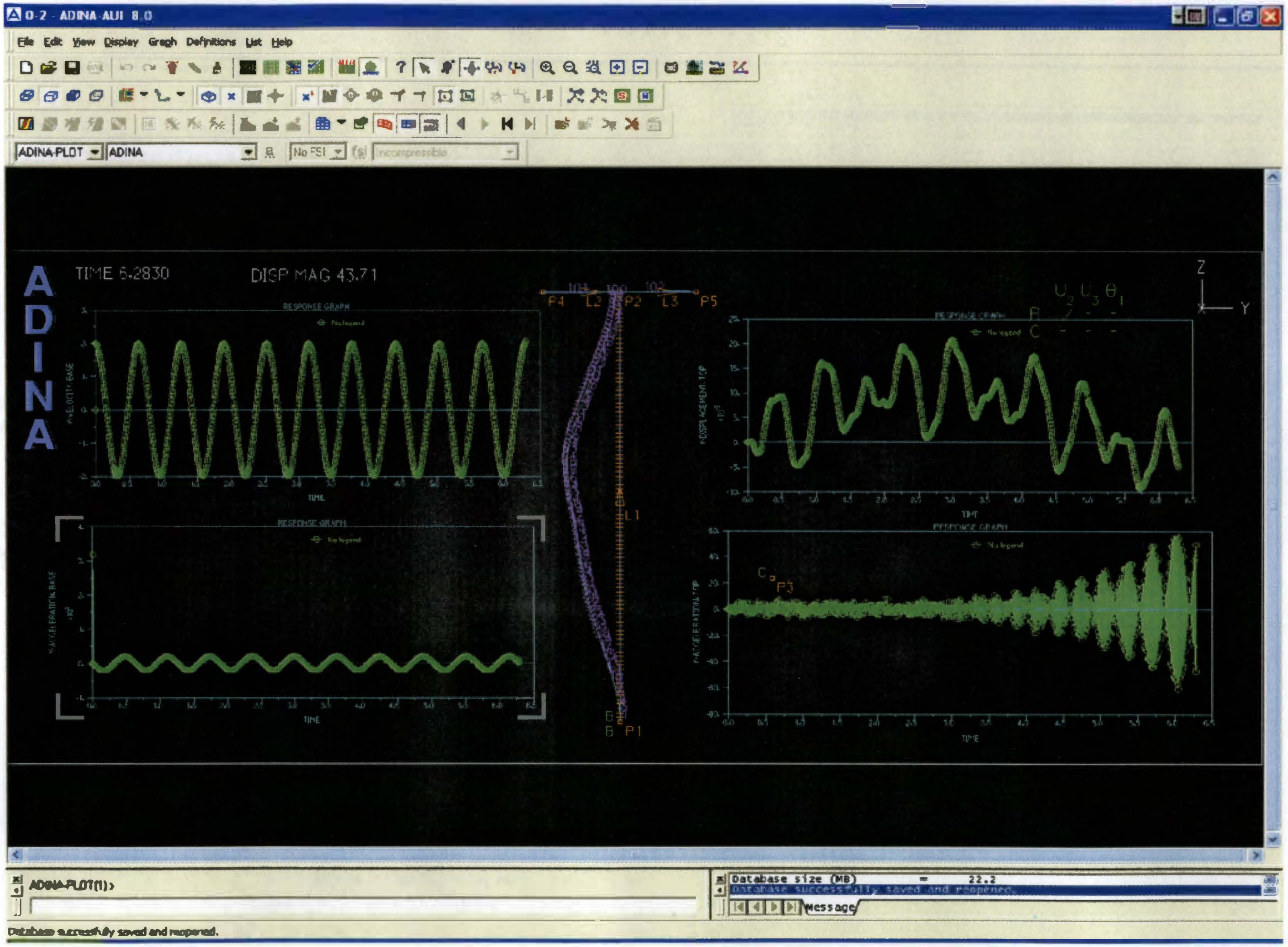


Figure-A-13 Model-13

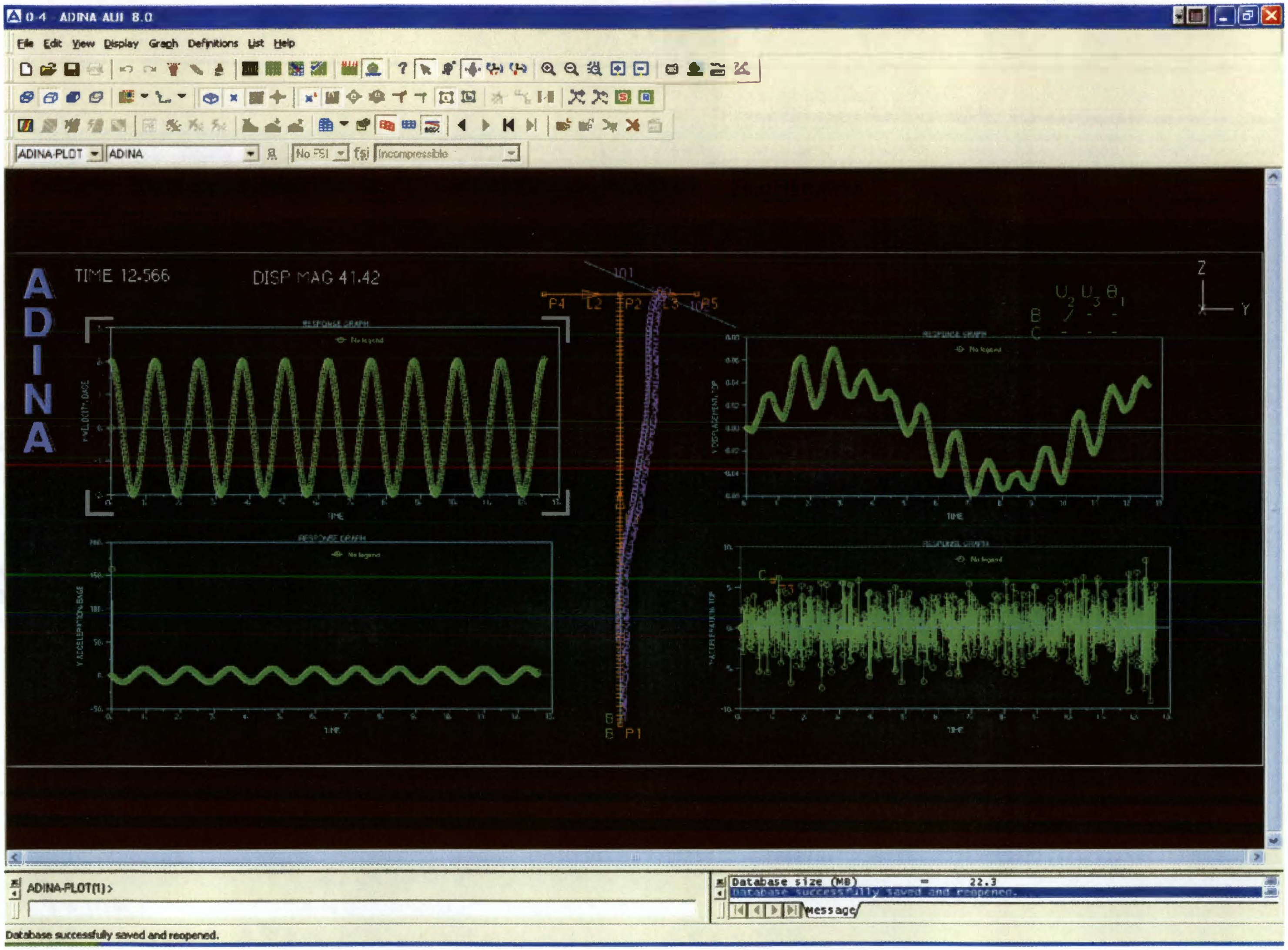


Figure-A-14 Model-14

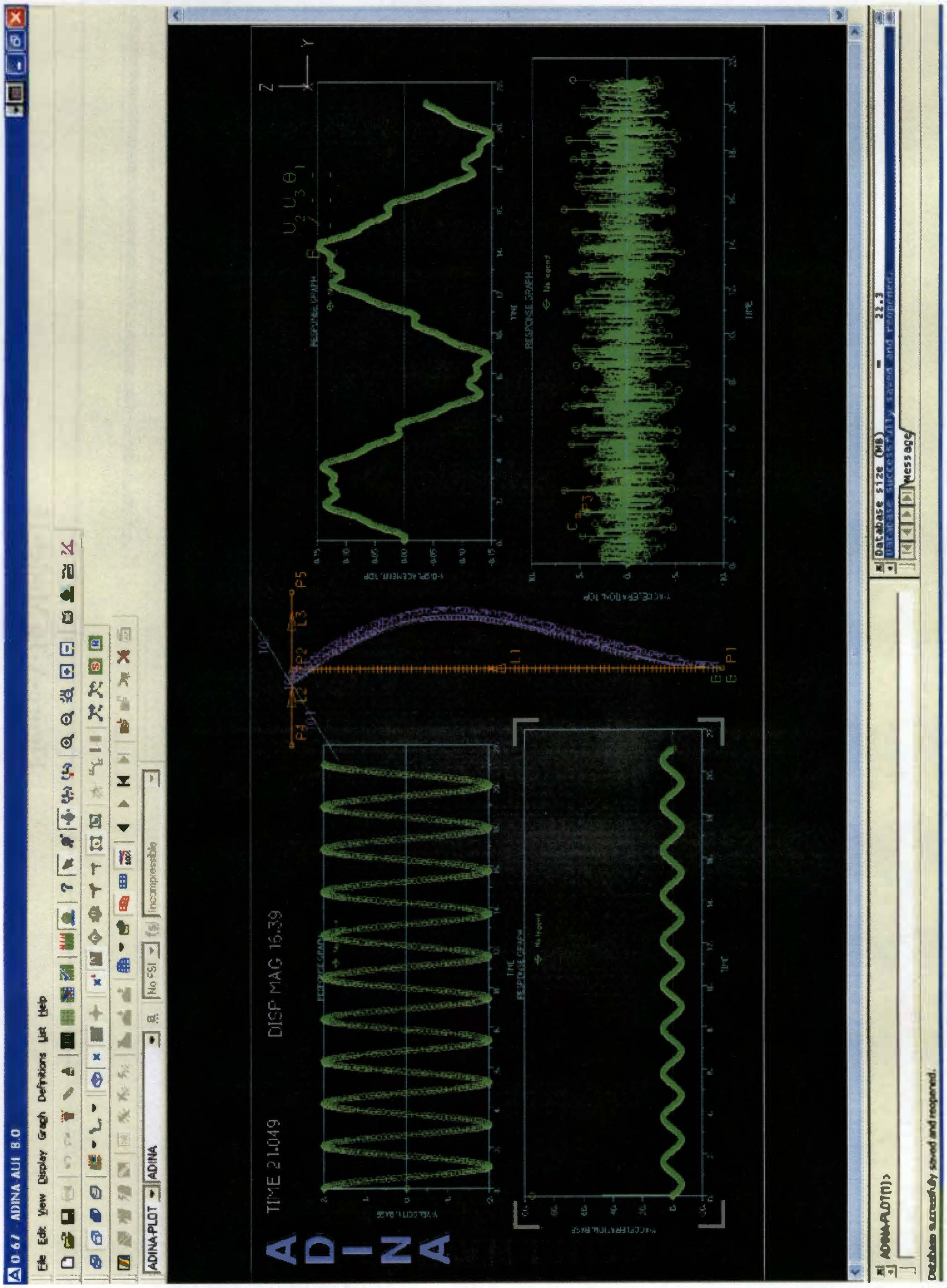


Figure-A-15 Model-15

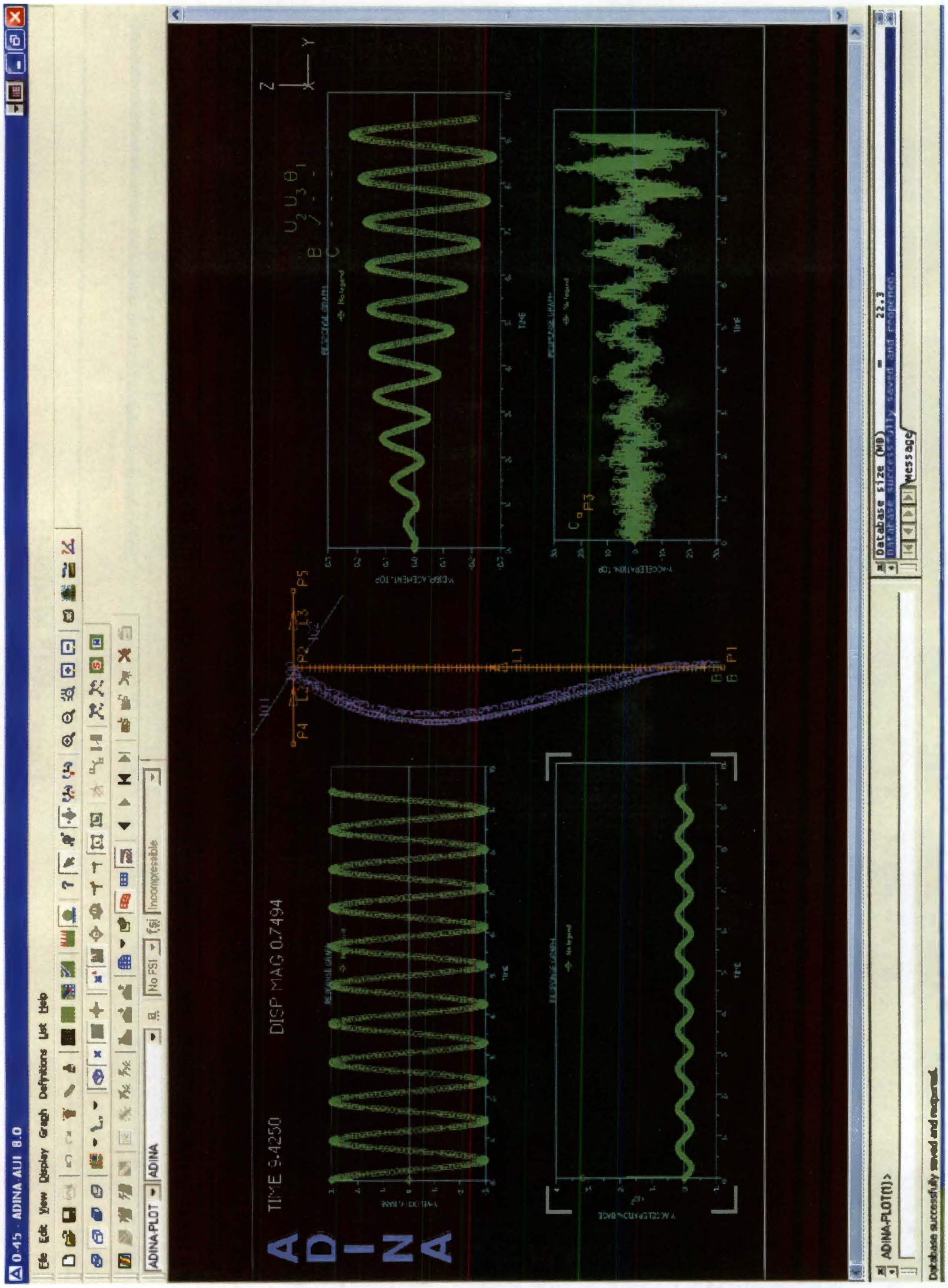


Figure-A-16 Model-16

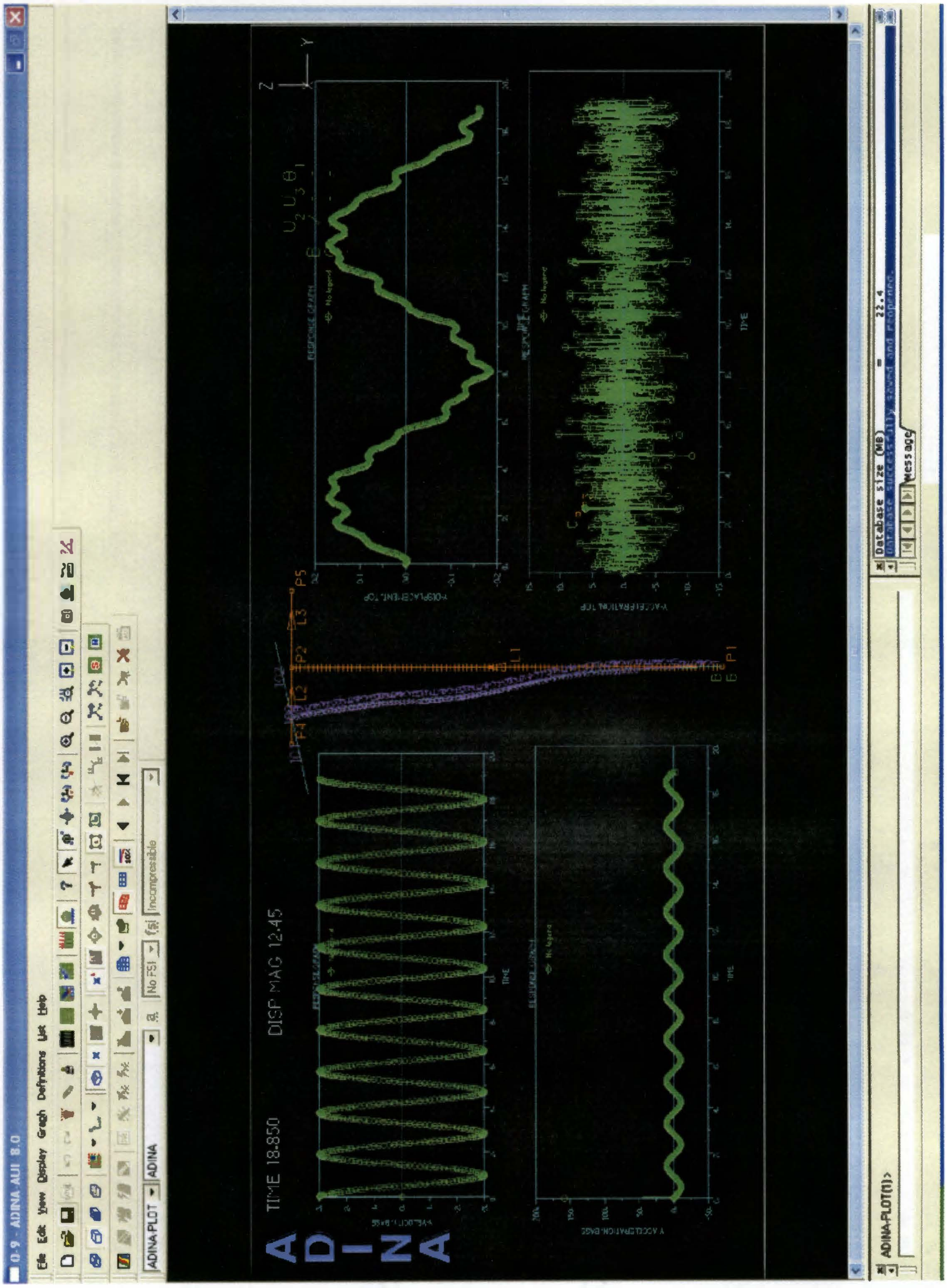


Figure-A-17 Model-17

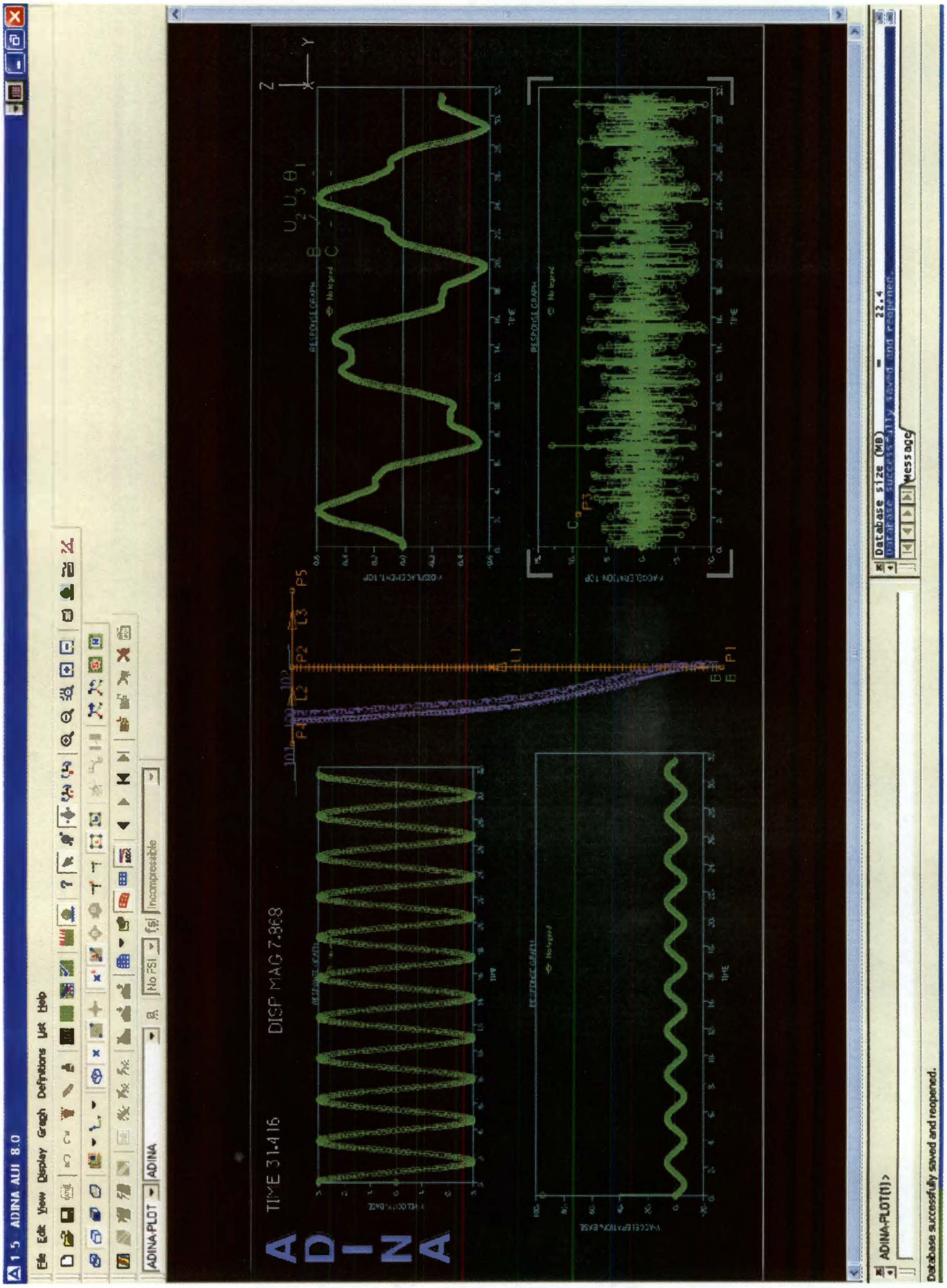


Figure-A-18 Model-18

APPENDIX B

PERMIT FROM ‘PEARSON EDUCATION’

PEARSON EDUCATION

One Lake Street
Upper Saddle River, New Jersey 07458
(201) 236-3281 Phone
(201) 236-3290 Fax
michelle.johnson@pearsoned.com

Michelle Johnson
Permissions Administrator
Legal Division 2C37

October 3, 2003

Sagar R. Kulkarni
3700 Sutherland Ave., #E16
Knoxville, TN 37919

Dear Mr. Kulkarni:

You have our permission to include figures 6.12.1 and 6.12.2 from our text, ***DYNAMICS OF STRUCTURE 2/E by Chopra,*** in your thesis.

The thesis is for the University of Tennessee and will be presented to Dr. Earl E. Ingram.

You may credit our material as follows
Chopra, DYNAMICS OF STRUCTURE, 2/E, ©2001, pp.241, 243. **Reprinted by permission of Pearson Education, Inc., Upper Saddle River, New Jersey.**

Sincerely,

Michelle Johnson
Permissions Administrator

Vita

Sagar Ramesh Kulkarni was born in Vasai, Maharashtra-India, on October 11, 1975. He was raised in Dombivli, in Thane District, where he attended Greens English School from K.G to grade ten. From there, he went to K.J.Somaiya Polytechnic and passed his Diploma in Civil Engineering with distinction in 1994. Later he joined Sardar Patel College of Engineering under University of Mumbai (Bombay) and graduated in 1997. After graduating from University of Mumbai in first class with honors, Sagar worked for the multinational consulting company Uhde India Limited in Mumbai for five years as a civil engineer. In 2002 he left the job to attend the graduate school in U.S at University of Tennessee, Knoxville and received a M.S in 2004.

He intends to pursue a job in United States.