DYNAMIC ANALYSIS OF TOWER STRUCTURES

A CASE STUDY :PTT KARS TELEVISION TOWER

A Master's Thesis

Presented by

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Approval of the Graduate School of Natural and Applied Sciences.

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I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Prof. Dr. Doğan ALTINBİLEK
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We certify that we have read this thesis and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science in Civil Engineering.

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ABSTRACT

DYNAMIC ANALYSIS OF TOWER STRUCTURES

A CASE STUDY :PTT KARS TELEVISION TOWER

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Structural analysis of PTT Kars Television Tower made both by using computer program Structural Analysis Program SAP90 and by equivalent static calculations using elementary beam theory.

Dynamic properties held by Ambient Vibration Experiment compared with the results of SAP90 computer model. It was observed that approximately the same results were obtained. It shows that, the computer model may generate correct results.

Dynamic and equivalent static calculations are compared. The results are completely different. Previous wind calculations made by the vendor are examined and the load results are compared.

It may be predicted that, the reason of the collapse is the underestimation of the dynamic loads and for special kind of structures like towers, dynamic loads are important and dynamic analysis should be held.

Key words : Tower Shaped Structures, Multi Element Structures,Vibrational Characteristics, Dynamic Properties, Structural Analysis, Equivalent Static Calculations

Science Code : 620.01.01

ÖZ

KULE TİPİ YAPILARIN DİNAMİK ANALİZİ

PTT KARS TELEVİZYON VERİCİ KULESİ

TUNCER, Mustafa

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Tez Yöneticisi : Prof.Dr. Çetin Yılmaz

Şubat 1993,129 Sayfa

PTT Kars Televizyon Verici Kulesi analizi, SAP90 Yapı Analiz Programı ile bilgisayarda dinamik olarak ve çeşitli eşdeğer statik yükleme standardları kullanılarak statik olarak ele alındı.

SAP90 yapı analiz programı ile serbest titreşim deneyi sonuşlarından elde edilen dinamik özellikler karşılaştırıldı. Sonuçların birbirlerine yakınlığı gözlendi.

Dinamik ve statik analizler karşılaştırıldı. Sonuçların tamamen farklı

olduğu gözlendi. Yapı ile ilgili daha önceden yapılmış statik rüzgar hesapları incelendi ve karşılaştırmalar yapıldı.

Binanın zarar görme sebebinin, dinamik yüklerin ihmal edilmesi olabileceği düşünüldü. Kule tipi yapılar gibi özel binalarda, dinamik analiz yapılması gerekliliği gözlendi.

Anahtar sözcükler : Kule Tipi Yapılar, Değişik Yapı Malzemeleri İle İmal Edilen Yapılar, Titreşim ve Dinamik Özellikler, Yapı Analizi, Eşdeğer Statik Yüklemeler

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I owe thanks to by bosses İsmail SALICI and Hasan ÖZDEMİR for their tolerance at Prokon and my family, my office friends Gülay KIR, Ömer ÇETİNKAYA and Ümit KAZAK for their helps those encouraged me during all of the study.

This work is dedicated to my father M.İsfendiyar TUNCER, who supported me for being a civil engineer and dedicated his life to me.

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NOMENCLATURE

Av	seismic coefficient representing the Effective Peak Velocity-Related Acceleration
c	damping coefficients
C	seismic design coefficient
C_0	seismic zone coefficient
C_d	the deflection amplification factor
C_{vn}	lateral load coefficient for nth level
$\sim C_d$	modified deflection amplification factor
D	width of the building at the foundation level
F_b	seismic base shear
F_n	lateral seismic force at nth level
F_t	lateral load applied on the top level of the structure
h_n	the height in feet above the base to the highest level of the building
H	height of the building
I	structural importance coefficient
k	spring constants
k_{ij}	stiffness influence coefficients
K	building type coefficient
K_n	generalized stiffness of nth normal mode

L	the overall length in feet of the building at the base in the direction under consideration
m	mass
M_n	generalized mass of nth normal mode
p(t)	forcing function
r_n	ratio of shear capacity to design shear for nth level
R	the seismic response modification factor
$\tilde{}R$	modified seismic response modification factor
S	coefficient for the soil profile characteristics, or structural dynamic
T	the fundamental period of the building
T_a	the approxiamate fundamental period for the building
u	displacement
u	velocity
u	acceleration
v_o	iteration starting vector
V	vector
w	undamped natural circular frequency
W	total weight of the building
ϕ_n	nth mode shape
∩	principal coordinates

Ω	eigenvalue diagonal matrix
Φ	orthonormalized eigenvectors matrix

CHAPTER I

INTRODUCTION

1.1. General

After the collapse of the top GRP(Glass Reinforced Plastic) element of PTT Kars Broadcasting Tower, following the Spitak (Armenian) Earthquake of December 7th, 1988, owner of the antenna PTT began to investigate the case. The interesting side of the collapse was that no other building had any damage during the earthquake but only the top portion of the antenna structure. The collapse was at the connection of the GRP antenna to the tower structure.

It was thought that some special dynamic characteristics of the tower may arouse the collapse. The glass fiber reinforced cylinders, supporting the 2nd channel TV antennas of KARS TV station, was at the top of previously built 85 meter high tower. The tower has three main parts:

1. A cylindrical shaped, reinforced concrete accommodation building

2. A cylindrical shaped, reinforced concrete variable section tower placed at the core of the accommodation building

3. A cylindrical shaped steel tower at the top of the reinforced concrete tower.

A detailed soil investigation, geological and technical specifications prepared by another team at METU and the ambient vibration of Kars Television Tower have been measured on November 7-8, 1990.

1.2. Problem

The GRP antenna was carefully investigated by Alan Dick & Company Limited of England for wind load affects and it was stated that lateral seismic affects are not governing for the antenna. Lateral load analysis of the antenna was held by taking the GRP section alone and load affects coming from the dynamic characteristics of the tower structure may be underestimated.

Yılmaz, Erdik and Akkaş [18] stated that "Analysis of the GRP section has been held as a separate structure in the calculations by the vendor and no interaction with the tower structure is considered. This error in the calculation is underestimate the structural stresses during dynamic loadings."

1.3. Scope

The dynamic characteristics of the Kars TV Antenna Structure have been measured by another team at METU and the ambient vibration of the tower have been measured.

In this paper, the finite element model of the structure is studied and dynamic characteristics are held. The tower was defined like a cantilever bar, that is fixed at the base, so the computer model was not a structurally well detailed model but the mass along the tower are modelled in detail. It is tried to be perfect on mass of the structure. The results of tests and mathematical model are compared.

After comparison of the frequency results of test and mathematical model, load effects at some critical points of the tower are investigated. In this study, the dynamic properties of the tower are investigated by the use of a computer model and results are compared with equivalent static load design algorithms.

CHAPTER II

LITERATURE SURVEY

For the analysis of parabolic cooling towers Arya and Thakkar[13], indicated that the exact nature of the equivalent lateral loads along the height of hyperbolic cooling towers is not known and can only be predicted by dynamic analysis.

Morrone[14] in his investigations on a self supporting stack had shown the practical accuracy of using the lumped parameter method to determine the fundamental frequency.

Radhakrishnan[16] studied the response to transient excitation with the effects of additional constrained masses and suggested that the inherent structural damping in the structure is the controlling parameter on the maximum stresses and deformations when subjected to dynamic loads. In his studies, it is concluded that

a.The use of elementary beam theory predicts the principal natural bending modes and frequencies quite closely for the case of triangulated frame tower structure.

b.The damping characteristics demonstrate an important difference in the damping factors associated with bending and torsional modes.

c.The oscillation decay method indicated a variation of damping factor with oscillation amplitude number.

Newton[17], while analyzing a tower model, has found that beam theory predicts results quite close with the experimental values of frequencies and mode shapes, but does not give any indication of the increasing complexity of the motion as the forcing function increases. He investigated the variation of damping with oscillation amplitude using the energy dissipation method. He concluded that theoretical models for the damping mechanisms, typical of the tower frame work should be of the form of equation but taking into account the fact that the damping index itself increases overall stress amplitude.

CHAPTER III

DESCRIPTION OF THE STRUCTURE

3.1. General

PTT Kars TV Antenna is designed to serve as a TV broadcasting antenna, shelter for equipment and accommodation for workers. The Kars TV tower is a multi material building : a concrete building at the base, a reinforced concrete circular wind shield, and a steel circular cylinder with the GRP cylinder mounted on the top.

Tower, has two main parts. First is a 5 story cylindrical shelter and accommodation building between +0.00 and +14.10 elevations and second is a cylindrical tower structure which is placed inside of the first structure eccentrically.

The shelter and accommodation structure is R=14m cylindrically shaped, 5 story reinforced concrete structure. Floor elevations are +0.000m, +3.000m,

+5.400m, +8.600m, +11.300m and +14.100m. +14.100m elevation is the terrace floor and staying broadcasting antenna equipments. Other floors are used for accomodation of technicians and broadcasting equipments.

The tower structure has 6 different structural geometry with three different type of structural material, placed inside of the shelter structure eccentrically : reinforced concrete, steel, and GRP materials placed on each other. The properties of the steel cylinder which is between levels 67.20m and 82.10m are tabulated in Table 2.1 and the cross section and some geometrical properties of reinforced concrete section are tabulated in Table 3.2.

a.Between +0.000m and +14.100m elevations, a 35cm thick reinforced concrete cylindrical shear wall, which is also serving as ladder tower for shelter and accomodation building.

b.Between +14.100m and +26.000m elevations, a 25cm thick reinforced concrete cylindrical shear wall with outer radius of 4.50m.

c.Between +26.000m and +30.000m elevations, a 150cm thick reinforced concrete cylindrical passing zone with outer radius of 4.50m.

Table 3.1 Properties of Steel Cylinder

Level	Length (m)	r_i (m)	t_i (mm)	F (m^2)	I (m^4) $x10^{-6}$
76.20~82.10	5.90	0.60	8	0.03036	5538.2
67.20~76.20	9.00	0.60	10	0.03801	6957.4
64.20~67.20	3.00	0.60	14	0.05339	9837.9
60.90~64.20	3.30	0.60	16	0.06112	11299.4

d.Between +30.000m and +60.900m elevations variable section cylindrical shear wall: at +30.000m elevation with outer radius of 2.40m and 45cm wall thickness; at 60.900m elevation with outer radius of 2.40m and 30cm wall thickness.

e.Between +60.900m and +82.100m elevation, a 18mm thick cylindrical steel plate wall with inner radius of 1.20m.

f.Over +82.100m GRP TV antenna structure.

Table 3.2 Properties of Concrete Cylinder

Level	Length (m)	r_o (m)	t_{lower} (m)	t_{top} (m)	F_{ay} (m^2)	I_{ay} (m^4)
54.00~60.90	6.90	1.15	0.30	0.30	1.884955	0.963683
48.00~54.00	6.00	1.15	0.30	0.30	1.884955	0.963683
42.00~48.00	6.00	1.15	0.30	0.30	2.016509	1.009830
36.00~42.00	6.00	1.15	0.30	0.30	2.267837	1.090333
30.00~36.00	6.00	1.15	0.30	0.30	2.503456	1.156675
26.50~30.00	3.50	2.25	0.30	0.30	10.178760	17.520190
21.50~26.50	5.00	2.25	0.30	0.30	3.337942	7.562524
19.15~21.50	2.35	2.25	0.30	0.30	3.337942	7.562524
16.80~19.15	2.35	2.25	0.30	0.30	3.337942	7.562524

On the tower, there are antenna platforms at +23.000m, +30.000m, +60.900m, and +82.100m elevations. Platforms at +23.000m and +30.000m are reinforced concrete and others are structural steel.

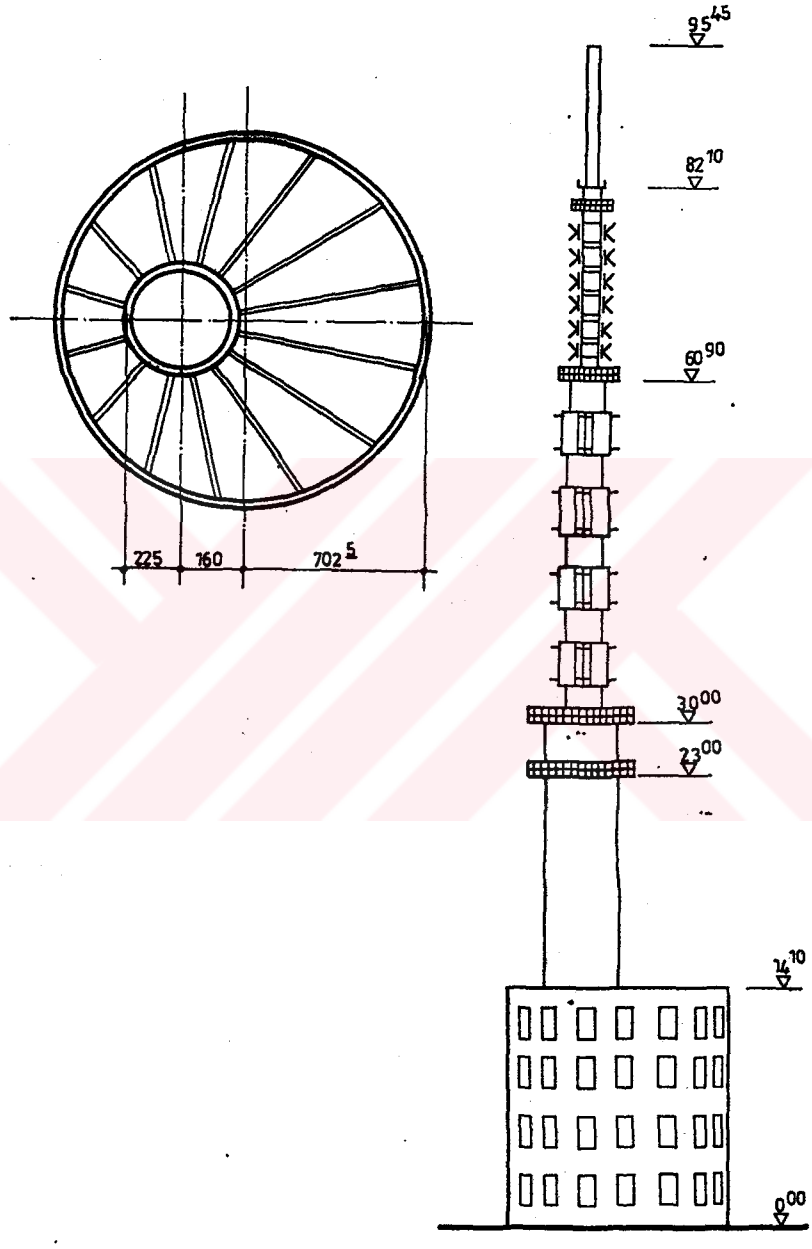

Figure 3.1 General Outline of the Tower

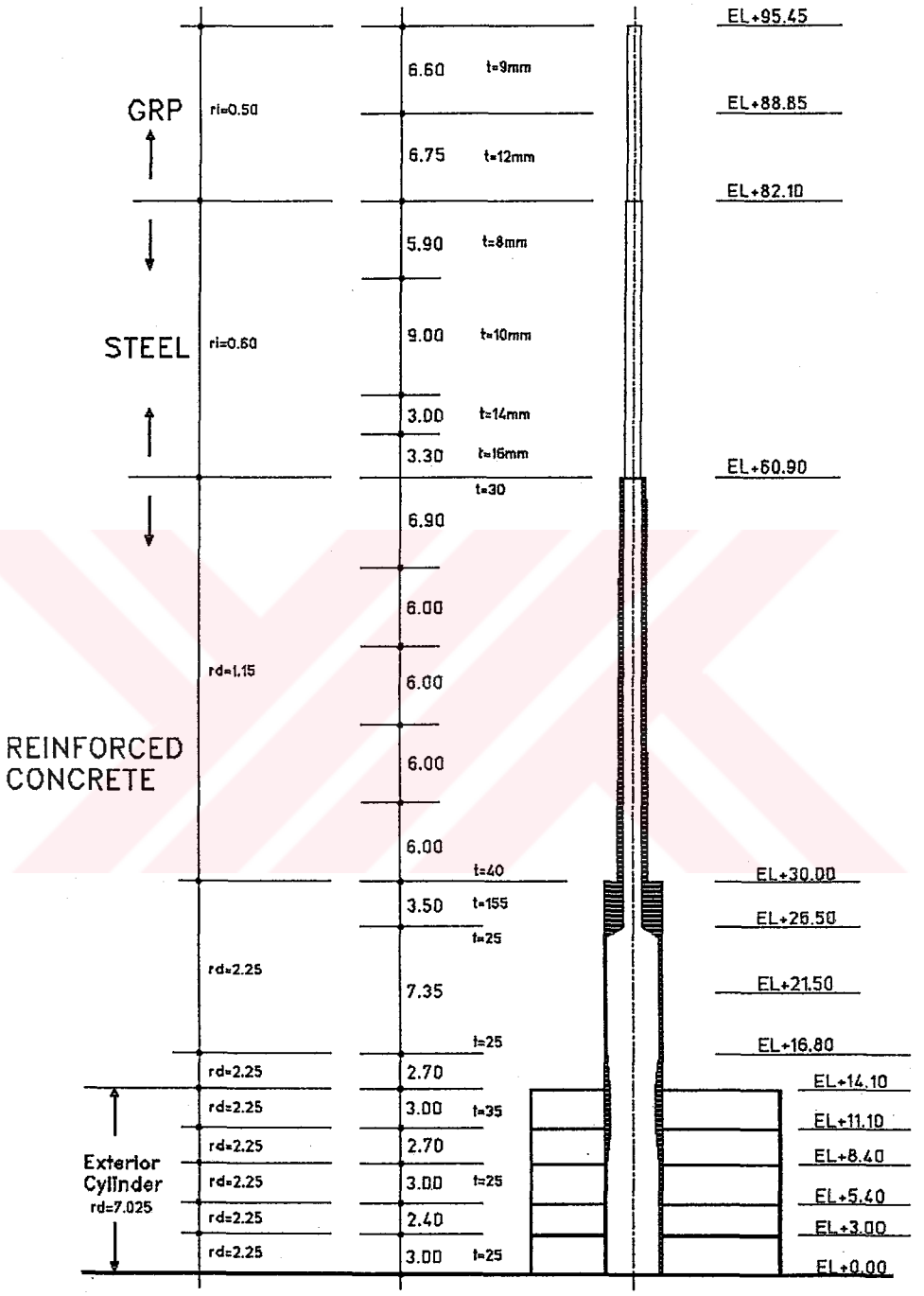

Figure 3.2 Summary of the Geometry of the Tower

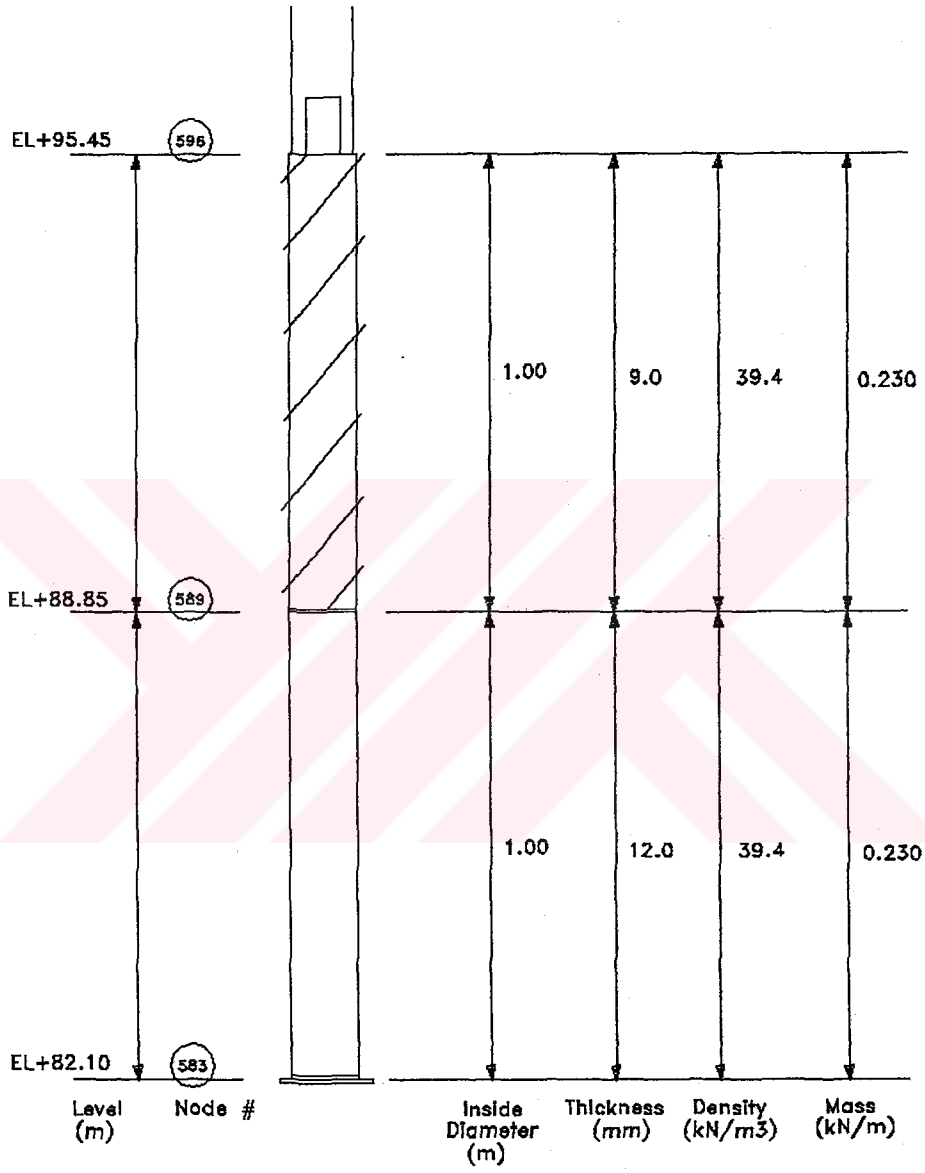

Figure 3.3 Geometry of GRP Part

CHAPTER IV

FINITE ELEMENT MODEL OF THE STRUCTURE

4.1. General

Finite element model of the structure is established by using SAP90 Structural Analysis Program Version 5.10 that have the capability of carrying out static and dynamic earthquake analysis of all kinds for linear structures.

Computer model of the structure is begin at the foundation level (+0.000). At +0.000m elevation, 15 nodal points are modelled for foundation. All nodal point boundary conditions at foundation level is assumed as fixed. The tower structure is modelled as a simple bar, and the foundation of the tower is presented by one node only. This modelling technique is established to minimize the run time for the dynamic analysis.

The building section of the structure has a star shaped beam orientation

inside of the cylindrical shaped outer beams. All dimensions and geometry is modelled using Foundation Formwork Plan and Sections Drawing that is provided by PTT.

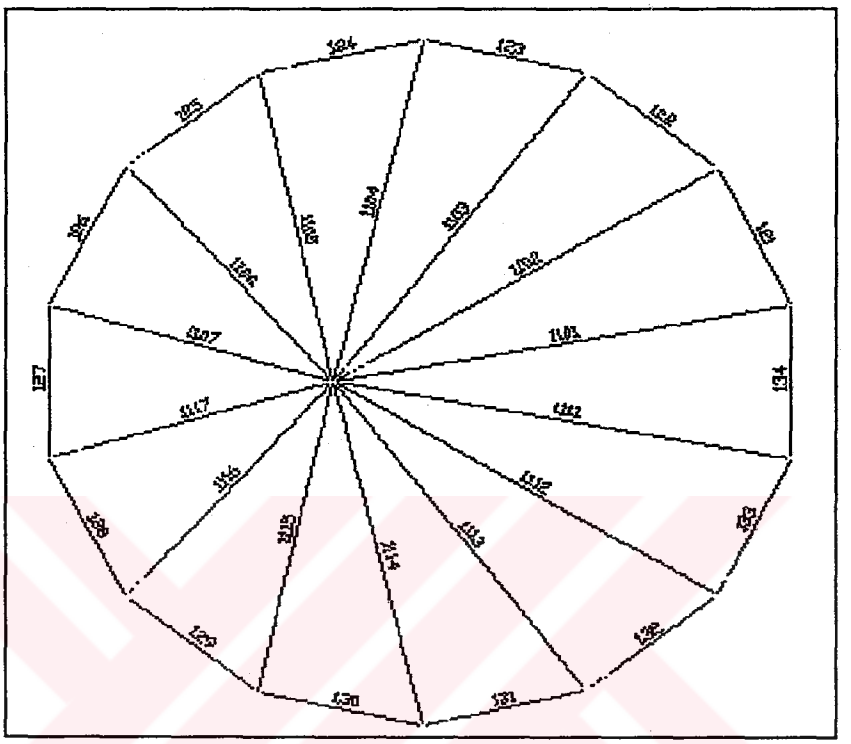

Figure 4.1 Finite Element Layout at +3.00m Elevation

At the model of bottom sections, up to +14.10m elevation, the inner beams are modelled longer than their real lengths. Length of the beams are corrected by defining rigid zones at the inner ends. For each beam that is modelled longer than its real length, rigid zones are defined for the half radius of the tower section.

Outer shear walls of the building section is defined by using shell elements. As the stress values on the shear walls are not of interest to its study, only one shell element is used for every span.

Around the shell elements, 20cmx20cm dummy beams and columns are defined. These dummy frame elements are used to examine the load distribution within the structure. Floors of the building section of the structure is modelled by using constraints at nodal points at floor levels. The lateral displacements of every nodal point at a floor level are constrained.

For all sections of the building the real section properties are defined. Building dead loads are taken into consideration by assigning gravitational loads to the sections. Assumed equipment dead loads are included in the self weights of the building sections.

No special modelling technique is used neither for the model of transition zone from reinforced concrete to steel nor from steel to GRP. For the modelling of antenna weights at +30.000m and +60.900m elevations, additional masses are defined at platform levels.The details of computer model are presented in Figures 4.1 to 4.17.

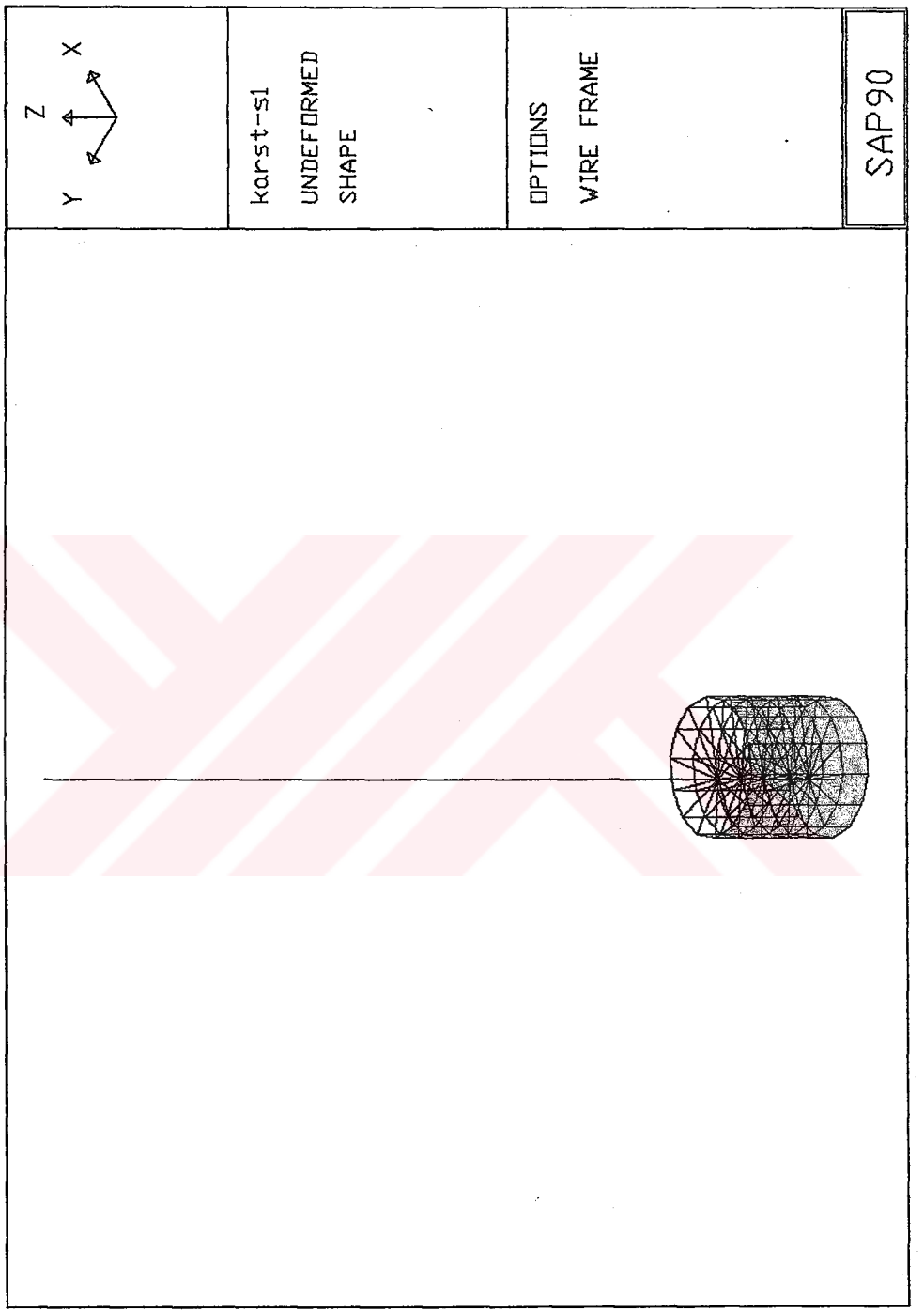

Figure 4.2 Model of the Structure with Shear Walls

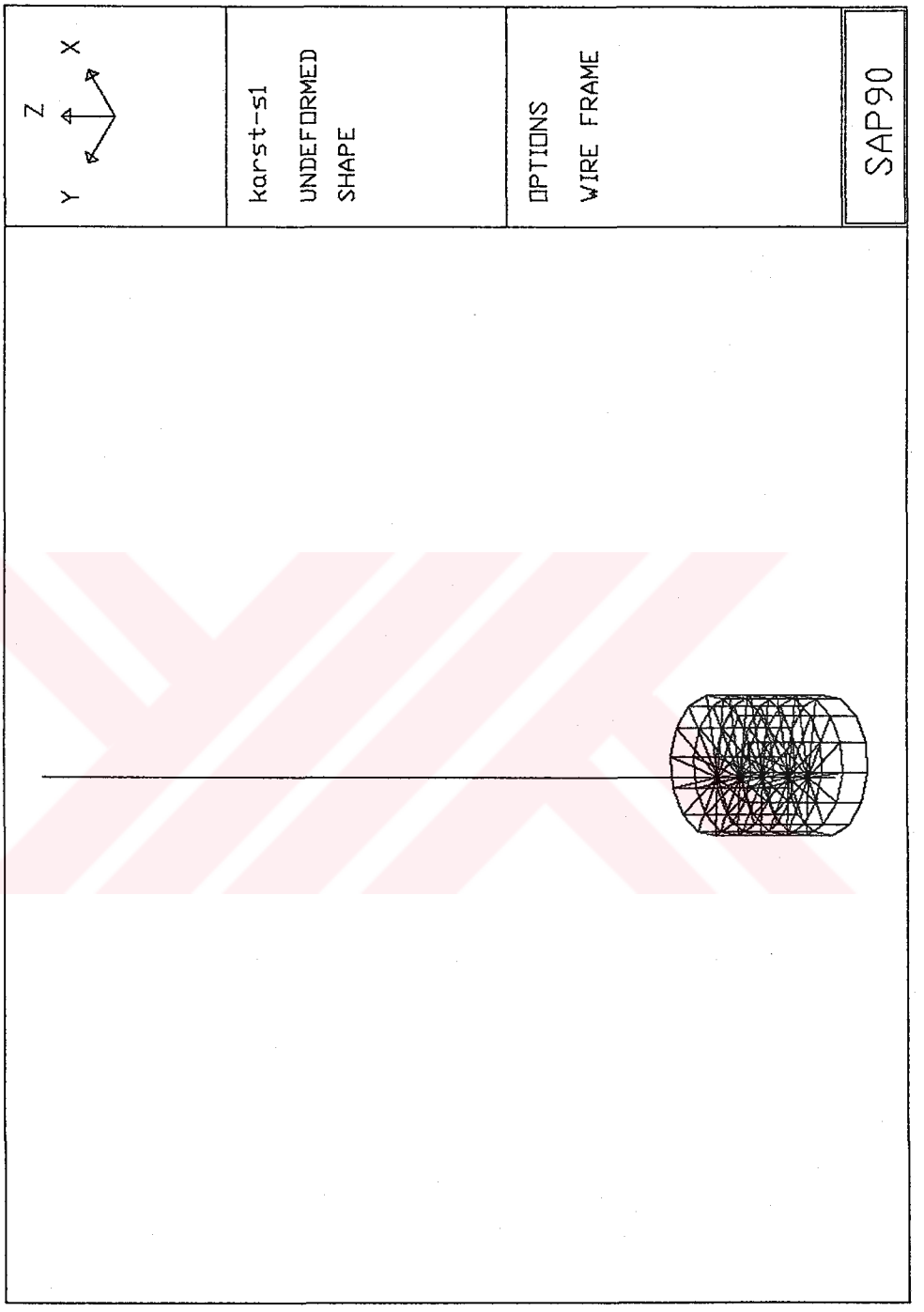

Figure 4.3 Model of the Structure : General View

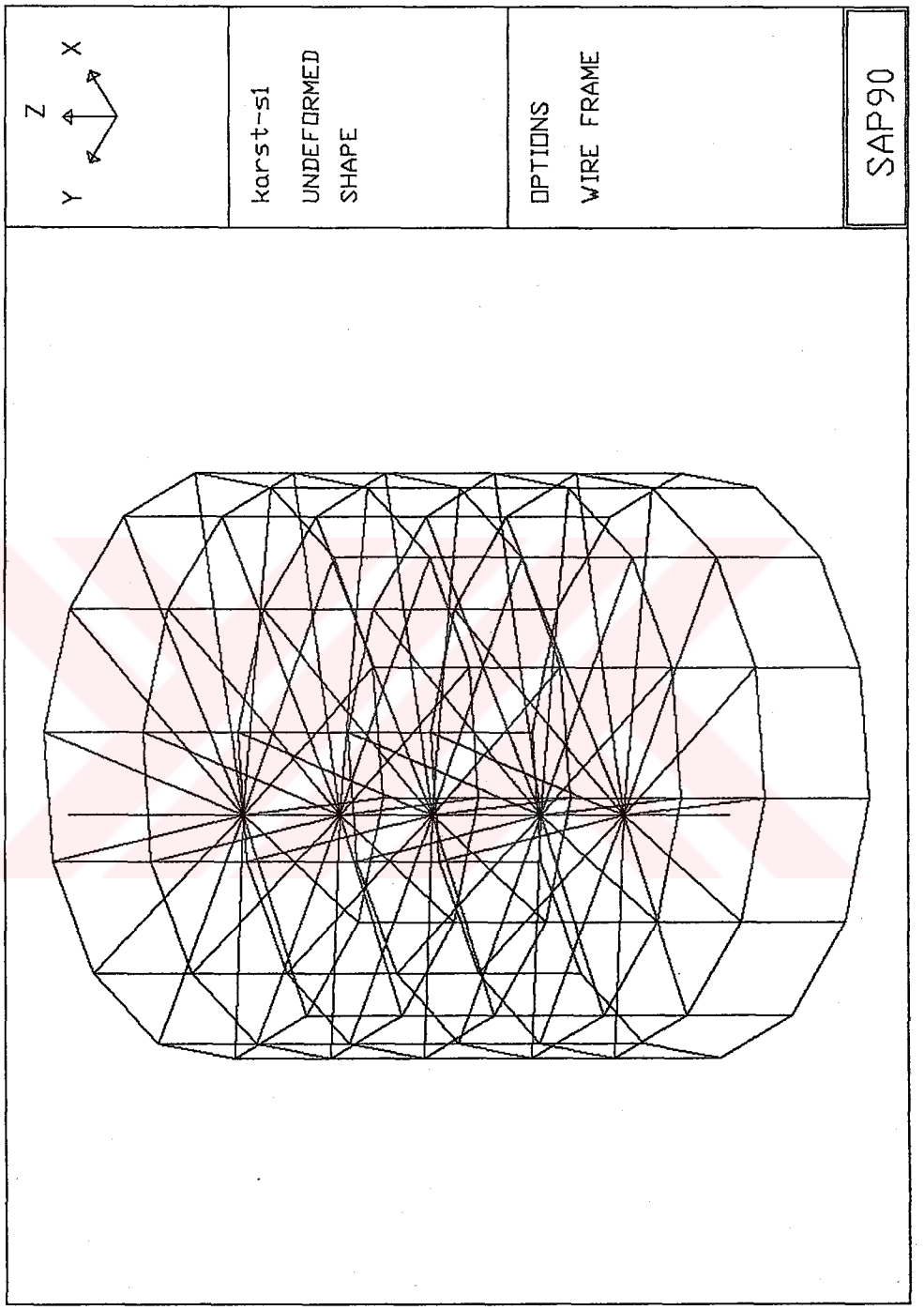

Figure 4.4 General View of the Model of the Building Part

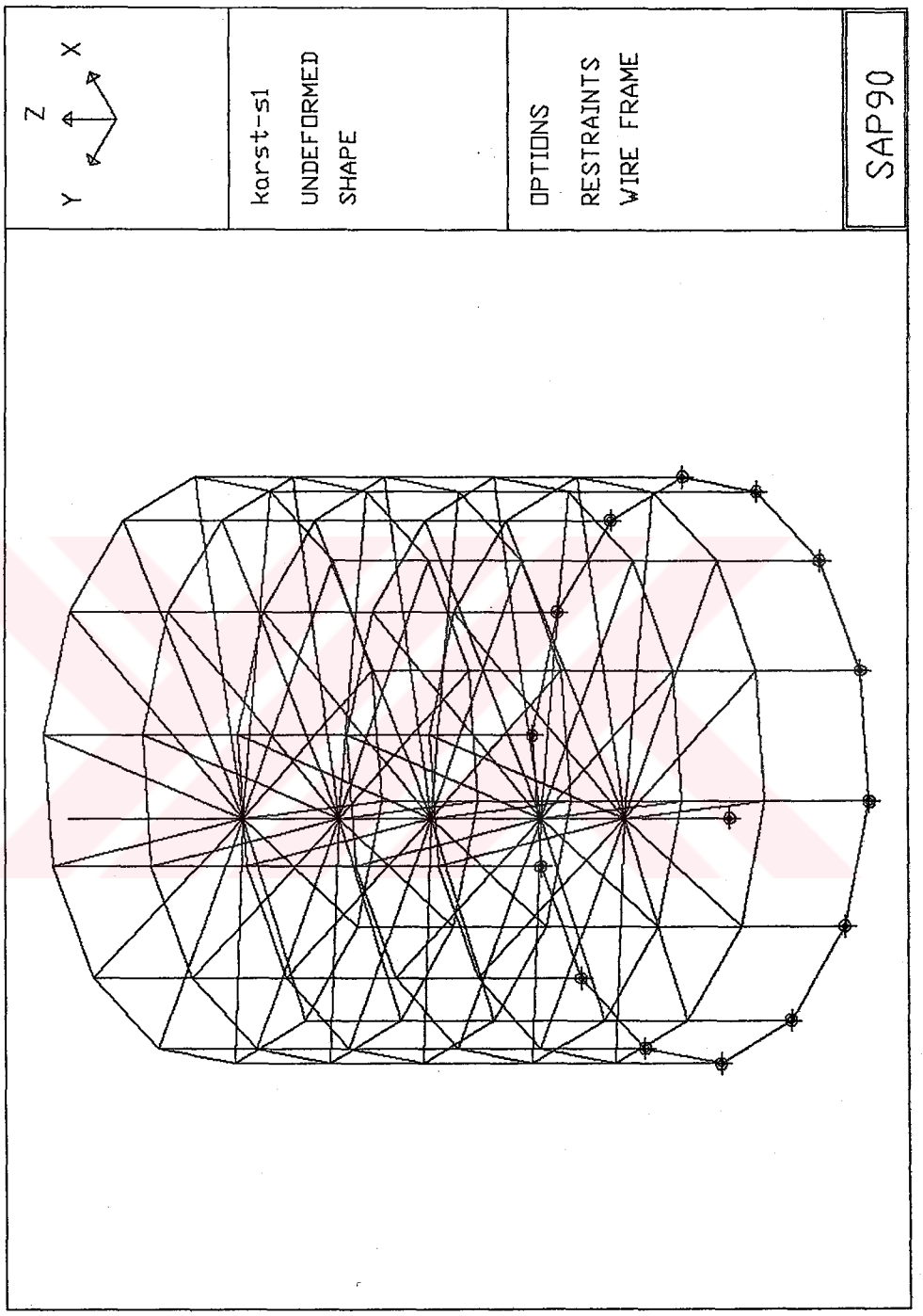

Figure 4.5 Model of the Building Part Showing Foundation Fix Restraints

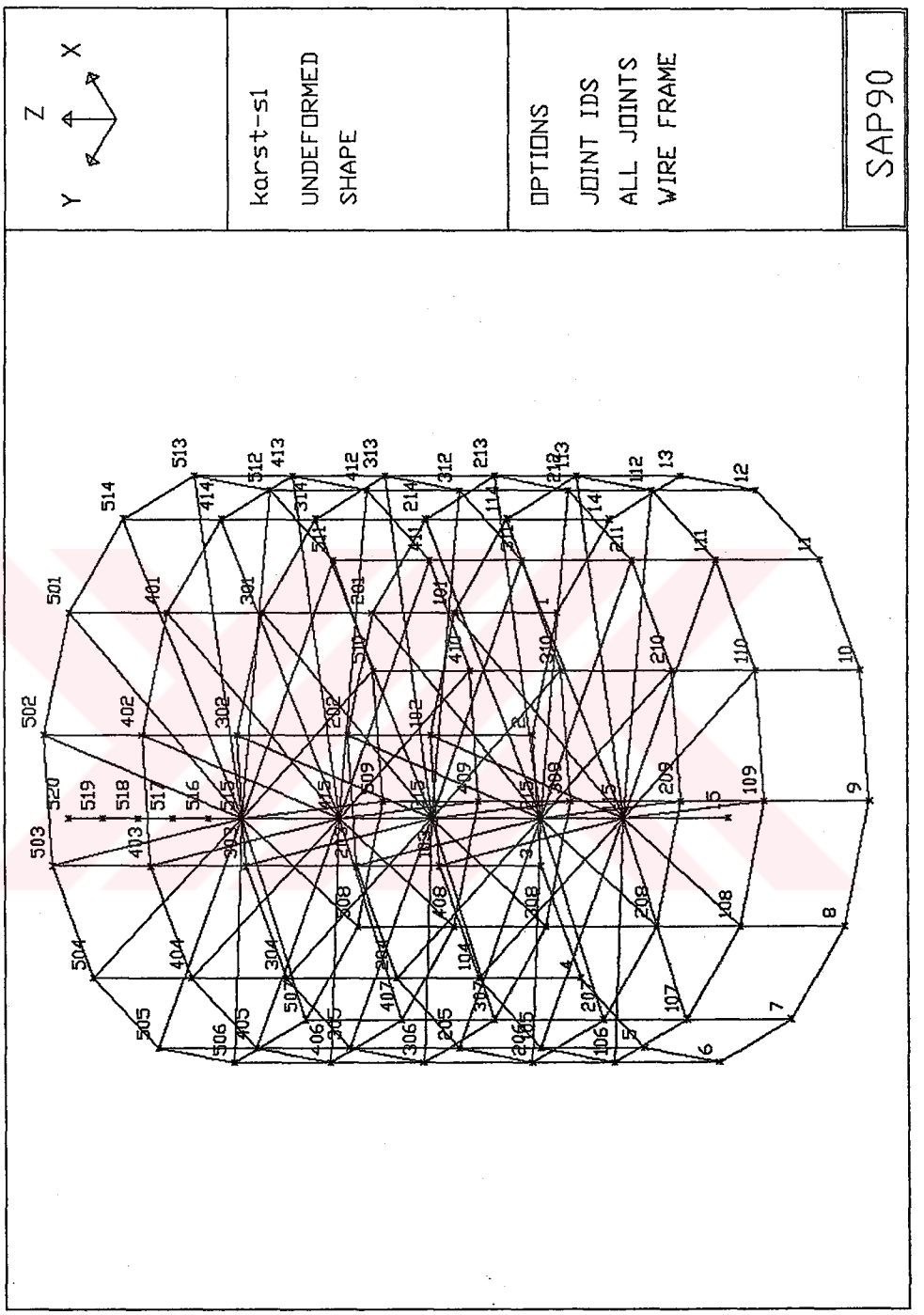

Figure 4.6 Model of the Building Part Showing Nodal Point Numbering

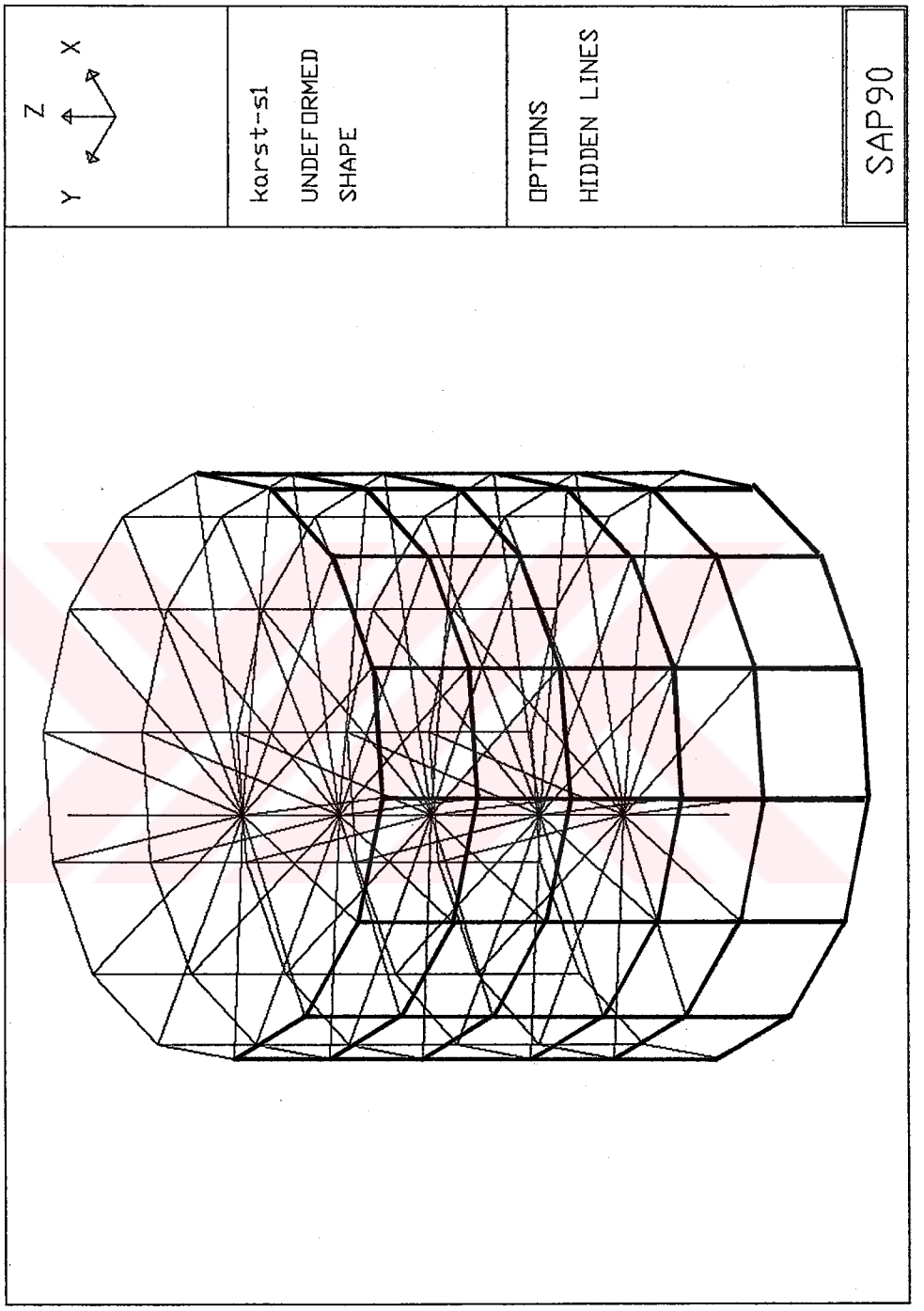

Figure 4.7 Model of the Building Part Showing Shear Walls

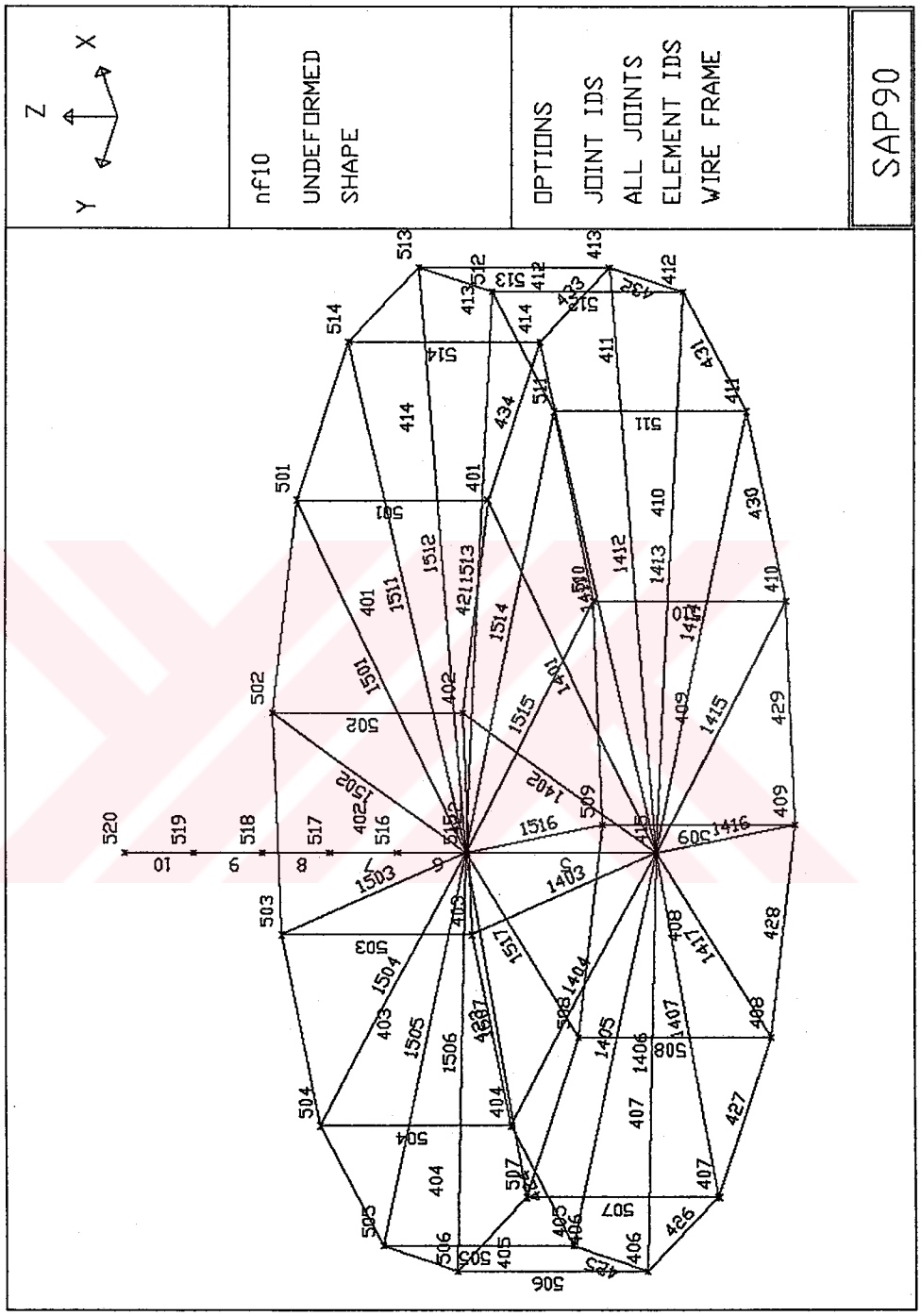

Figure 4.8 Model of the Structure between +10.00m ~ +20.00m Elevations

Showing Node and Element Numbering

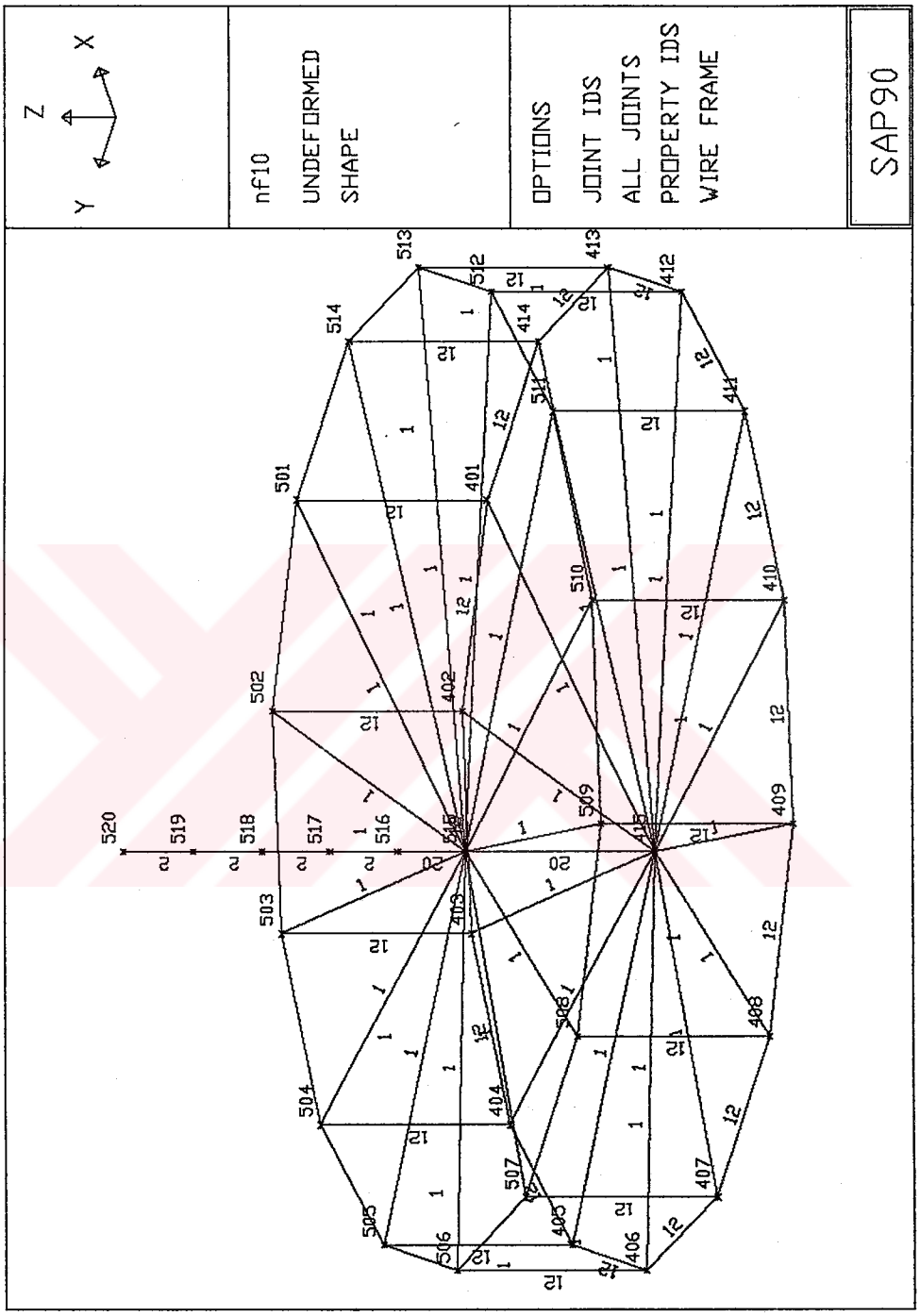

Figure 4.9 Model of the Structure between +10.00m ~ +20.00m Elevations

Showing Node and Element Section Property Numbering

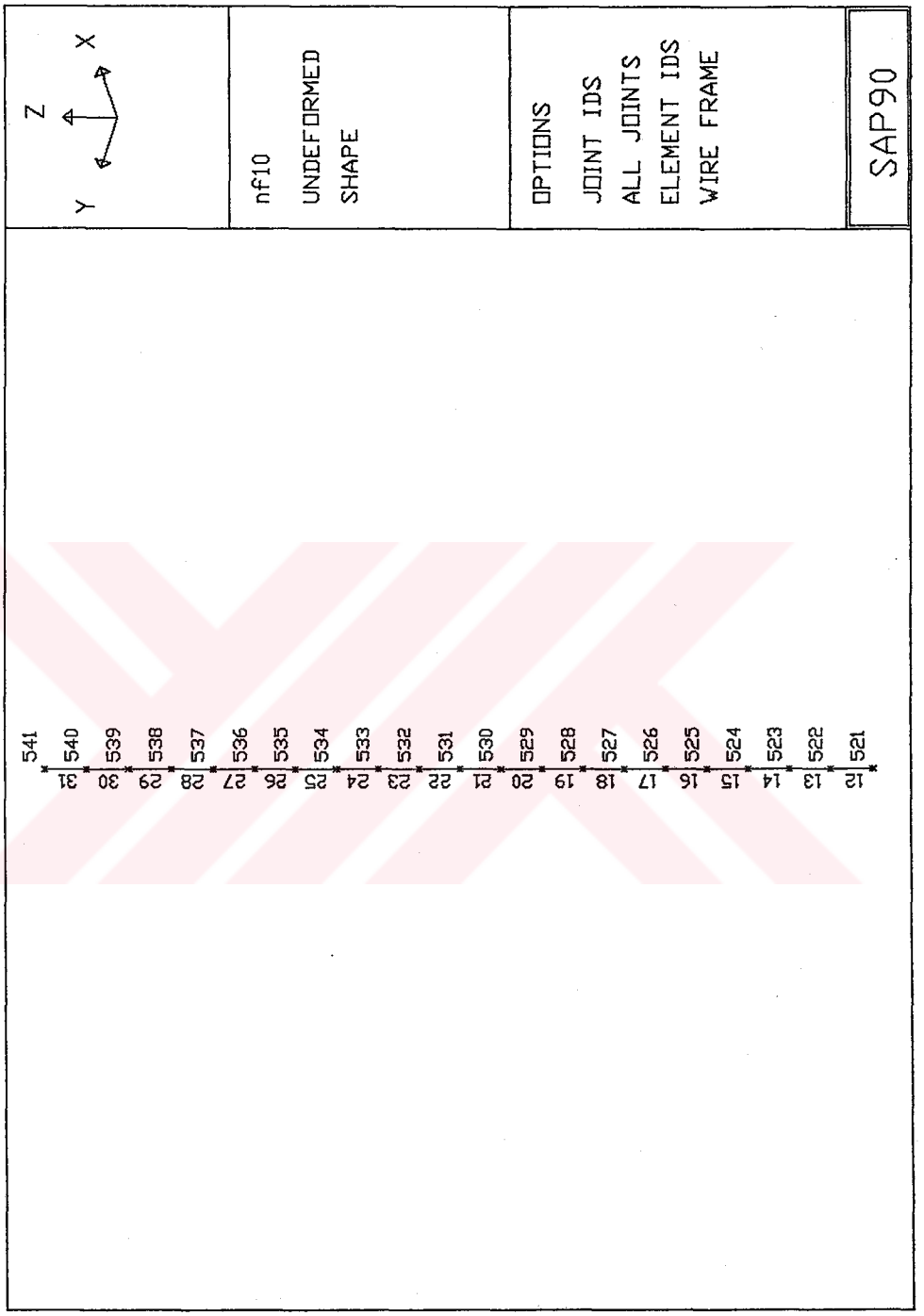

Figure 4.10 Model of the Structure between +20.00m ~ +40.00m Elevations

Showing Node and Element Numbering

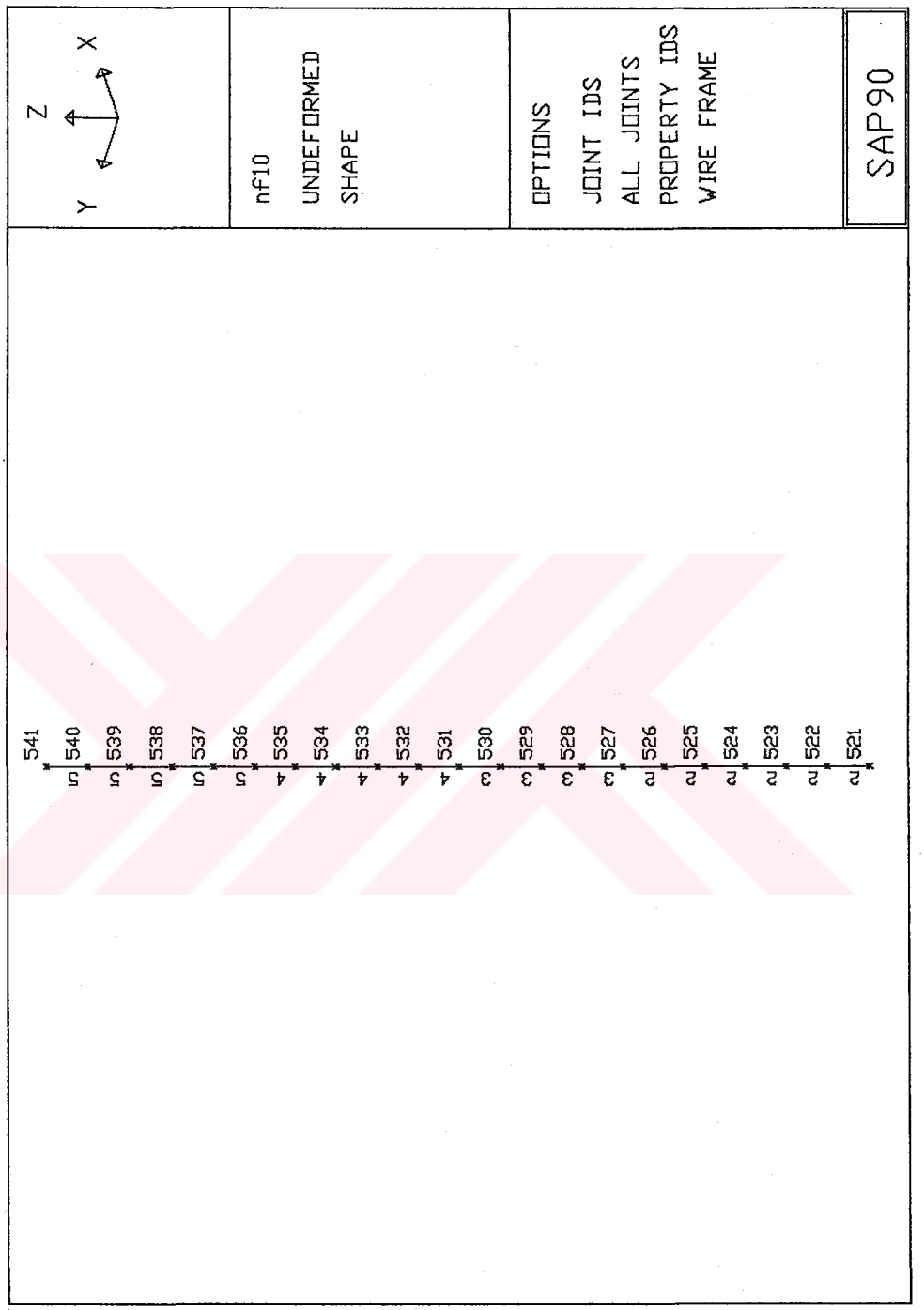

Figure 4.11 Model of the Structure between +20.00m ~ +40.00m Elevations

Showing Node and Element Section Property Numbering

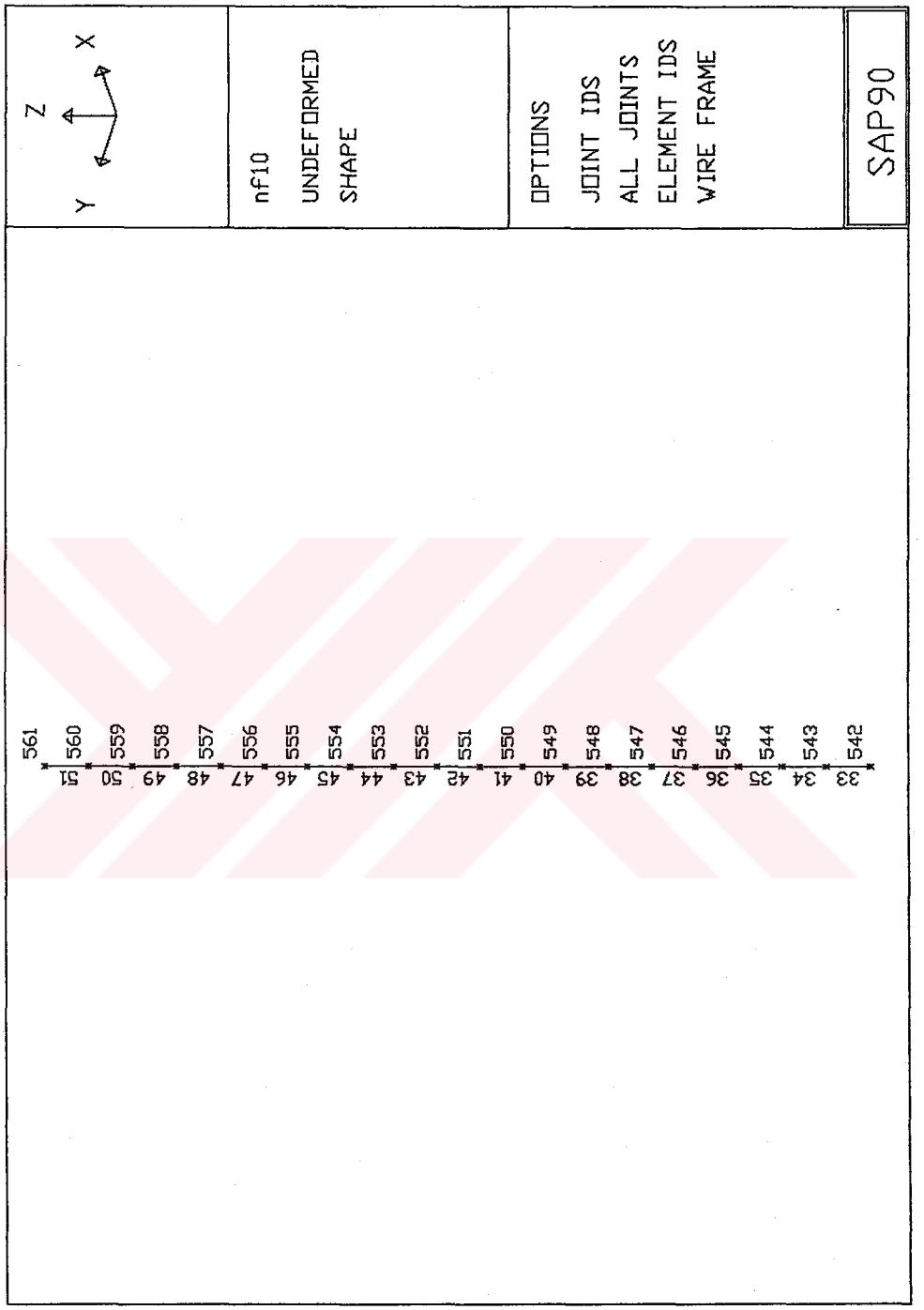

Figure 4.12 Model of the Structure between +40.00m ~ +60.00m Elevations

Showing Node and Element Numbering

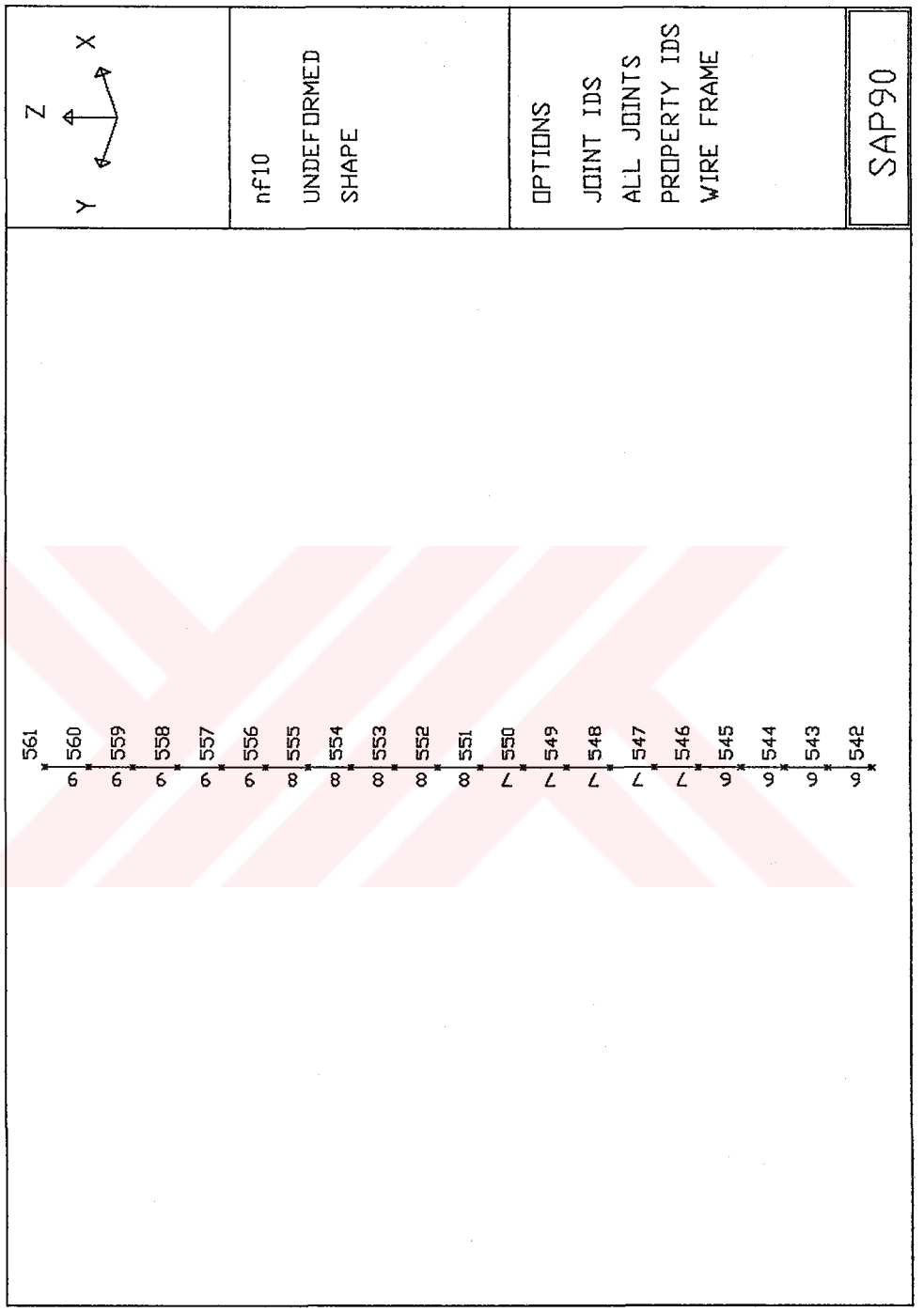

Figure 4.13 Model of the Structure between +40.00m ~ +60.00m Elevations

Showing Node and Element Section Property Numbering

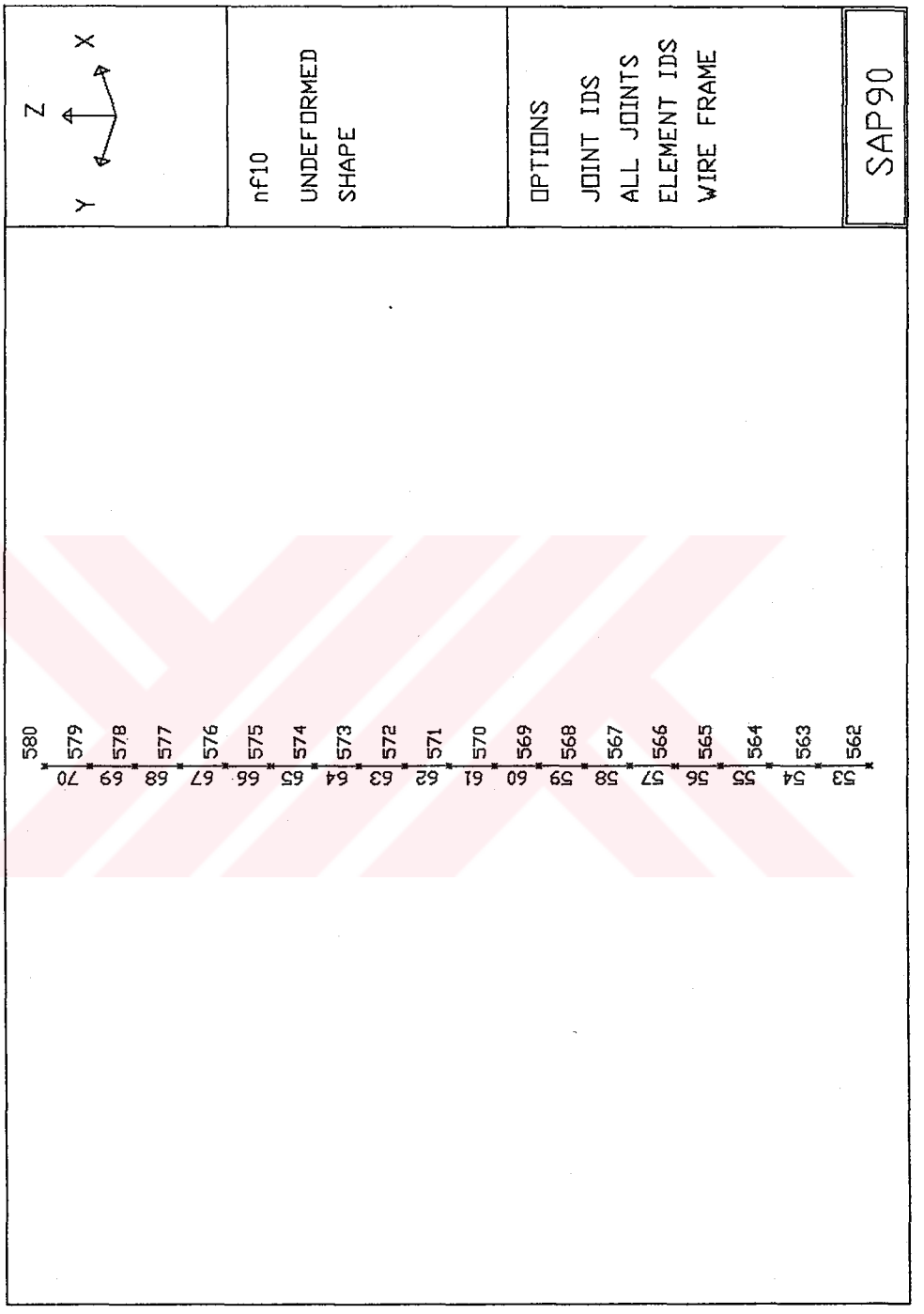

Figure 4.14 Model of the Structure between +60.00m ~ +80.00m Elevations

Showing Node and Element Numbering

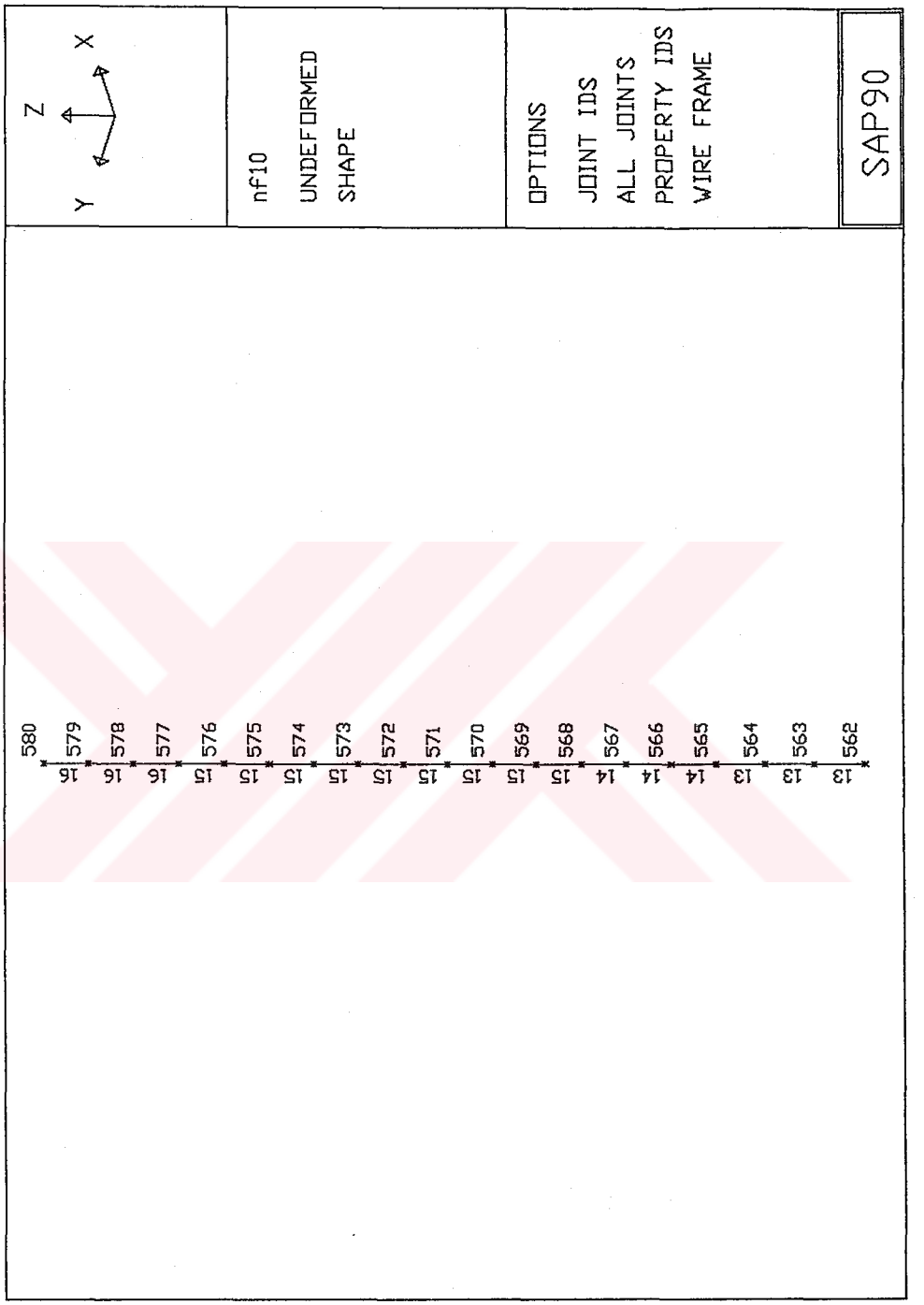

Figure 4.15 Model of the Structure between +60.00m ~ +80.00m Elevations

Showing Node and Element Section Property Numbering

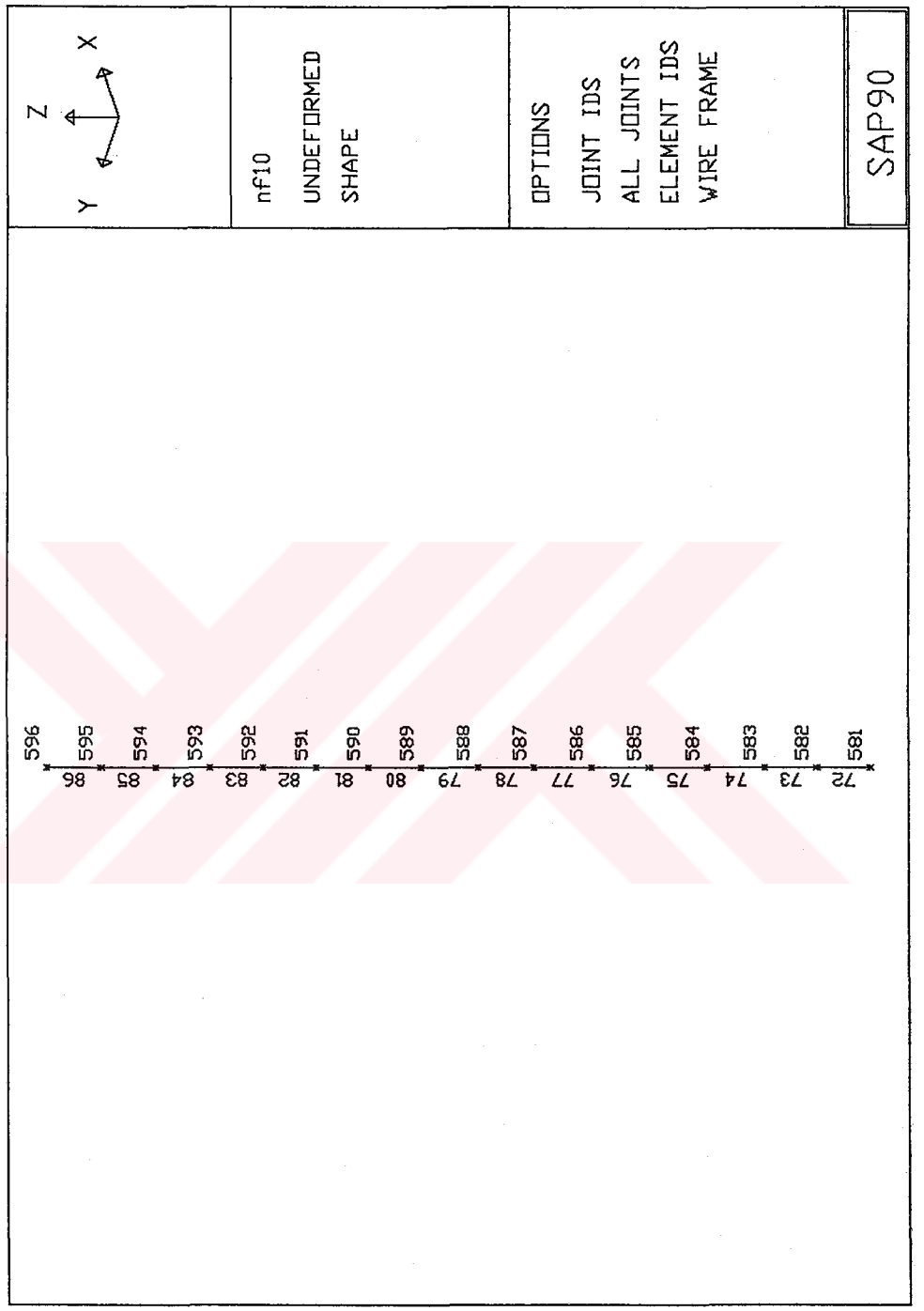

Figure 4.16 Model of the Structure between +80.00m ~ +96.00m Elevations

Showing Node and Element Numbering

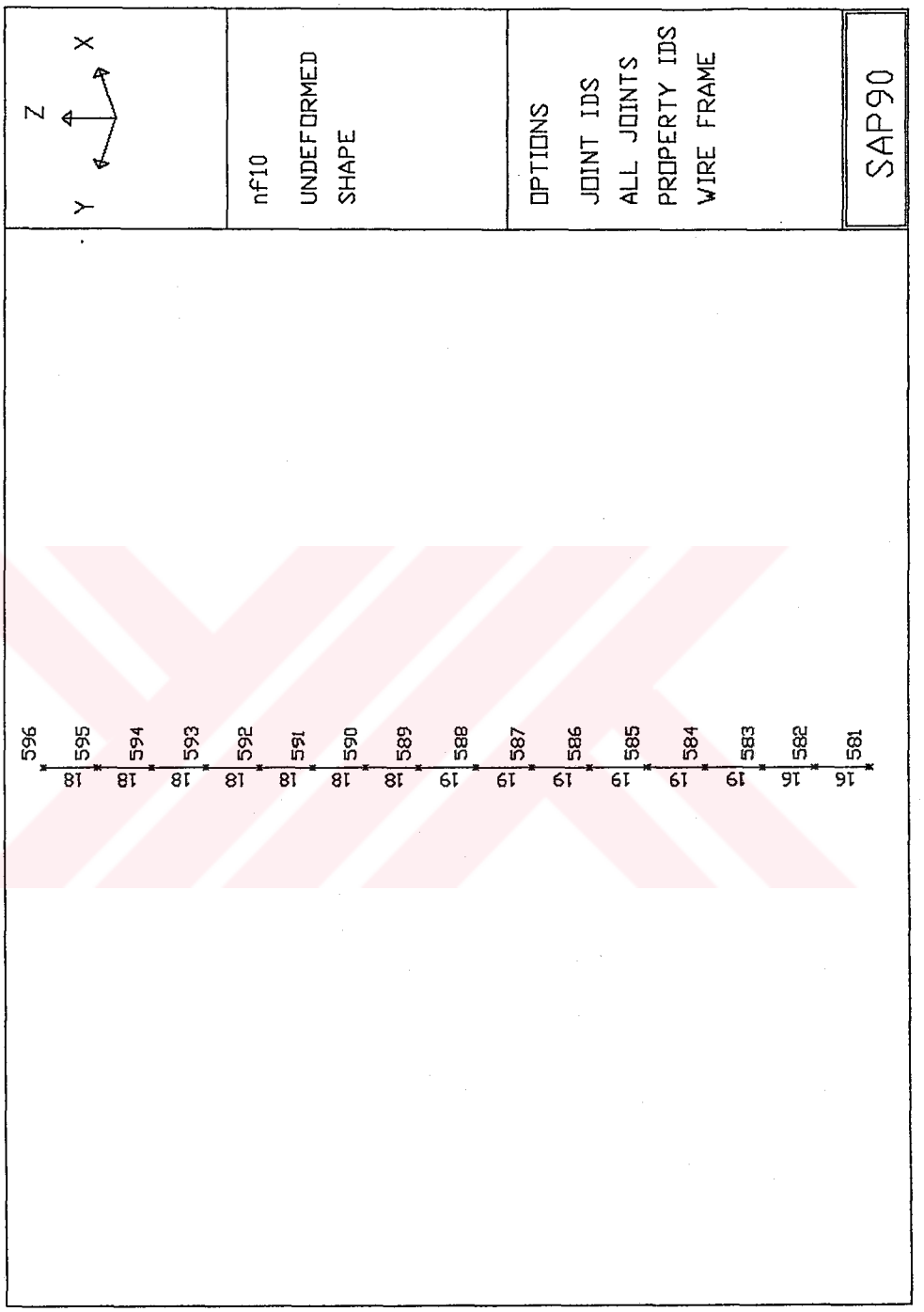

Figure 4.17 Model of the Structure between +80.00m ~ +96.00m Elevations

Showing Node and Element Section Property Numbering

4.2. Material and Element Properties

Model of the structure is defined as

a. building section by skeleton and shell elements at every span

b. tower section by one bar element between every story till +14.100m elevation and after +14.100m elevation by 1m long bar elements.

For different materials, 10 reinforced concrete, 1 steel and 1 GRP section properties are defined.

Reinforced Concrete:

Modiolus of Elasticity $=2.65 \times 10^6$ t/m^2 (TS500)

Weight per unit volume = 2.500 t/m^3 (TS500)

Structural Steel:

Modiolus of Elasticity = 2.1×10^7 t/m^2

Weight per unit volume = 7.850 t/m^3

Glass Reinforced Plastic:

Modiolus of Elasticity = 0.9×10^5 t/m^2 (ADC)

Weight per unit volume = 0.200 t/m (estimated from weight)

Section #1 is used to define reinforced concrete floor beams.

b= 30 cm , h = 50 cm

$A = 0.30 \times 0.50$

$= 0.150\ m^2$

$I_x = {}^{1}/_{12}\ 0.30 \times 0.50^3$

$= 0.003\ m^4$

$I_y = {}^{1}/_{12}\ 0.50 \times 0.30^3$

$= 0.001\ m^4$

$M = \{(0.150 + 1.50 \times 0.15) \times 2.500 + 0.200\}/9.81$

$= 0.116$ t/m

Section #2 is used to define the tower structure reinforced concrete section between +0.000m and +26.00m elevations.

D = 4.50 , t = 25cm

$A = ({}^{3.14}/_{4})\ (4.50^2 - 4.00^2)$

$= 3.336\ m^2$

$I = {}^{3.14}/_{64}\ (4.50^4 - 4.00^4)$

$= 7.559\ m^4$

$M = (3.336 \times 2.500) / 9.81$

$= 0.850$ t/m

Section #3 is used to define the reinforced concrete tower passing zone section between +26.00m and +30.00m elevations.

D = 4.50 , t = 150cm

$$A = (^{3.14}/_{4})\ (4.50^2 - 1.50^2)$$

$$= 14.130\ m^2$$

$$I = {}^{3.14}/_{64}\ (4.50^4 - 1.50^4)$$

$$= 19.870\ m^4$$

$$M = (14.130 \times 2.500) / 9.81$$

$$= 3.432\ t/m$$

Section #4 is used to define reinforced concrete tower section at +30.00m elevation.

D = 2.40 , t = 45cm

$$A = (^{3.14}/_{4})\ (2.40^2 - 1.50^2)$$

$$= 2.755\ m^2$$

$$I = {}^{3.14}/_{64}\ (2.40^4 - 1.50^4)$$

$$= 1.379\ m^4$$

$$M = (2.755 \times 2.500) / 9.81$$

$$= 0.702\ t/m$$

Section #9 is used to define reinforced concrete tower section at +60.90m elevation.

D = 2.40 , t = 30cm

$$A = (^{3.14}/_{4})\ (2.40^2 - 1.80^2)$$

$$= 1.978\ m^2$$

$$I = {}^{3.14}/_{64}\ (2.40^4 - 1.80^4)$$

$$= 1.113\ m^4$$

$$M = (1.978 \times 2.500) / 9.81$$

$$= 0.504\ t/m$$

(Section properties between #4 and #9 are defined by interpolating section properties #4 and #9. They are used for reinforced concrete sections between +30.00m and +60.90m elevations.)

Section #10 is used to define structural steel section at +60.90m elevation.

D = 1.20 , t = 18mm

$$A = (^{3.14}/_{4})\ (1.20^2 - 1.164^2)$$

$$= 0.067\ m^2$$

$$I = {}^{3.14}/_{64}\ (1.20^4 - 1.164^4)$$

$$= 0.187\ m^4$$

$$M = (0.067 \times 2.500 + 0.500) / 9.81$$

$= 0.105$ t/m

Section #11 is used to define Glass Reinforced Plastic tower appendages. (Section dimensions are taken from the report prepared by Alan Dick & Company Limited, Structural Analysis of Turkish Antenna GRP Cylinders)

D = 1.024m , d = 1.000m

$$A = (^{3.14}/_{4})\ (1.024^2 - 1.000^2)$$

$$= 0.038\ m^2$$

$$I = {}^{3.14}/_{64}\ (1.024^4 - 1.000^4)$$

$$= 0.00013\ m^4$$

$$M = 0.200 / 9.81$$

$$= 0.020\ t/m$$

(GRP antenna weight is assumed as 200 kg/m)

Section #12 is used to define dummy reinforced concrete beams and columns. bw = 20cm , d = 20cm

$$A = 0.20 \times 0.20$$

$$= 0.040\ m^2$$

$$I_x = I_y = {}^{1}/_{12}\ 0.20 \times 0.20^3$$

$$= 0.0001333\ m^4$$

Shell elements are used to define the reinforced concrete thin shear walls at outer sides of the building part of the structure.

t = 20 cm

$$M = (0.20 \times 2.500) / 9.81$$

$$= 0.051 \text{ t/m}^2$$

4.3. Previous Investigations

Measurement of structural response to the ambient excitations is one of the best methods utilized to assess the dynamic characteristics of such massive structures so the dynamic characteristics of the Kars TV Antenna Structure have been measured to compare the results with mathematical model.

The peak frequencies observed on the Fourier Amplitude spectra are shown to be directly related to the modal frequencies of vibration of the structure. The first four modal frequencies of vibration are determined to be 0.55, 1.28, 2.25 and 3.24 Hz. The first four mode shapes, based on the amplitude ratios indicated above, are illustrated in Figures 4.18, 4.19, 4.20, and 4.21 where the lines indicate the modal shape in direction perpendicular to the internal stair case, and the dashed lines indicate the modal shape in direction parallel to internal stair case. Due to the specific mass and stiffness distribution of the tower structure the ratio

of the modal amplitudes at the 10th and 90th meter elevations of the tower for the first mode is in the vicinity of 100.

4.4. Evaluation of Results

The tower is described in the X-Y-Z space, z axis being the axis along the corresponding mode shapes have been obtained. For each mode, two results are obtained: first for the mode in the X-Z plane and the other in the Y-Z plane. Due to the fact that the structure has similar characteristics in both planes, the results are similar and there is no reason to present two similar modes. Therefore, only the odd modes are presented.

The natural frequencies and periods obtained from the finite element model and those obtained experimentally are given in Table 4.1. The corresponding mode shapes are given in Figs. 4.18, 4.19, 4.20 and 4.21. A study of Table 4.1 reveals that the experimentally obtained natural frequencies and those obtained from the finite element model are in very satisfactory agreement. Especially the results for the first and second modes are almost the same. The discrepancy between experimental and numerical results increases as the mode number increases but this is expected at higher modes.

The agreement between the experimental and numerical results assures the verification of the finite element model. Thus, having gained confidence in the finite element model, dynamic analysis of the tower can be applied in both time and frequency domain.

Table 4.1 Numerical and Experimental Natural Frequencies and Periods

Mode	Experimental		Finite Element Model	
	Freq.(Hz)	Period(s)	Freq.(Hz)	Period(s)
1	0.55	1.818	0.556	1.799
2	1.28	0.781	1.260	0.794
3	2.25	0.444	2.087	0.479
4	3.24	0.309	3.451	0.290

4.5 Time Domain Analysis

For the near field earthquake analysis, an earthquake at an epicentral distance of 15 km, having a Peak Ground Acceleration (PGA) level of 0.3g at the bedrock level and 0.38g at the soil surface corresponding to a magnitude of Ms=6.0 was considered. On the other hand, for the far field earthquake analysis, an earthquake at an epicentral distance of 90 km, having a PGA level of 0.05g at the bedrock level and 0.11g at the soil surface corresponding to a magnitude Ms=7.0 was taken into account.

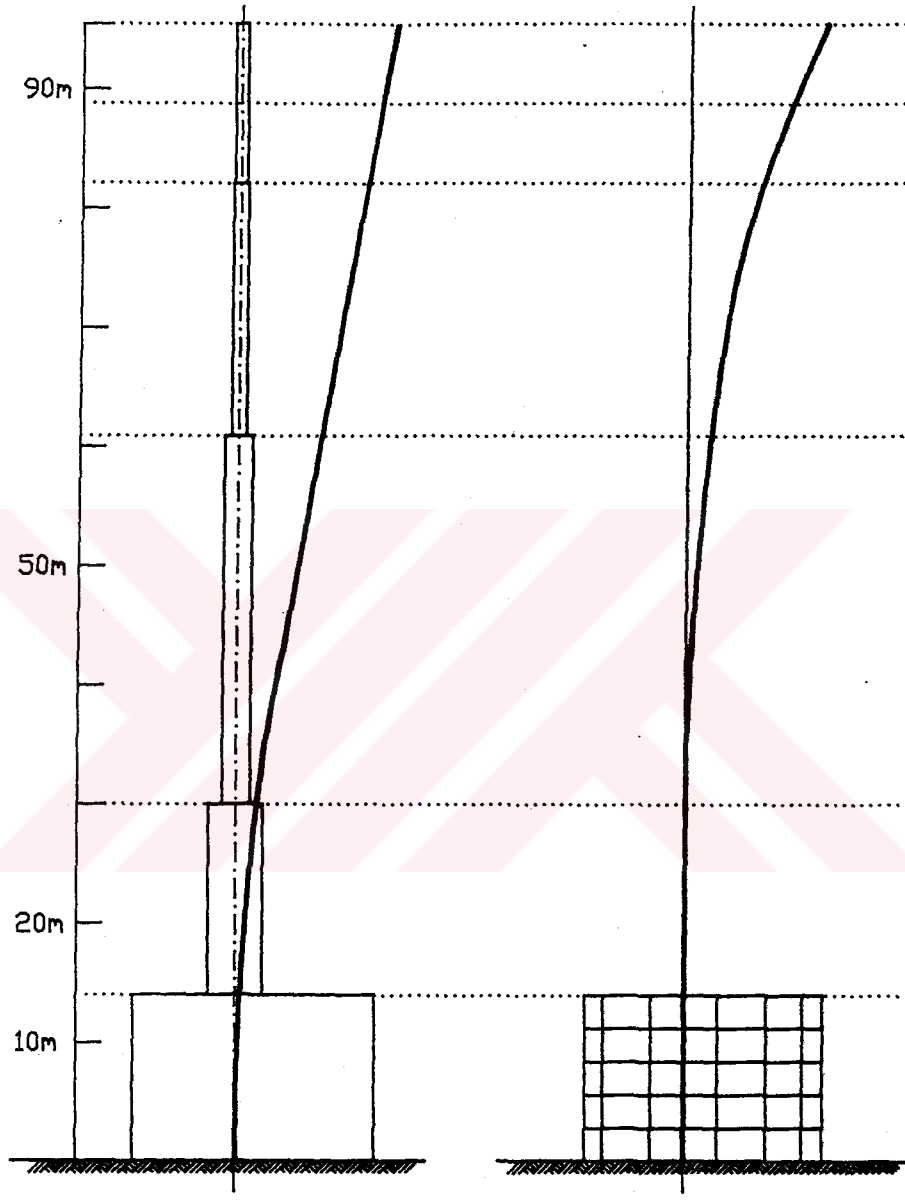

Figure 4.18 First Vibration Mode Shapes

Left : Ambient Vibration Test , F=0.55 Hz

Right : Dynamic Analysis by Computer , F=0.556 Hz

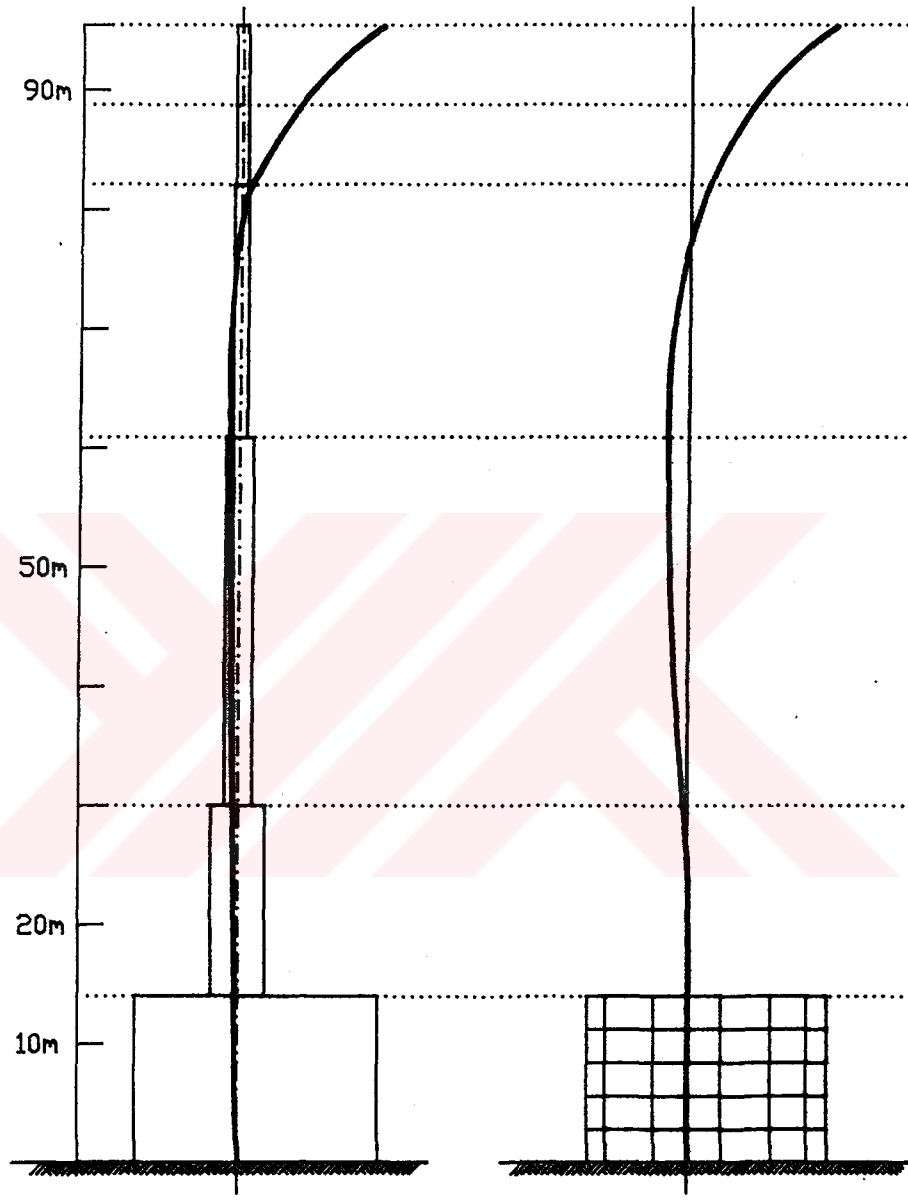

Figure 4.19 Second Vibration Mode Shapes

Left : Ambient Vibration Test , F=1.28 Hz

Right : Dynamic Analysis by Computer , F=1.260 Hz

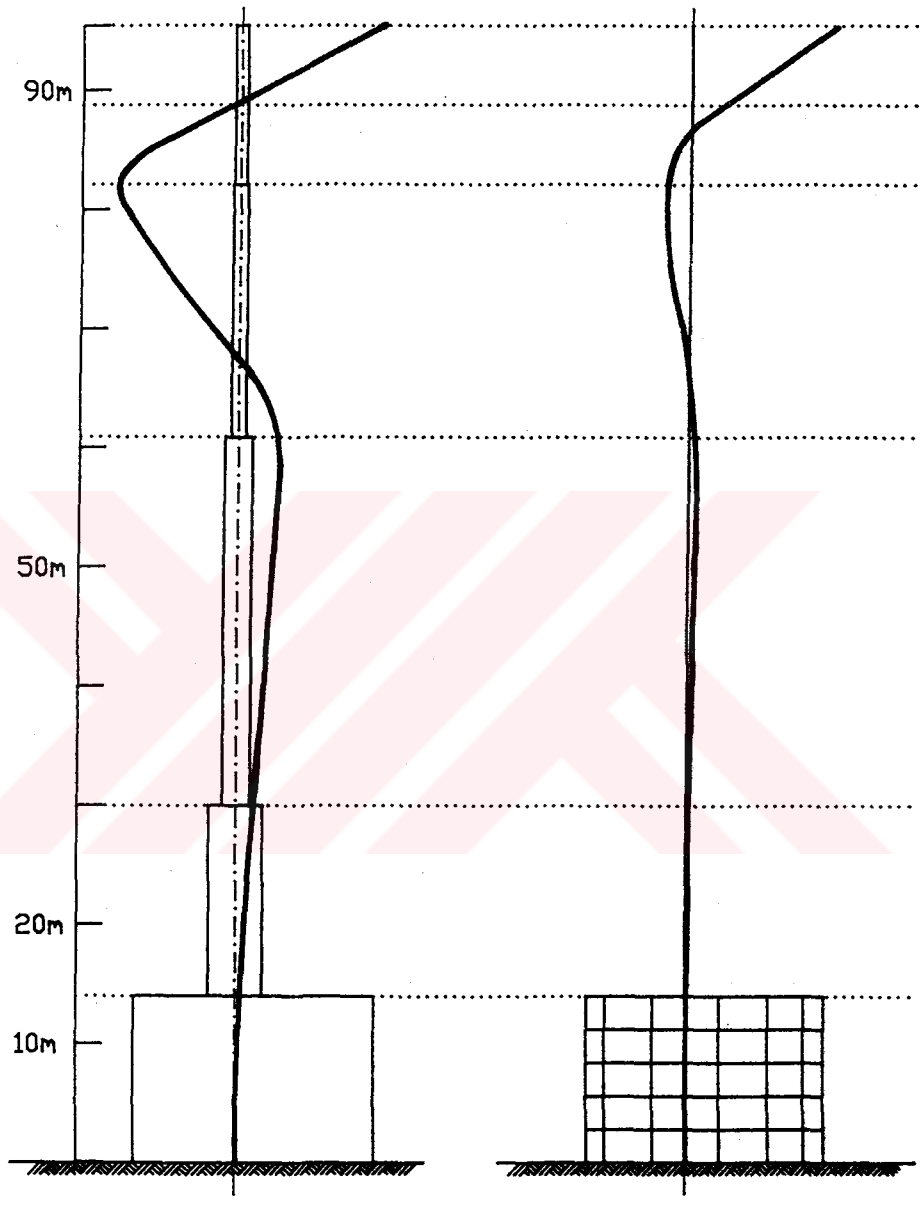

Figure 4.20 Third Vibration Mode Shapes

Left : Ambient Vibration Test , F=2.25 Hz

Right : Dynamic Analysis by Computer , F=2.087 Hz

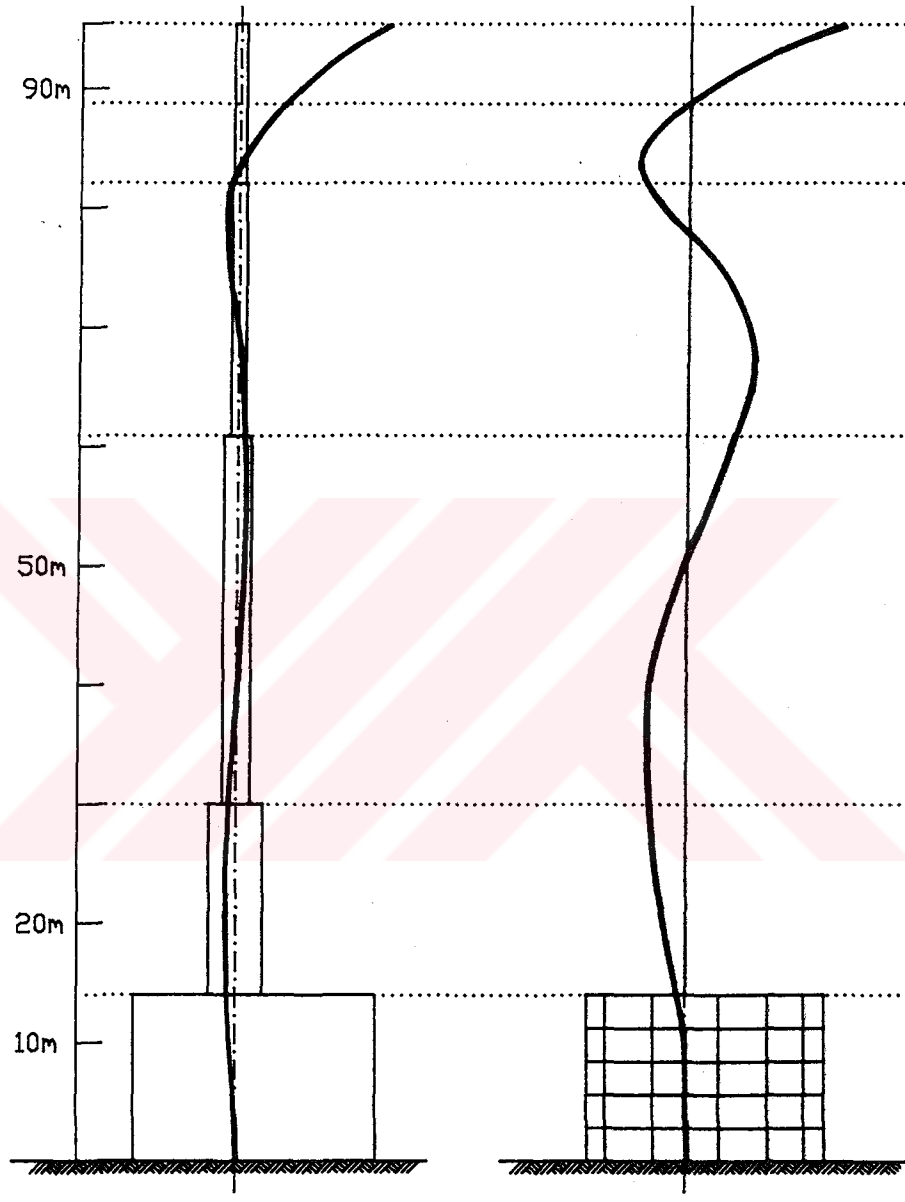

Figure 4.21 Fourth Vibration Mode Shapes

Left : Ambient Vibration Test , F=3.24 Hz

Right : Dynamic Analysis by Computer , F=3.451 Hz

As the observed damage on the structure, it seems to be caused by Spitak (Armenian) earthquake, which is a far field earthquake, only far field earthquake case is observed for time domain analysis.

The acceleration function acting at the base of the tower is given in Figure 4.22. As given before, the maximum acceleration is 0.11g as seen in the figure. The same acceleration function, but now with m/sec^2 units, is shown in Figure 4.23.

There different structural damping values are used in the numerical analysis. They are 0.01, 0.02 and 0.05. It should be noted that low strain levels in the tower are expected to generate low damping. Far field earthquakes, especially the one considered in the present work, are expected to cause low strain levels. A damping value of 0.01 encompasses low strain levels and, at the other end, a damping value of 0.05 is handled. The maximum acceleration, displacement, moment and shear force at the bottom of the GRP section of the tower are given in Table 5.2 for damping values of 0.01, 0.02 and 0.05.

Following figures show the acceleration, displacement, moment, shear force as function of time for the node and the element at the bottom of the GRP section of the tower, for the damping values of 1%, 2% and 5%, respectively for

the node.

Table 4.2 Maxima for Different Damping Values

Damping	1%	2%	5%
Max. Acceleration (m/s^2)	11.63	8.63	5.36
Max. Displacement (m)	0.129	0.104	0.0717
M_{max} (t.m)	62.45	51.53	32.05
V_{max} (t)	5.324	-	-

Damping value of 1% is more realistic for the present case since the far field earthquake under consideration is expected to cause low strain levels corresponding to the activation of low damping values. Figure 5.28, 5.29, and 5.30 gives the acceleration as a function of time at the midpoint (nodal point 590), top (nodal point 596) of the GRP section and displacement at the top of GRP section. Maximum acceleration is 18.54 m/s^2 at the midpoint, 65.80 m/s^2 at the top and maximum displacement is 0.485m at the top.

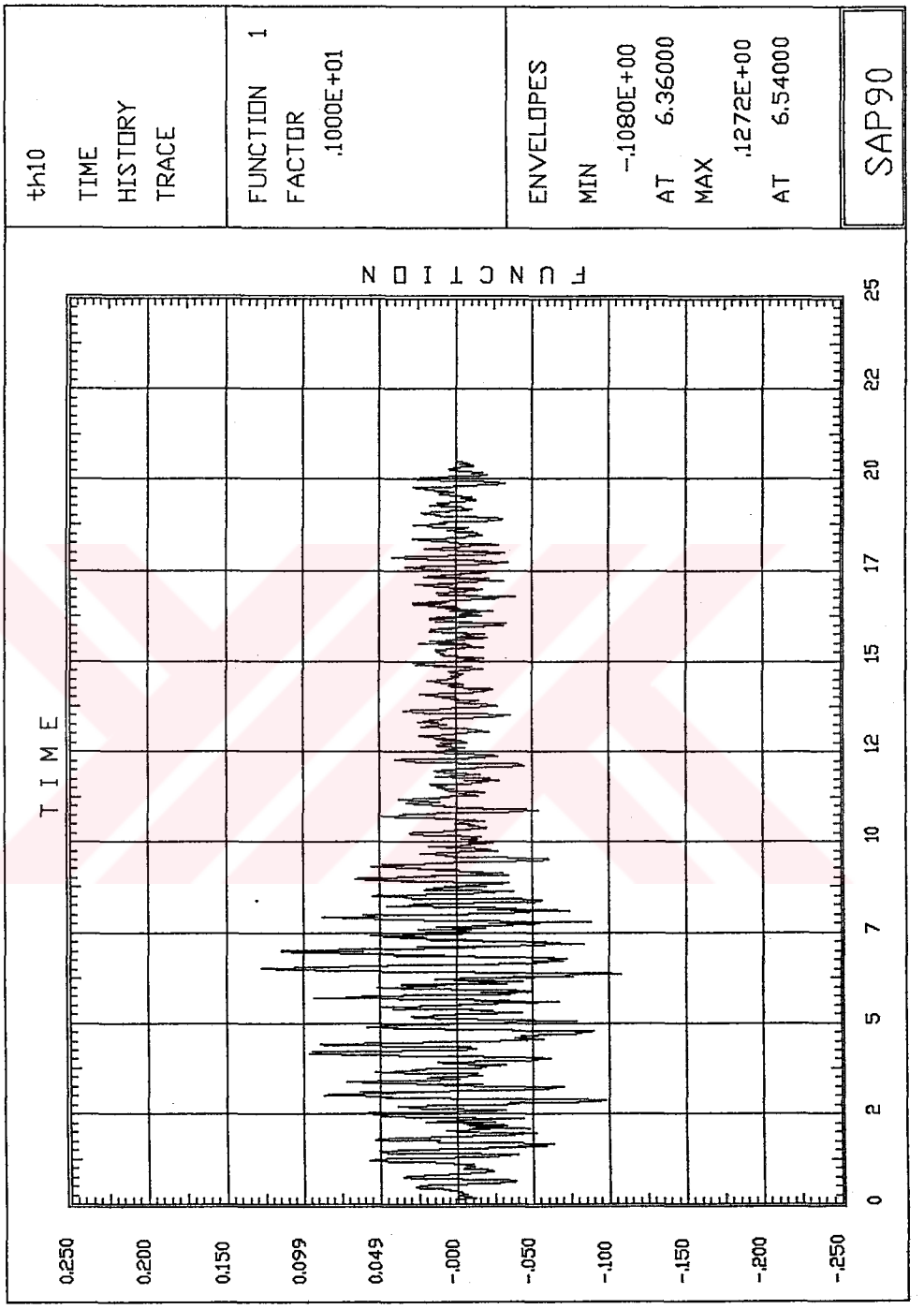

Figure 4.22 Time History Function of Computer Model versus Time

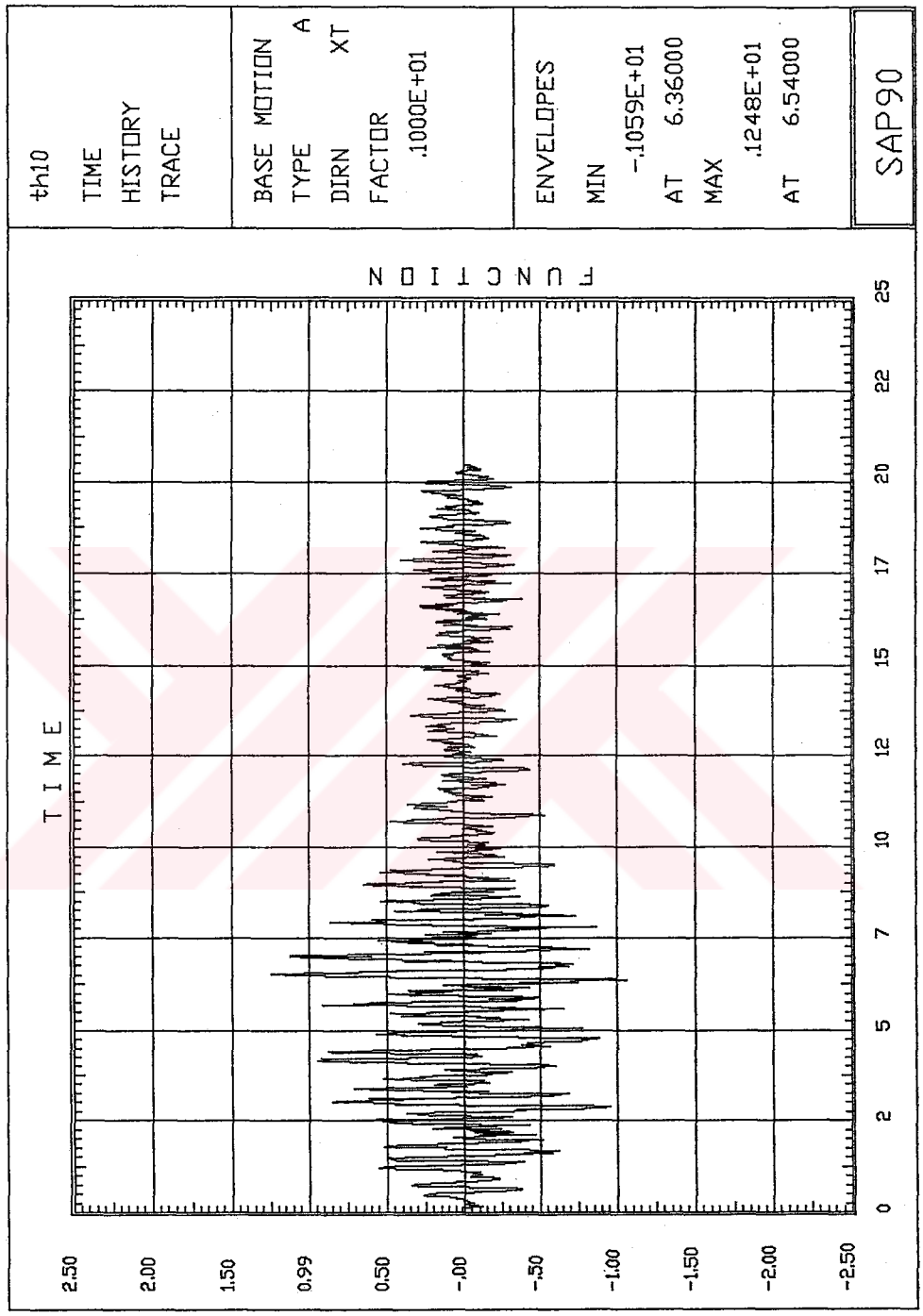

Figure 4.23 Base Acceleration at XT Direction versus Time

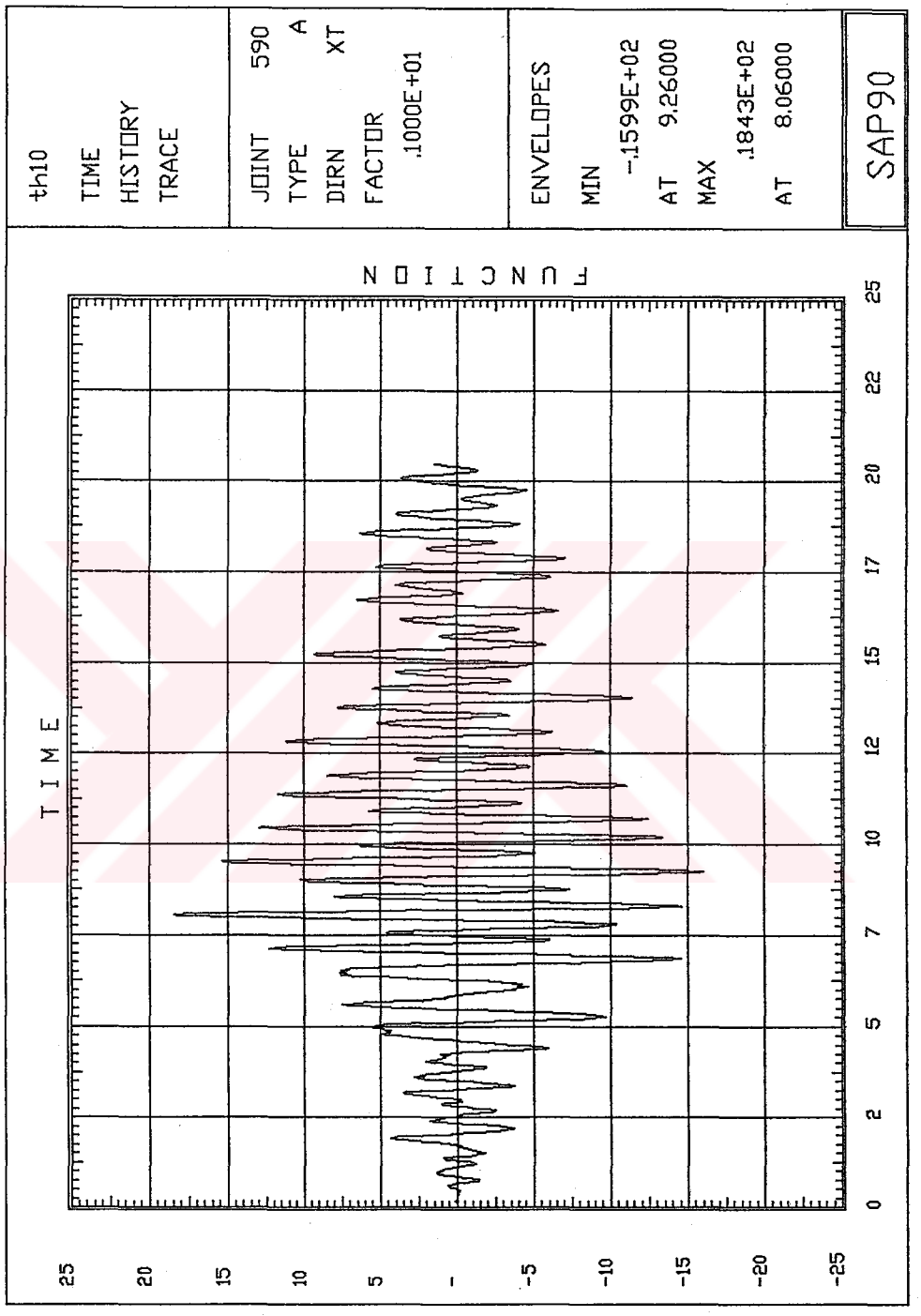

Figure 4.24 Acceleration of Node 590 at XT Direction versus Time

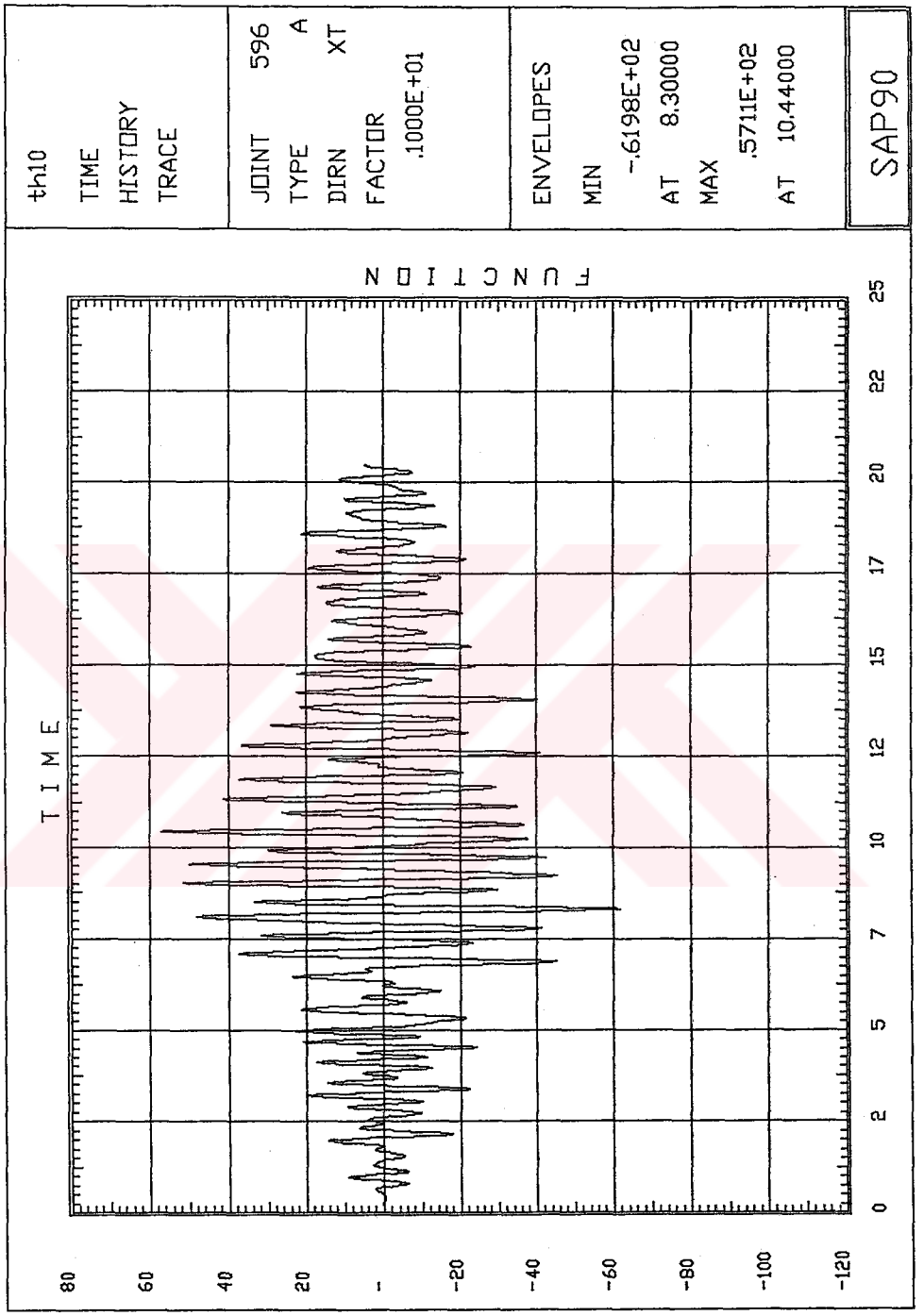

Figure 4.25 Acceleration of Node 596 at XT Direction versus Time

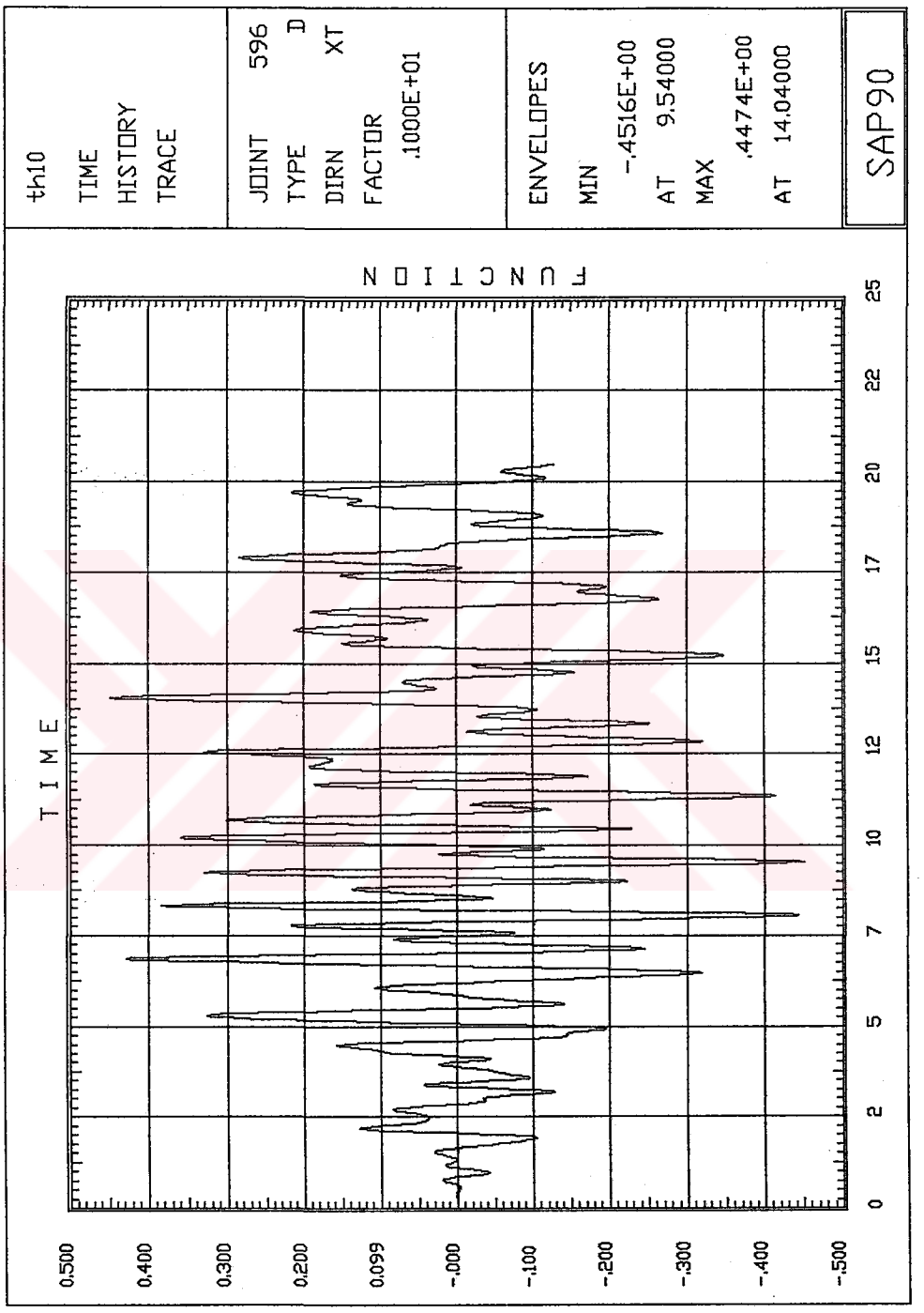

Figure 4.26 Displacement of Node 596 at XT Direction versus Time

The values obtained from the numerical analysis in the time domain should be verified with those obtained in the frequency domain. This requires response spectrum analysis which is presented in the following section.

4.6. Response Spectrum Analysis

For the near field earthquake analysis, an earthquake at an epicentral distance of 15 km, having a Peak Ground Acceleration (PGA) level of 0.3g at the bedrock level and 0.38g at the soil surface corresponding to a magnitude of 15km was considered. On the other hand, for the far field earthquake analysis, an earthquake at an epicentral distance of 90 km, having a PGA level of 0.05g at the bedrock level and 0.11g at the soil surface corresponding to a magnitude Ms=7.0 was considered.

In the response spectrum analysis, it is necessary to provide the deterministic design basis response spectra for both the near field (PGA=0.38g) and far field (PGA=0.11g) events. These spectra are obtained following the ATC 3-06 spectral shapes using a site factor of S=1 by another team. The ATC 3-06 type spectra corresponding to the near and far-field events are provided in following figures for 1%, 2%, and 5% damping respectively.

The empirical regression equations developed by Joyner and Boore can be utilized for the frequency domain simulation of the earthquake ground motion at the site. This regression analysis provides the spectral amplitudes with respect to earthquake magnitude, epicentral distance and the site conditions at selected frequencies. These empirical spectra, obtained for the near field and the far field events, have been compared with those provided in Figures 4.27 and 4.28 and it has been determined that ATC 3-06 based spectra provide prudent level of conservation to be utilized as the design basis response spectra.

The response spectra provided are for the horizontal component of the ground motion and for a relative viscous damping ratio of 5%. The vertical spectra for the same can be computed by multiplying the horizontal spectral ordinates by 2/3 following the USBRC and IAEA criteria. On the basis of the same criteria, the following factors can be used to multiply the spectral ordinates of the spectrum with 5% damping to obtain the spectrum for other damping factors:

Table 4.3 Spectral Adjustment Factors for Damping

Damping Ratio (%)	1	2	5	7	10	15	20
Factor	2.00	1.25	1.00	0.90	0.80	0.70	0.60

The response spectra obtained as described above are now used in

SAP90 to determine the deformed shape, moment distribution and shear distribution along the GRP section of the tower.

The maximum displacement occurs at the top of the GRP section. Maximum moment and shear are at element number 74 which is at the bottom of the GRP section.

Table 4.4 Maxima Obtained from the Response Spectrum Analysis

	Near field			Far field		
Damping	0.01	0.02	0.05	0.01	0.02	0.05
Max. Displacement (m)	1.34	0.89	0.70	0.47	0.31	0.24
M_{max} (t.m)	130.90	88.95	69.81	38.19	25.95	20.37
V_{max} (t)	13.01	8.79	6.91	3.81	2.57	2.02

4.7. Discussion

The maximum moment occurring at the bottom of the GRP section of the tower has been obtained by various approaches. The fact that either one of the near and far field earthquakes may be critical has been taken into consideration. The fact that the damping value may play a critical role in the design process has been taken into consideration. Both time domain and response spectrum analyses

have been carried out to verify the results. The maximum moments occurring at the bottom of the GRP section of the tower have been obtained for all possible combinations of the parameters mentioned and the results are summarized in Table 5.5.It is concluded that, for this structure for far field analysis, a damping value of 1% should be used. This corresponding to a maximum moment of 39.19 t.m obtained from the spectrum analysis. On the other hand, the structural design has been performed for the maximum moment considering wind loading only which was calculated to be 19.1t.m.

Table 4.5 Maximum Moment Occurring at the Bottom of the GRP Section

		Damping	M_{max}(t.m)
Near Field	Time Analysis	0.01	-
		0.02	-
		0.05	-
	Spectrum Analysis	0.01	130.90
		0.02	88.95
		0.05	69.85
Far Field	Time Analysis	0.01	62.45
		0.02	51.53
		0.05	32.05
	Spectrum Analysis	0.01	38.19
		0.02	25.95
		0.05	20.37

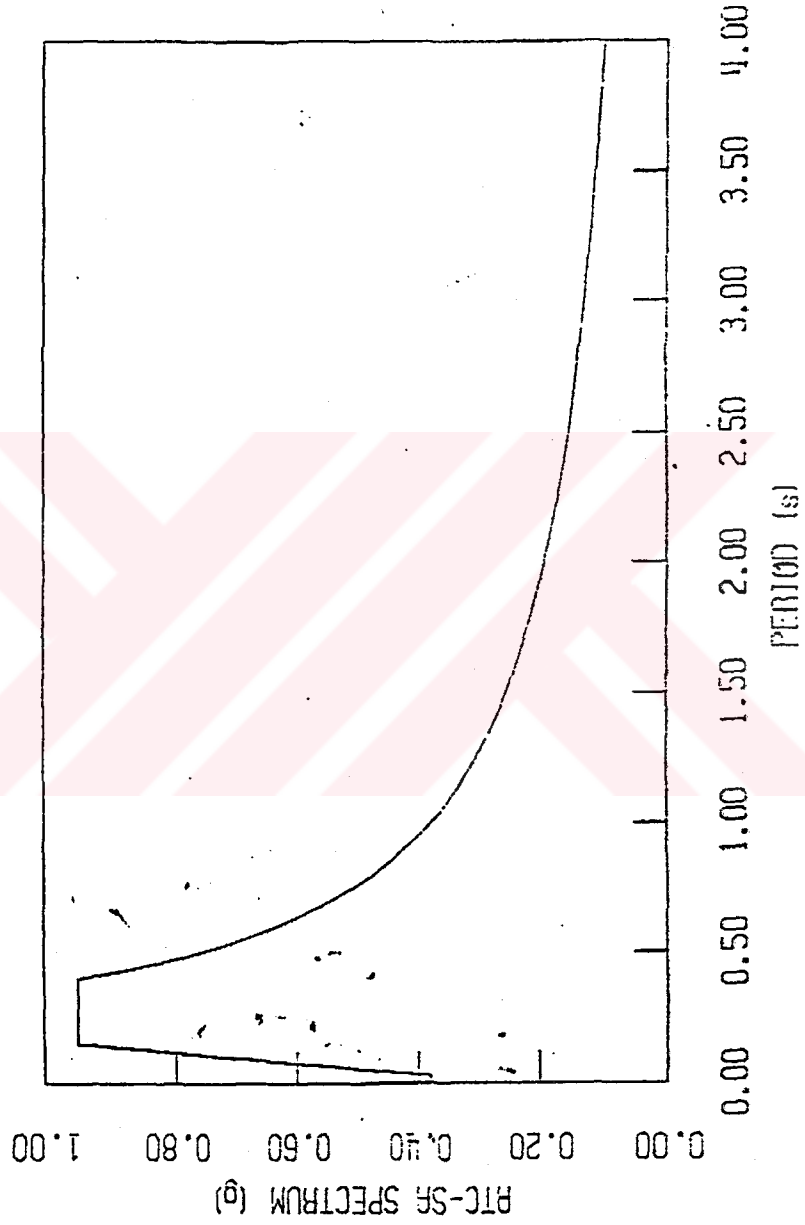

Figure 4.27 Near Field Design Response Spectrum, Damping 5%.

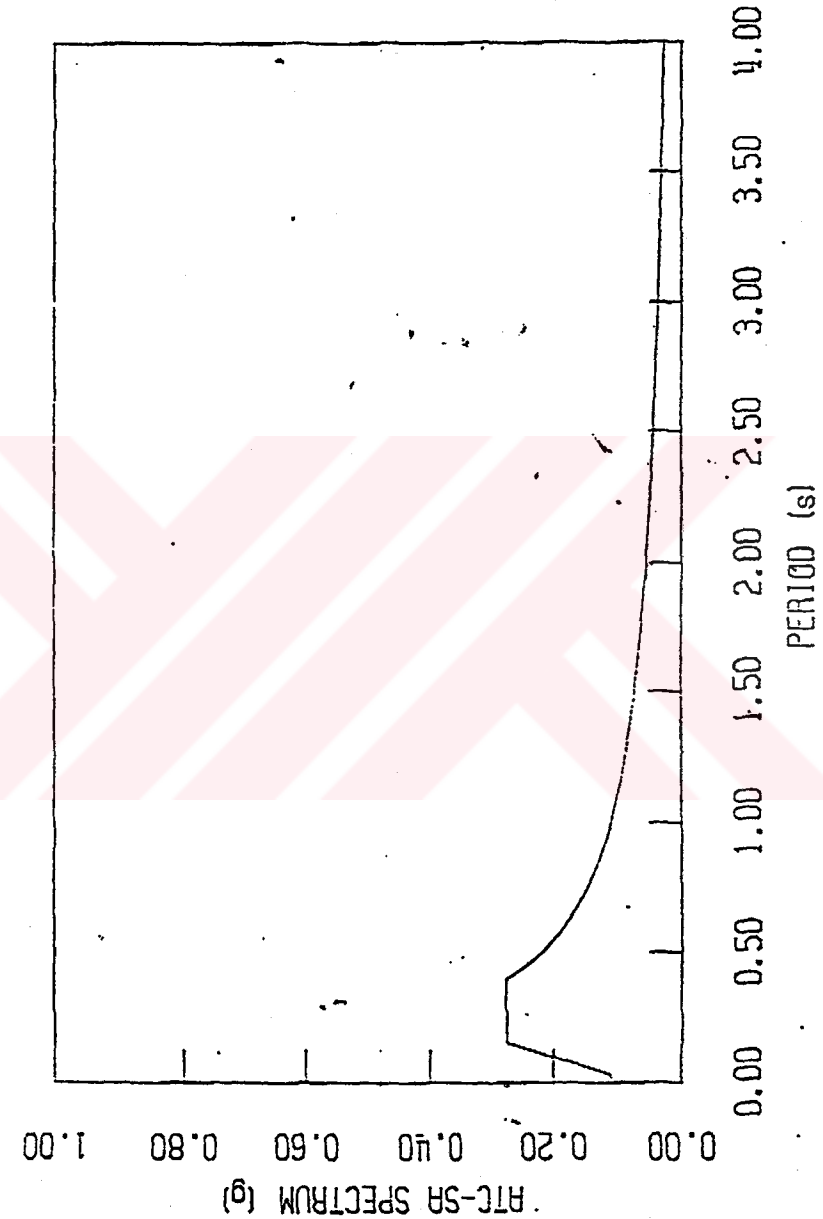

Figure 4.28 Far Field Design Response Spectrum, Damping 5%.

CHAPTER V

EQUIVALENT STATIC CALCULATIONS

5.1. General

It is a common practice in design codes to specify equivalent static lateral loading on various structures for estimating the earthquake response. Such an approach is considered useful for preliminary design because of its simplicity.

Although the exact nature of distribution of the equivalent lateral loads along the height of tower shaped structures is not yet known and can only be predicted by dynamic analysis, a careful static analysis may be enough for some structures.

There are different provisions for design of earthquake resistant structure using static loading and some of them is presented in this chapter. Aim of this presentation is to compare dynamic analysis with static equivalent calculations.

Table 5.1 Loads Along the Tower and Moments with respect to Elevations +0.00m, +14.10m, +30.00m, +60.90m and +82.10m

Height	M. wrt +0.00	M. wrt +14.10	M. wrt +30.00	M. wrt +60.90	M. wrt +82.10	Load
+0.00	0.00	-	-	-	-	
+3.00	1,428.48	-	-	-	-	476.16
+5.40	2,868.48	-	-	-	-	531.20
+8.40	4,230.91	-	-	-	-	503.68
+11.10	5,896.32	-	-	-	-	531.20
+14.10	948.37	0.00	-	-	-	67.26
+16.80	519.12	83.43	-	-	-	30.90
+21.50	870.75	299.70	-	-	-	40.50
+26.50	1,733.37	811.08	-	-	-	65.41
+30.00	1,898.70	1,006.31	0.00	-	-	63.29
+36.00	1,287.72	783.36	214.62	-	-	35.77
+42.00	1,348.20	895.59	385.20	-	-	32.10
+48.00	1,400.64	989.20	525.24	-	-	29.18
+54.00	1,522.80	1,125.18	676.80	-	-	28.20
+60.90	1,209.47	929.45	613.67	0.00	-	19.86
+64.20	147.66	115.23	78.66	7.59	-	2.30
+67.20	374.98	296.30	207.58	35.15	-	5.58
+76.20	528.07	430.35	320.17	106.03	-	6.93
+82.10	287.35	238.00	182.35	74.20	0.00	3.50
+88.85	133.28	112.13	88.28	41.93	10.13	1.50
+95.45	71.59	61.01	49.09	25.91	10.01	0.75
Total	28,706.26	8,176.32	3,341.66	290.81	20.14	2,475.27

For the static equivalent calculations of the tower, same masses as the dynamic analysis are used. Vertical loads and moments without seismic lateral load coefficient with respect to height along the tower are presented in Table 5.1

5.2 Design of Connections

It is advised that all parts of the building shall be interconnected and the connections shall be capable of transmitting the seismic force, Fp, induced by the parts being connected. As a minimum, any smaller portion of the building shall be tied to the remainder of the building with elements having at least a strength to resist Av/3 times the weight of the smaller portion where Av is the seismic coefficient representing the Effective Peak Velocity-Related Acceleration but not less than 5% of the portions weight[1].

Table 5.2 Coefficient Av and Seismicity Index

Map Area Number	Coefficient Av	Seismicity Index
7	0.40	4
6	0.30	4
5	0.20	4
4	0.15	3
3	0.10	2
2	0.05	2
1	0.05	1

As a minimum, a positive connection for resisting a horizontal force shall be provided for each appendage to its support which shall have a minimum strength acting along the span of the member equal to 5% of the dead and live load reaction.

Table 5.3 Minimum Load That Connections Should Resist

Height of Connection	Weight of Smaller Portion (t)	(Av/3) x Weight (t)
+30.00	197.32	13.15
+60.90	40.42	2.69
+82.10	5.75	0.38

The connections that holds the GRP antenna, should resist a 0.38t lateral load.

5.3 Equivalent Lateral Force Procedure

The general form of equivalent lateral force procedure is the Formula 5.1 for base shear, which gives the total seismic force F_b in terms of two factors, a seismic coefficient, C, and the total gravity load of the building, W.

$$F_b = C.W \qquad (5.1)$$

where

C = the seismic design coefficient

W = the total gravity load of the building

C can be calculated by different formulas according to different codes. Here, the ATC 3-06 and Turkish codes are presented.

5.3.1 ATC 3-06 For Earthquake Resistant Structures

When the period of the building is computed, the seismic coefficient C_o shall be determined in accordance with the following formula:

$$C_o = \frac{1.2A_vS}{RT^{\frac{2}{3}}} \qquad (5.2)$$

where

Av = the coefficient representing Effective Peak Velocity-Related Acceleration which can be taken as 0.20 for this case

S = the coefficient for the soil profile characteristics of the site which can be taken as 1.2 for this case

R = the response modification factor which is 4.50 for this case

T = the fundamental period of the building, that may be determined based on the properties of the seismic resisting system in the direction of being analyzed and the use of the methods of mechanics assuming the base of the building to be fixed but shall not exceed 1.2Ta. Alternatively the value of T, may be taken equal to the approximate fundamental period of the building, T_a, used to establish a minimum seismic base shear for the building.

The tower is nor a concrete neither a steel framing building so the equation at the below is used to calculate the fundamental period.

$$T_a = \frac{0.05h_n}{\sqrt{L}} \qquad (5.4)$$

where h_n = the height in feet above the base to the highest level of the building which is 313 feet for the tower

L = the overall length in feet of the building at the base in the direction under consideration which is 46 feet for the tower

The fundamental period T, is calculated as 2.31 using above formulas, whereas the period computed at the ambient vibration tests and dynamic analysis were 1.81 and 1.799, respectively. The period that was calculated by static formulas, was used in static lateral load procedures.

The lateral seismic shear force F_x, induced at any level, shall be determined accordance with the following formula:

$$F_x = C_{vx} V \tag{5.4}$$

where

$$C_{vx} = \frac{w_x h^k_x}{\sum w_i h^k_i} \tag{5.5}$$

k is an exponent related to the building period as follows :

For buildings having a period of 0.5 seconds or less, k = 1

For buildings having a period of 2.5 seconds or more, k = 2

For the tower that have a period between 0.5 and 2.5 seconds, k is determined by linear interpolation between 1 and 2 and found to be 1.905.

w_i, w_x = the portion of W located at or assigned to level i or x.

h_i, h_x = the height above the base to level i or x

From above equations C can be determined as 0.037 and the lateral load distribution is presented in Table 5.4.

If the tower structure is taken as isolated from the building, then the total weight is about 520.27t and the lateral load is 2.37t. The lateral load

Table 5.4 Lateral Load Distribution Along the Tower According to ATC 3-06

Height	Load	W_ih_i	F_i
+0.00		0.00	0.00
+3.00	30.90	92.70	0.14
+5.40	30.90	166.86	0.26
+8.40	30.90	259.56	0.40
+11.10	30.90	342.99	0.52
+14.10	30.90	435.69	0.67
+16.80	30.90	519.12	0.79
+21.50	40.50	870.75	1.33
+26.50	65.41	1,733.37	2.65
+30.00	63.29	1,898.70	2.90
+36.00	35.77	1,287.72	1.97
+42.00	32.10	1,348.20	2.06
+48.00	29.18	1,400.64	2.14
+54.00	28.20	1,522.80	2.33
+60.90	19.86	1,209.47	1.85
+64.20	2.30	147.66	0.23
+67.20	5.58	374.98	0.57
+76.20	6.93	528.07	0.81
+82.10	3.50	287.35	0.44
+88.85	1.50	133.28	0.20
+95.45	0.75	71.59	0.11
Total	520.27	14,631.50	22.37

distribution along the stories is given in Table 5.4. The overturning moment calculated by the values at Table 5.4 at the base of the GRP section is 2.82tm.

5.3.2 Turkish Code For Earthquake Resistant Structures

When the period of the building is computed, the seismic coefficient C shall be determined in accordance with the following formula:

$$C=C_o.K.S.I \qquad (5.6)$$

where

C_o = seismic zone coefficient which is 0.08 for II. Seismic Zone of Turkiye

K = building type coefficient which is 2.0 for towers

S = structural dynamic coefficient which is 1.0 maximum

I = structural importance coefficient which is 1.50 for PTT buildings

From above equation the lateral load coefficient is calculated as 0.240 and distribution of the total lateral seismic load at any level may be calculated by the formula:

$$F_i = (F - F_t)\frac{W_i \cdot h_i}{\sum W_i \cdot h_i} \qquad (5.7)$$

where

F = total lateral seismic load

W_i = vertical load of story i

h_i = height of the story i from the foundation level

F_t = lateral load applied on the top of the structure which will be calculated by:

$$F_t = 0.004 \cdot F\left(\frac{H}{D}\right)^2 \qquad (5.8)$$

where

H = height of the building which is 95.45m

D = width of the building at the foundation level which is about 14m

F_t can not be larger than 15% of F.

F_t is the term that may be used to include the second mode effects. In tower shaped structures, the second mode effect is not important so F_t should not be included.

If the tower structure is taken as isolated from the building, then the total weight is about 520.27t and the lateral load is 124.87t. The lateral load at the top story is not considered. The lateral load distribution along the stories is given in Table 5.5. The overturning moment calculated by the values at Table 5.4 at the base of the GRP section is 15.77 tm.

5.4 Cantilever Method

Inverted pendulum type structures like the tower being investigated are structures where the seismic resisting system acts essentially as an isolated cantilevers. Supporting columns or piers of inverted pendulum type structures shall be designed for the bending moment calculated at the base determined using the procedures of Equivalent Lateral Force Procedure for the base of the structure. One half of the calculated bending moment at the base is applied at the top and the moments along the column are varied from 1.5M at the base to 0.5M at the top. The addition of one half the moment calculated at the base is based on analysis of inverted pendulum covering a wide range of practical conditions.

Table 5.5 Lateral Load Distribution Along the Tower According to Turkish Code

Height	Weight	$W_i h_i$	F_i
+3.00	30.90	92.70	0.79
+5.40	30.90	166.86	1.42
+8.40	30.90	259.56	2.21
+11.10	30.90	342.99	2.92
+14.10	30.90	435.69	3.70
+16.80	30.90	519.12	4.41
+21.50	40.50	870.75	7.40
+26.50	65.41	1,733.37	14.73
+30.00	63.29	1,898.70	16.14
+36.00	35.77	1,287.72	10.95
+42.00	32.10	1,348.20	11.46
+48.00	29.18	1,400.64	11.91
+54.00	28.20	1,522.80	12.94
+60.90	19.86	1,209.47	10.28
+64.20	2.30	147.66	1.26
+67.20	5.58	374.98	3.19
+76.20	6.93	528.07	4.49
+82.10	3.50	287.35	2.44
+88.85	1.50	133.28	1.13
+95.45	0.75	71.59	0.61
Total	520.27	14,631.50	124.38

5.5. Strength Difference Method

ATC 3-06 states that large discontinuities in story strength can cause adverse response effects in a building. However, the usual practice is to determine what size, length, or strength of the resisting element is required (if more than the required strength is provided, so much the better), unfortunately, the extra strength in a story, if significantly different than that in adjacent stories, can produce responses which vary greatly from those calculated by using common procedures.

ATC 3-06 considers the following approach to this problem.

1. Compute the ratio of shear capacity to the design shear for each story. Denote this ratio for story n by r_n.
2. Compute r, the average r_n over all stories.
3. If for any story r_n is less than $(^2/_3)r$, modify R and Cd for the building, to $\sim R$ and $\sim Cd$ where

$$\sim C_d = 1 + \frac{C_d - 1}{2} \qquad (5.9)$$

$$\sim R = \frac{\sim C_d}{C_d} R \qquad (5.10)$$

4. Use $\sim R$ instead of R to recompute the lateral forces, and $\sim Cd$ instead of Cd in computing story drifts.

Table 5.6 Shear Capacity of Different Sections Along the Tower

Height	A E (r_n)	r	$(^2/_3)$r
+16.80 ~ +26.50	$8.85x10^6$	$7.7x10^6$	$5.2x10^6$
+26.50 ~ +30.00	$2.30x10^7$		
+30.00 ~ +60.90	$5.80x10^6$		
+60.90 ~ +82.10	$9.45x10^5$		
+82.10 ~	$3.8x10^4$		

Instead of the shear capacity ratio, AE is used and the results presented in Table 5.5.

~C_b is computed as 2.5 and ~R is computed as 2.82. So, the coefficient C_o is comes to be 0.058, but not 0.037 and the Table 5.4 revised into Table 5.7. The overturning moment calculated by the values presented at Table 5.6 at the base of the GRP section is 4.63 tm.

5.6w. Japanese Practice

Society of Steel Construction of Japan, states that the lateral shear for any kind of appendages on buildings, q, shall be determined in accordance with formula

$$q = k\ W \qquad (5.11)$$

where

k : seismic design coefficient for appendages which shall be 5 times the standard base shear coefficient, C_o

w : weight of appendage

The total weight of the GRP cylinder is about 2.50t and the lateral seismic load coefficient is about 0.240. Then, the seismic coefficient for the appendage and the overturning moment at the base of the GRP cylinder can be calculated easily. The seismic coefficient for appendage is 1.20 and the overturning moment is 121.0t.m by using the coefficient two times: first for the steel appendage on top of reinforced concrete and second for the GRP appendage on top of steel.

Table 5.7 Revised Lateral Loads Along the Tower with respect to Height

Height	Load	$W_i h_i$	F_i
+0.00		0.00	0.00
+3.00	30.90	92.70	0.23
+5.40	30.90	166.86	0.41
+8.40	30.90	259.56	0.64
+11.10	30.90	342.99	0.84
+14.10	30.90	435.69	1.07
+16.80	30.90	519.12	1.27
+21.50	40.50	870.75	2.14
+26.50	65.41	1,733.37	4.25
+30.00	63.29	1,898.70	4.66
+36.00	35.77	1,287.72	3.16
+42.00	32.10	1,348.20	3.31
+48.00	29.18	1,400.64	3.44
+54.00	28.20	1,522.80	3.74
+60.90	19.86	1,209.47	2.97
+64.20	2.30	147.66	0.36
+67.20	5.58	374.98	0.92
+76.20	6.93	528.07	1.30
+82.10	3.50	287.35	0.71
+88.85	1.50	133.28	0.33
+95.45	0.75	71.59	0.18
Total	520.27	14,631.50	22.37

CHAPTER VI

DISCUSSION OF THE RESULTS AND CONCLUSION

6.1. The Wind Loading Calculation by the Vendor

Wind load calculations were originally done for British Standard CP3, 1974 by ADC the vendor for GRP cylinders and presented as a report titled "Structural Analysis of Turkish Antenna GRP Cylinders"[19]. The wind speed is stated as 200 km/h, and this is utilized in the design.

Wind pressure according to BS-CP3 is calculated from

$$q=\frac{1}{2}pv^2 \qquad (6.1)$$

for $p=1225$ kg/m^3 and $v=200$ km/hr $=55.55$ m/s yielding $q=1890$ N/m^2.

The corresponding overturning moment at the bottom of the GRP section is calculated as 191 kN.m by assuming the bottom as a fixed base. The

design is performed for the maximum base moment using a longitudinal "instantaneous" modiolus of elasticity of $E=1x10^{10}N/m^2$, a moment of inertia of $I=4.885x10^{-3}m^4$ and an outside diameter of d=1.00m. The stress at the bottom section of the GRP antenna was found as 0.199 t/cm^2 with a maximum fibre strain of 0.25% which is lower than the allowable fubre strain which is 0.375%. It was concluded that the GRP antenna portion of the tower is safe under wind loads.

Table 6.1 Comparison of Test and Computer Results

		1st Mode	2nd Mode	3rd Mode	4th Mode
Test Results	Frequency (Hz)	0.55	1.28	2.25	3.24
	Period (s)	1.818	0.781	0.444	0.309
Computer Results	Frequency (Hz)	0.556	1.260	2.087	3.451
	Period (s)	1.799	0.794	0.479	0.290

6.2. Dynamic Analysis by Computer

The dynamic characters of the structure calculated by the tests are compared with the results of the computer model. The comparison of the results are presented in Table 6.1.

Comparison of test and computer results are perfect in first two modes

and pretty good in third and fourth modes. So the model may assumed that, it will show the load effects on the structure very well.

The tower was modelled like a cantilever bar that have the true stiffness and masses, and the shear walls were not modelled in detail. The model was not a detailed model in geometry but the masses and loads on the structure are tried to be perfect.

Comparing the two dynamic analysis methods, it was seen that time domain analysis gave higher load values that response spectrum analysis. This may arouse from an error in response spectrum analysis.

Comparison of the loads at the bottom section of the GRP antenna is presented in Table 6.2. The maximum overturning moment generated by the computer is 130.90tm which is nearly 6.7 times higher than the value held after the wind load calculations. At the assumed state of collapse, the overturning moment generated by the computer is 62.45tm which is 3.2 times higher than the wind load values. At the case of far field earthquake with 1% damping (that is considered as the state of collapse) the stress at the bottom of the GRP section is found as 0.639t/cm2. This stress value is much more larger than the allowable stress.

6.3. Equivalent Static Calculations

Equivalent static calculations are investigated mainly using ATC 3_06 and Turkish Code. Also, using some regulations of Specifications of Society of Steel Construction of Japan[3], static calculations are presented.

Table 6.2 Comparison of Equivalent Static Calculations

	Shear (t)	Moment (tm)
Design of Connections	0.38	-
Equivalent Lateral Force Procedure ATC 3-06	0.75	2.82
Equivalent Lateral Force Procedure Turkish Regulations	22.32	15.77
Stiffness Difference Method	1.22	4.63
Cantilever Method	-	171.40
Society of Steel Construction of Japan	-	121.0
Dynamic Solution:		
max, Near field earthquake, 1% damping		130.90
state of collapse, Far field earthquake, 1% damping		62.45
min, Far field earthquake, 5% damping		20.37

The results obtained from the computer model were 130.90tm maximum seismic moment at near field with 0.01 damping and 20.37tm minimum seismic moment at far field with 0.05 damping. Maximum stress at the bottom of the GRP antenna was found as 1.338t/cm^2, and the stress was found as 0.638t/cm^2 at the

state of collapse which was assumed as the far field earthquake with damping value of 1%.

Equivalent static calculation results are presented in Table 6.2 and the results of computer model are presented in Table 6.3.

Table 6.3 Maximum Moment Occurring at the Bottom of the GRP Section

		Damping	M_{max}(t.m)
Near Field	Time Analysis	0.01	-
		0.02	-
		0.05	-
	Spectrum Analysis	0.01	130.90
		0.02	88.95
		0.05	69.85
Far Field	Time Analysis	0.01	62.45
		0.02	51.53
		0.05	32.05
	Spectrum Analysis	0.01	38.19
		0.02	25.95
		0.05	20.37

The comparison between equivalent static calculations showed that every static calculation gave different results. But comparing the static solution results with

dynamic solution results it may be assumed that static calculations are useful for preliminary design only for special type of structures.

6.4. Conclusion

Model of the structure is prepared using correct masses and structural rigidities. Dynamic variables of the model is compared with the values of ambient vibration tests. Comparison of dynamic variables for the first two modes was perfect and also good for other modes. After this comparison, it may be assumed that the model demonstrates the real structure perfectly.

After comparing period and frequency values for different modes with computer model and ambient vibration test, confidence was gained about the mathematical model. The load values generated by computer model for near and far field earthquakes, different damping values and two different dynamic analysis methods are presented in Table 6.3.

Various static equivalent load calculations are used for the load values at the bottom of GRP antenna. For different methods of static calculations, completely different load values are obtained. Only, the calculations done by using the provisions presented by Society of Steel Construction of Japan, gave close

values to computer solution.

Calculations of the GRP cylinder presented by the vendor was examined. After considering all investigations about the tower, it may be said that the reason of the collapse was the underestimation of dynamic loads on the structure. The stress found at the base of the GRP antenna at far field earthquake with 1% damping was 0.639t/cm2 which is 3.20 times higher than the maximum stress calculated by the vendor considering only wind loads. Calculations of the GRP cylinder made bty the vendor was based on only static wind loads, and the dynamic characters of the structure were completely underestimated. This error in the calculation may arrouse the collapse of the GRP antenna.

It may be said that by a simple model like the model that is used in this work, it is not very hard to get a real animation of a structure. Cantilever bar like computer models may be used in dynamic analysis of tower like structures also if the structure have great structural rigidity differences.

The load effects of special structures depend on their structural geometry with the characteristics of ground motion and the dynamic characteristics of the structure such as natural frequencies, mode shapes, and damping.

The response of special structures may be computed by complex and careful dynamic analysis requiring time wise superposition of responses in different modes of vibration. In response spectrum analysis, verification of the response spectrum is very important. Vibration characteristics and the dynamic properties of the tower shaped multi element structures are important design parameters controlling their earthquake safety. Any error in variables of response spectrum analysis is very important and may cause significant differences in results of the analysis.

The dual tower shaped structures are required to be designed for seismic forces particularly when they are located in moderate to severe seismic zones. Complete elastic response of tower shaped structures to earthquake motion can be computed only by dynamic analysis requiring time wise superposition of responses in different modes of vibration. However, it is a common practice in design codes to specify equivalent static lateral loading on various structures for estimating the earthquake response. Such an approach is considered useful for preliminary design because of its simplicity.

Although, it is a common practice to define static solutions equivalent to dynamic solutions, it is advised to use equivalent static loads for special structures only for preliminary design purposes. Exact response of such special buildings to

seismic effects may only be predicted by dynamic analysis. However, the exact nature of distribution of the equivalent lateral loads along the height of tower shaped structures is not yet known, a careful static analysis may be enough for some structures but not for tower shaped high rise structures which have structural rigidity differences between floors, and build with different types of materials.

Special type of structures, like towers, multi element structures, and the ones which have large stiffness differences within floors have to be designed for seismic forces particularly when they are located in moderate or severe seismic zones. The analysis of a structure and the provision of a design requirements are necessary to provide adequate earthquake resistance in structures.

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APPENDICES

APPENDIX A

DYNAMIC ANALYSIS BY COMPUTER PROGRAM SAP90

A.1. Mode-Superposition Method

The equations of motion of a linear MDOF system is given by

$$mu + cu + ku = p(t) \qquad (A.1)$$

In general, the coefficient matrices in Eq.(A.1), m,c, and k, may have non zero coupling terms (e.i. $k_{ij} = k_{ji} = 0$), so that to solve Eq.(A.1) in its present form would require simultaneous solution of N equations in N unknowns. Mode-superposition is a method by which such a set of coupled equations can be transformed into a set of uncoupled equations through use of the normal modes of the system.

Eq.(A.1) is the original set of coupled equations of motion for an N-DOF

system, where u(t) may be physical or generalized coordinates. The response of the system to the excitation p(t) and to the initial conditions

$$u(0) = u_0 \ , \ \dot{u}(0) = \dot{u}_0 \qquad \text{(A.2)}$$

is sought.

The first step in a mode-superposition solution is to obtain the natural frequencies and natural modes of the system. If all N nodes are to be considered, then the natural frequencies and modes satisfy

$$(k - w_r^2 \, m)\phi_r = 0 \qquad \text{(A.3)}$$

giving (w_r^2 , ϕ), r = 1, 2,...,N. If the modes ϕ_r are assumed to have been normalized then modal mass, M_r and modal stiffness, K_r , calculated using

$$M_r = \phi_r^T \, m \, \phi_r$$
$$K_r = w_r^2 \, M_r \qquad \text{(A.4)}$$

Assuming that if there are any repeated frequencies' the associated modes have been orthogonalized so that the orthogonality equations

$$\phi_r^T \, m \, \phi_s = \phi_r^T \, k \, \phi_s = 0 \qquad (A.5)$$

are satisfied for all r = s. The modes are then collected to form the modal matrix Φ, that is,

$$\Phi = [\Phi_1 \;\; \Phi_2 \ldots \Phi_N] \qquad (A.6)$$

The key step in the mode-superposition procedure is to introduce the coordinate transformation.

$$u(t) = \Phi.\Omega(t) = \Sigma^N_{r=1} \, \Phi_r.\Omega_r(t) \qquad (A.7)$$

The coordinates $\Omega_r(t)$ will be referred to as principal coordinates. Eq.(A.7) is substitiuted into Eq.(A.1) and the resulting equation is multiplied by Φ^T to give the equation of motion in principal coordinates, namely

$$M\Omega + C\Omega + K\Omega = P(t) \qquad (A.8)$$

where

$M = \Phi^T \, m \, \Phi$ = modal mass matrix

$C = \Phi^T \, c \, \Phi$ = modal damping matrix

$K = \Phi^T \, k \, \Phi$ = modal stiffness matrix (A.9)

$P(t) = \Phi^T \, p(t)$ = modal force matrix

Due to the orthogonality conditions, of Eq.(A.5), M and K are diagonal matrices, so the equations of motion in modal coordinates, Eq.(A.8), are coupled only through nonzero off-diagonal coefficients in the modal damping matrix C.

The total response $\cap(t)$ can be obtained as a superposition of the response due to initial conditions alone and response due to the excitation alone.

$$u(0) = \Phi.\cap(0)$$

$$u(0) = \Phi.\cap(0) \qquad \text{(A.10)}$$

Multiplying these equations by Φ^T m

$$\Phi^T m\, u(0) = M \cap(0)$$

$$\Phi^T m\, u(0) = M \cap(0) \qquad \text{(A.11)}$$

Since M is diagonal, Eq.(A.11) can be solved for the modal initial conditions giving

$$\cap_r(0) = (1/M_r)\, \Phi_r^T m\, u(0)$$

$$r = 1, 2,..., N \qquad \text{(A.12)}$$

$$\cap_r(0) = (1/M_r)\, \Phi_r^T m\, u(0)$$

A.2.Calculations of Frequencies and Mode Shapes

The dynamic analysis of a structural system using mode superposition requires the solution of the generalized eigenvalue problem, as the first step.

$$K \Phi = w^2 M \Phi \qquad (A.13)$$

where w and Φ are free vibration frequency and mode shape, respectively. The mass matrix is diagonal with partly zero diagonal elements. The program assumes that only the lowest p eigenvalues and corresponding eigenvectors are needed. The solution of Eq.(A.1) can therefore be written as

$$K \Phi = M \Phi \Omega^2 \qquad (A.14)$$

where Ω^2 is a diagonal matrix with the p smallest eigenvalues, i.e. $\Omega^2 = \text{diag}(w_i^2)$ and Φ stores the corresponding M-orthonormalized eigenvectors $\Phi 1, \Phi 2, ..., \Phi p$. Two different solution procedures are used in the program, a determinant search technique or a subspace iteration solution. The determinant search solution is carried out when the stiffness matrix can be contained in high-speed storage in one block. Therefore, for systems of large order and bandwidth, the subspace iteration method is used. Both solution techniques solve a transformation to the standard form.

A.3. The Determinant Search Solution

The determinant search technique is best suited for the analysis of large systems in which K and M have small bandwidths. Basically the solution algorithm combines triangular factorization and vector inverse iteration in an optimum manner to calculate the required eigenvalues and eigenvectors; these are obtained in sequence starting from the least dominant eigenpair w_1^2, Φ_1. An efficient accelerated secant iteration procedure which operates on the characteristic polynomial.

$$p(w^2) = \det (K - w^2 M) \qquad (A.15)$$

is used to obtain a shift near the next unknown eigenvalue. The eigenvalue separation theorem is used in this iteration. Each determinant evaluation requires a triangular factorization of the matrix $K-w^2 M$. Once a shift near the unknown eigenvalue has been obtained, inverse iteration is used to calculate the eigenvector; the eigenvalue is obtained by adding the Rayleigh Quotient correction to the shift value.

A.4. The Subspace Iteration Solution

When the system is too large to be completely contained in high speed

storage, the subspace iteration solution is carried out. The iteration can be interpreted as a repeated application of the Ritz method, in which the computed eigenvectors from one step are used as a trial basis vectors for the next iteration until convergence to the required p eigenvalues and eigenvectors is obtained.

The solution is carried out by iterating simultaneously with q linearly independent vectors, where q>p. In the k'th iteration the vectors span the q-dimensional subspace are calculated; i.e. when the vectors span the p-dimensional least dominant subspace, the required eigenvalues and eigenvectors are obtained.

Let v_0 store the starting vectors, then the k'th iteration is described as follows:

Solve for vectors V_k which span ϵ_k

$$K V_k = M V_{k-1} \tag{A.16}$$

Calculate the properties of K and M onto ϵ_k

$$K_k = V_k^T K V_k \tag{A.17}$$

$$M_k = V_k^T M V_k \tag{A.18}$$

Solve for the eigensystem of K_k and M_k

$$K_k Q_k = M_k Q_k \Omega_k^2 \qquad (A.19)$$

and calculate the k'th improved approximation to the eigenvectors

$$V_k = V_k.Q_k \qquad (A.20)$$

Provided that the starting subspace is not orthogonal to any of the required eigenvectors, the iteration converges to the desired result. i.e. $\Omega_k^2 \rightarrow \Omega^2$ and $V_k \rightarrow \Phi$ as $k \rightarrow \infty$. The number of vectors q used in the iteration is taken greater than the desired number of eigenvectors in order to accelerate the convergence of the process. The number of iterations required to achieve satisfactory convergence depends, of course, on the quality of the starting vectors v_o . Unless otherwise the program generates q starting vectors where q = min(2p,p+8), which has proven to be effective in general applications. At convergence a Sturm sequence check can be requested to verify that the lowest p eigenvalues have been found.

A.5.Dynamic Analysis

In dynamic response analysis the solution of the equations

$$M\,u + C\,u_r + K\,u_r = R(t) \qquad (A.21)$$

is required, where R(t) can be a vector of arbitrary time varying loads or of effective loads which result from ground motion. Specificaly, in the case of ground motion, if it is assumed that the structure is uniformly subjected to the ground acceleration u_g , the equilibrium equations considered are

$$M.u_r + C.u_r + K.u_r = -M.u_g \qquad (A.22)$$

where u_r is the relative displacement of the structure with respect to the ground, i.e. $u_r = u - u_g$

The program can carry out a history analysis for solution of Eqs.(A.15) and (A.16), or a response spectrum analysis for solution of Eq.(A.16). The history anaylsis can be carried out using mode superposition or direct integration. The response spectrum analysis necessities, of course, first the solution of the required eigensystem.

A.6.Response History Analysis by Mode Superposition

In the mode superposition analysis, it is assumed that the structural response can be described adequately by the p lowest vibration modes, where p<<n. Using the transformation u = ΦX, where the columns in Φ are the p M_orthonormalized eigenvectors, Eq.(A.15) can be written as

$$X + \delta X + \Omega^2 X = \Phi^T R \tag{A.23}$$

where

$$\delta = \text{diag}(2w_i \Gamma_i); \qquad \Phi^2 = \text{diag} (w_i^2) \tag{A.24}$$

In Eq.(A.18) it is that the damping matrix C satisfies the modal orthogonality condition

$$\Phi_i^T C \Phi_j = \Phi \quad (i = j) \tag{A.25}$$

Eq.(A.17) therefore represents p uncoupled second order differential equations. These are solved in the program using the Wilson Θ-Method, which is an unconditionally stable step-by-step integration scheme. The same time step is used in the integration of all equations to simplify the calculation of stress components at preselected times.

In the case of prescribed ground motion $u_r = \Phi X$ and in Eq.(A.17) the right hand side is given by $-\Phi^T.M.u_g$, where the ground acceleration is considered as the sum of the components in the x, y and z directions.

A.7. Response History Analysis by Direct Integration

The solution of the equations of motion, Eqs.(A.15) and (A.16), can be obtained by direct integration. In the program the Wilson Θ-method is used, which is unconditionally stable. The algorithm employed is summarized in Table (A.1). This form of damping is easily taken account of in the analysis, because no storage and no multiplications for a damping matrix is required.

A.8. Response Spectrum Analysis

In this analysis the ground acceleration vector in Eq.(A.16) is written as

$$u_g = u_{gx} + u_{gy} + u_{gz} \tag{A.26}$$

where u_{gx} , u_{gy} and u_{gz} are the ground acceleration in the x, y and z directions, respectively. The equation for the response in the r'th mode is therefore

$$x_r + 2\,\epsilon_r\, u_r\, x_r + u_r^2\, x_r = r_{rx} + r_{ry} + r_{rz} \tag{A.27}$$

Table A.1 Step-by-Step Direct Integration Algorithm

Initial Calculations

1. Calculate the following constants

   (Assume $C = \alpha.M + \beta.K$)

   $\Phi = 1.4$, $\tau = \Theta\ \delta t$

   $a_0 = (6 + 3\alpha\tau) / (\tau^2 + 3\beta\tau)$ $\quad b_0 = \alpha - \beta a_0$

   $a_1 = 6 / \tau^2 + 3b_0 / \tau$ $\quad a_2 = 6 / \tau + 2b_0$

   $a_3 = 2 + \tau b_0 / 2$ $\quad a_4 = 6 / [\ \Theta(38\tau + \tau^2)\ ]$

   $b_1 = \beta\ a_4$

   $a_5 = 3b_1 / \tau + 6 / (\tau^2 + \Theta)$ $\quad a_6 = 2b_1 - 6 / (\tau\Theta)$

   $a_7 = (b_1\ \tau / 2) + 1 - (3 / \Theta)$ $\quad a_8 = \delta t / 2$

   $a_9 = \delta t^2 / 3$ $\quad a_{10} = (1/2)\ a_9$

2. Form effective stiffness matrix $K^* = K + a_0 M$

3. Triangularize K_*

For Each Time Increment

1. Form effective load vector R_t^*

   $R_t^* = R_t - \Theta(R_{t+\delta t} - R_t) + M(a_1 u_t + a_2 u_t + a_3 u_t)$

2. Solve for effective displacement vector u_t^*

   $K^*\ u_t^* = R_t^*$

3. Calculate new acceleration, velocity and displacement vectors,

   $u_{t+\delta t} = a_4\ u_t + a_5\ u_t + a_6\ u_t + a_7\ u_t$

$$u_{t+\delta t} = u_t - a_8 \left(u_{t+\delta t} - u_t \right)$$

$$u_{t+\delta t} = u_t + \delta t\, u_t + a_9\, u_t + a_{10}\, u_{t+\delta t}$$

4. Calculate element stresses if desired where x_r is the t'th element in X and

$$r_{rx} = -\phi_r^T.M.u_{gx} \; ; \; r_{ry} = -\phi_r^T.M.u_{gy} \; ; \; r_{rz} = -\phi_r^T.M.u_{gz}$$

Using the definition of the spectral displacement, the maximum absolute modal displacements of the structure subjected to an acceleration into the x direction are

$$r_{rx}^{(max)} = \phi_r \, |\phi_r^T.M.I_x| \; S_x (w_r) \qquad (A.28)$$

where $S_x (w_r)$ is the spectral displacement into the x direction corresponding to the frequency w_r and I_x is a null vector except that those elements are equal to one which correspond to the x-translational degrees of freedom. Similarly, for the responses due to a ground acceleration into y and z-directions

$$u_{ry}^{(max)} = \phi_r \, |\phi_r^T.M.I_y \, | \; S_y (w_r) \qquad (A.29)$$

$$u_{rz}^{(max)} = \phi_r \, |\phi_r^T.M.I_z \, | \; S_z (w_r) \qquad (A.30)$$

and the total maximum response in the r'th mode is assumed to be

$$u_r^{(max)} = u_{rx}^{(max)} + u_{ry}^{(max)} + u_{rz}^{(max)} \quad (A.31)$$

Program SAP90 calculates the maximum responses in each of the p lowest modes, where the spectra (displacements or acceleration) into the x, y and x-directions are assumed to be proportional to each other. The total response for displacements and stress resultants is calculated as the square root of the sum of the squares of the modal maximum responses.

APPENDIX B

COMPUTER INPUT FOR SAP90

The computer model input prepared by an ASCII editor is given at below:

PTT KARS TV ANTENNA DYNAMIC ANALYSIS

SYSTEM

N=1517 V=8

JOINTS

1 X=+6.727 Y=+1.531 Z=0.000

2 X=+5.401 Y=+4.291

3 X=+3.010 Y=+6.206

4 X=+0.022 Y=+6.897

5 X=-2.971 Y=+6.224

6 X=-5.382 Y=+4.314

7 X=-6.725 Y=+1.541

8 X=-6.725 Y=-1.541

9 X=-5.382 Y=-4.319

10 X=-2.971 Y=-6.224

11 X=+0.022 Y=-6.897

12 X=+3.010 Y=-6.206

13 X=+5.401 Y=-4.291

14 X=+6.727 Y=-1.531

15 X=-1.600 Y=+0.000

101 X=+6.727 Y=+1.531 Z=3.000

102 X=+5.401 Y=+4.291

103 X=+3.010 Y=+6.206

104 X=+0.022 Y=+6.897

105 X=-2.971 Y=+6.224

106 X=-5.382 Y=+4.314

107 X=-6.725 Y=+1.541

108 X=-6.725 Y=-1.541

109 X=-5.382 Y=-4.319

110 X=-2.971 Y=-6.224

111 X=+0.022 Y=-6.897

112 X=+3.010 Y=-6.206

113 X=+5.401 Y=-4.291

114 X=+6.727 Y=-1.531

115 X=-1.600 Y=+0.000

201 X=+6.727 Y=+1.531 Z=5.400

202 X=+5.401 Y=+4.291

203 X=+3.010 Y=+6.206

204 X=+0.022 Y=+6.897

205 X=-2.971 Y=+6.224

206 X=-5.382 Y=+4.314

207 X=-6.725 Y=+1.541

208 X=-6.725 Y=-1.541

209 X=-5.382 Y=-4.319

210 X=-2.971 Y=-6.224

211 X=+0.022 Y=-6.897

212 X=+3.010 Y=-6.206

213 X=+5.401 Y=-4.291

214 X=+6.727 Y=-1.531

215 X=-1.600 Y=+0.000

301 X=+6.727 Y=+1.531 Z=8.600

302 X=+5.401 Y=+4.291

303 X=+3.010 Y=+6.206

304 X=+0.022 Y=+6.897

305 X=-2.971 Y=+6.224

306 X=-5.382 Y=+4.314

307 X=-6.725 Y=+1.541

308 X=-6.725 Y=-1.541

309 X=-5.382 Y=-4.319

310 X=-2.971 Y=-6.224

311 X=+0.022 Y=-6.897

312 X=+3.010 Y=-6.206

313 X=+5.401 Y=-4.291

314 X=+6.727 Y=-1.531

315 X=-1.600 Y=+0.000

401 X=+6.727 Y=+1.531 Z=11.300

402 X=+5.401 Y=+4.291

403 X=+3.010 Y=+6.206

404 X=+0.022 Y=+6.897

405 X=-2.971 Y=+6.224

406 X=-5.382 Y=+4.314

407 X=-6.725 Y=+1.541

408 X=-6.725 Y=-1.541

409 X=-5.382 Y=-4.319

410 X=-2.971 Y=-6.224

411 X=+0.022 Y=-6.897

412 X=+3.010 Y=-6.206

413 X=+5.401 Y=-4.291

414 X=+6.727 Y=-1.531

415 X=-1.600 Y=+0.000

501 X=+6.727 Y=+1.531 Z=14.100

502 X=+5.401 Y=+4.291

503 X=+3.010 Y=+6.206

504 X=+0.022 Y=+6.897

505 X=-2.971 Y=+6.224

506 X=-5.382 Y=+4.314

507 X=-6.725 Y=+1.541

508 X=-6.725 Y=-1.541

509 X=-5.382 Y=-4.319

510 X=-2.971 Y=-6.224

511 X=+0.022 Y=-6.897

512 X=+3.010 Y=-6.206

513 X=+5.401 Y=-4.291

514 X=+6.727 Y=-1.531

515 X=-1.600 Y=+0.000

527 Z=+26.000 G=515,527,1

531 Z=+30.000 G=527,531,1

562 Z=+60.900 G=531,562,1

583 Z=+82.100 G=562,583,1

598 Z=+97.000 G=583,598,1

RESTRAINTS

1 15 1 R=1,1,1,1,1,1

101 115 1 R=0,0,0,0,0,0

201 215 1 R=0,0,0,0,0,0

301 315 1 R=0,0,0,0,0,0

401 415 1 R=0,0,0,0,0,0

501 598 1 R=0,0,0,0,0,0

CONSTRAINTS

101 114 1 C=115,115,0,0,0,0

201 214 1 C=215,215,0,0,0,0

301 314 1 C=315,315,0,0,0,0

401 414 1 C=415,415,0,0,0,0

501 514 1 C=515,515,0,0,0,0

FRAME

NM=12

1 A=0.15 I=0.00100,0.00300 M=0.110 E=2.7E6 : Reinf.Concrete

2 A=3.336 I=7.55900,7.55900 J=0.0001 M=0.850 :

3 A=14.13 I=19.8700,19.8700 J=0.0001 M=3.432

4 A=2.755 I=1.37900,1.37900 J=0.0001 M=0.702

5 A=2.600 I=1.32600,1.32600 J=0.0001 M=0.662

6 A=2.444 I=1.27300,1.27300 J=0.0001 M=0.623

7 A=2.289 I=1.21900,1.21900 J=0.0001 M=0.583

8 A=2.133 I=1.11600,1.11600 J=0.0001 M=0.544

9 A=1.978 I=1.11300,1.11300 J=0.0001 M=0.504

10 A=0.067 I=0.18700,0.18700 J=0.00001 M=0.105 E=2.1E7 : Steel

11 A=0.038 I=0.00488,0.00488 J=0.00001 M=0.020 E=0.9E6 : GRP

12 A=0.04 I=0.00013,0.00013 E=2.7E6 : 20x20 Column

C MAIN TOWER

1 15 115 M=2 LP=2,0

2 115 215

3 215 315

4 315 415

5 415 515

6 515 516 G=11,1,1,1

18 527 528 M=3 G=3,1,1,1

22 531 532 M=4 G=4,1,1,1

27 536 537 M=5 G=4,1,1,1

32 541 542 M=6 G=4,1,1,1

37 546 547 M=7 G=4,1,1,1

42 551 552 M=8 G=4,1,1,1

47 556 557 M=9 G=5,1,1,1

53 562 563 M=10 G=20,1,1,1

74 583 584 M=11 G=14,1,1,1

C 20x20 COLUMNS

101 1 101 M=12 LP=2,0 G=4,100,100,100

102 2 102 G=4,100,100,100

103 3 103 G=4,100,100,100

104 4 104 G=4,100,100,100

105 5 105 G=4,100,100,100

106 6 106 G=4,100,100,100

107 7 107 G=4,100,100,100

108 8 108 G=4,100,100,100

109 9 109 G=4,100,100,100

110 10 110 G=4,100,100,100

111 11 111 G=4,100,100,100

112 12 112 G=4,100,100,100

113 13 113 G=4,100,100,100

114 14 114 G=4,100,100,100

C 20x20 BEAMS

121 101 102 LP=1,0 G=3,100,100,100

122 102 103 G=3,100,100,100

123 103 104 G=3,100,100,100

124 104 105 G=3,100,100,100

125 105 106 G=3,100,100,100

126 106 107 G=3,100,100,100

127 107 108 G=3,100,100,100

128 108 109 G=3,100,100,100

129 109 110 G=3,100,100,100

130 110 111 G=3,100,100,100

131 111 112 G=3,100,100,100

132 112 113 G=3,100,100,100

133 113 114 G=3,100,100,100

134 114 101 G=3,100,100,100

C FLOOR BEAMS

1101 101 115 M=1 LP=1,0 RE=0,2.125

1111 114 115 RE=0,2.125

1102 102 115 RE=0,2.125

1112 113 115 RE=0,2.125

1103 103 115 RE=0,2.125

1113 112 115 RE=0,2.125

1104 104 115 RE=0,2.125

1114 111 115 RE=0,2.125

1105 105 115 RE=0,2.125

1115 110 115 RE=0,2.125

1106 106 115 RE=0,2.125

1116 109 115 RE=0,2.125

1107 107 115 RE=0,2.125

1117 108 115 RE=0,2.125

1201 201 215 M=1 RE=0,2.125

1211 214 215 RE=0,2.125

1202 202 215 RE=0,2.125

1212 213 215 RE=0,2.125

1203 203 215 RE=0,2.125

1213 212 215 RE=0,2.125

1204 204 215 RE=0,2.125

1214 211 215 RE=0,2.125

1205 205 215 RE=0,2.125

1215 210 215 RE=0,2.125

1206 206 215 RE=0,2.125

1216 209 215 RE=0,2.125

1207 207 215 RE=0,2.125

1217 208 215 RE=0,2.125

1301 301 315 M=1 RE=0,2.125

1311 314 315 RE=0,2.125

1302 302 315 RE=0,2.125

1312 313 315 RE=0,2.125

1303 303 315 RE=0,2.125

1313 312 315 RE=0,2.125

1304 304 315 RE=0,2.125

1314 311 315 RE=0,2.125

1305 305 315 RE=0,2.125

1315 310 315 RE=0,2.125

1306 306 315 RE=0,2.125

1316 309 315 RE=0,2.125

1307 307 315 RE=0,2.125

1317 308 315 RE=0,2.125

1401 401 415 M=1 RE=0,2.125

1411 414 415 RE=0,2.125

1402 402 415 RE=0,2.125

1412 413 415 RE=0,2.125

1403 403 415 RE=0,2.125

1413 412 415 RE=0,2.125

1404 404 415 RE=0,2.125

1414 411 415 RE=0,2.125

1405 405 415 RE=0,2.125

1415 410 415 RE=0,2.125

1406 406 415 RE=0,2.125

1416 409 415 RE=0,2.125

1407 407 415 RE=0,2.125

1417 408 415 RE=0,2.125

1501 501 515 M=1 RE=0,2.125

1511 514 515 RE=0,2.125

1502 502 515 RE=0,2.125

1512 513 515 RE=0,2.125

1503 503 515 RE=0,2.125

1513 512 515 RE=0,2.125

1504 504 515 RE=0,2.125

1514 511 515 RE=0,2.125

1505 505 515 RE=0,2.125

1515 510 515 RE=0,2.125

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1506 506 515          RE=0,2.125

1516 509 515          RE=0,2.125

1507 507 515          RE=0,2.125

1517 508 515          RE=0,2.125

MASSES

583   M=1.000,1.000

562   M=1.000,1.000

SHELL

NM=1

1   U=0.20     M=0.051   E=2.65E6

  1  JQ=  1,  2,101,102   ETYPE=0   M=1   TH=0.20   LP=0   G=13,1

 14  JQ= 14,  1,114,101

101  JQ=101,102,201,202                    G=13,1

114  JQ=114,101,214,201

201  JQ=201,202,301,302                    G=13,1

214  JQ=214,201,314,301

301  JQ=301,302,401,402                    G=13,1

314  JQ=314,301,414,401

401  JQ=401,402,501,502                    G=13,1

414  JQ=414,401,514,501
```