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Seismic Response and Resistance Capacity of "As Built" VVER 440-230 NPP Kozloduy - Verification of the Results by Experiments and Real Earthquake

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1. INTRODUCTION

NPP Kozloduy 440 MW model 230 units weren't designed for earthquakes but they have withstood succesfuly the Vranchea earthquake on 4-th of March 1977 with site peak ground acceleration 83 sm/s^2. Only crakcs in shear masonry walls and joints between prefabricated walls and roof panels were observed. Displacements up to 140 mm in hung steam generators were recorded.
Units 1 and 2 as well as units 3 and 4 (under construction during the earthquake) have been recalculated for maximum peak ground acceleration 0.1 g. Response spectra from N-S accelerogram recorded in Bucharest and scaled to 0.1 g have been determined. According to calculations in 1980 the main reactor buildings have had sufficient earthquake resistance capacity for the accepted design seismic excitation. The floor response spectra have been calculated by simple two-dimensional frame mathematical models in both directions, taking into consideration only columns and beams without shear walls. On this basis Japanese hydraulic snubbers for reducing the displacements in primary curcuit components have been designed and instaled.
After dynamic experimental testing of 440 MW units 1984-1989 a 2 - 3 time smaller natural periods have been determined and on this basis new two-dimensional (2D) mathematical models have been established taking into consideration shear walls and soil-structure interaction. Computing codes SFA - Willson and FLUSH have been used for structural analyses. The new floor response spectra under the same seismic conditions significantly differs from previous ones, because of different rigidities and natural periods of the matematical models.
The next more detailed full scale experiments [1] of each of the four 440 MW units have demonstrated :
a.) The nonsimmetric design of 440 MW structures in plan and elevation , the large excentricitiy between the center of rigidities and masses lead to significant space rotationnal effects which can't be investigated by 2D models;
b.) All turbine halls of the four 440 MW units are connected in dilatational gaps by 2 crane rails. The reactor buildings of units 1 and 2, respectively 3 and 4, are connected with 4 crane rails. There are other technological connections (steel stairs, pipes,

platforms) between the substructures which change dynamic response of structures in the four units;
c.) Establishing of simplified stick/beam models for 440 MW nonsimmetric and complicated structures is not sufficient [2,3].
Above mentioned peculiarities and circumstances lead to the necessity of establishing a detailed 3D mathematical model verified by different experiments and a real (Vranchea 1977) earthquake which had produced cracks in some components of units 1 and 2. The demages were used as a criteria for reaching the first limit stage in shear walls.
To define 3D model for detailed analyses of "as built" building is necessary to include in the model :
a.) all structural components resisting and participating in earthquake vibrations including infilling walls;
b.) technological connections in dilatational gaps between the substructure and separate units with respective spring constants;
c.) foundations with respective spring constants;
d.) soil characteristics according to the respective ground models for soil-structure interaction analyses.
The principles and results in establishing of the 3D models and their verification by experiments are given in Annex 1.
Some results of seismic response analyses, seismic safety margins evaluation and resistance capacity of 440 MW units are presented in Annex 2.

2. OBJECTIVES OF THE DETAIL SEISMIC RESPONSE INVESTIGATIONS

The existing System for Industrial Antiseismic Protection (SIAZ) for automatic shut down of the six NPP Kozloduy reactors and recording of earthquakes on free field, foundation, floores and critical equipment have been reconstructed and improved in the frame of the IAEA Techical Assistence Project BUL/9/012.
During an eartquake 87 sensors will record the seismic excitation and the data will be used for control of :
a.) the established design envelope response spectra for site free field;
b.) site peak ground acceleration determined in function of the source magnitude, distance and attenuation lows;
c.) mathematical models, theoreticaly calculated dinamic characteristics of the structures and floor response spectra.
Establishing of 3D mathematical models for detailed structural analyses of "as built" 44o MW units is necessary not only for structural response control but also for :
2.1. Determination of seismic triggering levels for automaticaly shutdown of reactors form SIAZ sistem in funtion of :
- seismic safety margins of respective "as built" units;
- siesmic resistance capacity of critical components.
2.2. Planning and organizing of post earthquake walk down inspections for :
- control behaviour of safety sistems, equipment, structure and so on during eathquakes;
- taking decisions for starting the shut down reactors immediatly, or after some upgrading, or after few months analyses and upgrading.

2.3. Control units seismic safety by analyses with 3D models and recorded earthquake scaled to maximum probable magnitude.
2.4. Increasing of units seismic safety on the basis of :
- walk down post earthquake inspections;
- detailed analyses with 3D mathematical models with recorded and scaled earthquakes.

3. SCOPE OF INVESTIGATIONS BY 3D MODELS

35 model variants were established for investigation of :
a.) Response of structures for simultaneous three component uncoupled accelerograms (designed and from real earthquake) and analyses of :
- floor response accelerograms, velocigrams and seismograms and respective response spectra in three directions at respective critical dampings;
- comparing the floor response spectra from 3D models with those from 2D models;
- distribution of maximum acceleration and displacements in structures in function of different structural and ground conditions.

b.) Soil-structure interaction effects on response in linear and equivalent nonlinear assumptions.
c.) Interaction between the substructures and adjacent heavy masses in respective structural unit.
d.) Interaction between neighbouring units as a result of existing crane rails and other connections.
e.) stress-strain structural analyses during the time duration of 3-component accelerograms.
f.) Earthquake resistance capacity of "as built" units and defining critical structural components.
Data from systems and equipment seismic safety margins and critical components have been used from other detailed investigations [4]. Some of the results of the above mentioned problems are discussed in Annex 2.

4. CONCLUSIONS

The nonsimmetric design of 440 MW structures in plan and elevation, the large excentricity betweenthe center of rigidities and masses, as well as technological connections between the separate substructures and units leads to complicated space response and rotational effects which can't be investigated by simplified 2D models or simplified 3D stick / beam models.
Three dimentional detailed "as built" mathematical models were established and verified by series of experiments and real earthquake for :
a.) detailed analyses of "as built" structural response for past and future three component accelerograms;
b.) comparing the results from 2D and 3D models with records from earthquakes;
c.) detailed analyses of seismic safety margins in respect :
- to define critical structural components and the safety of existing technological connections;
- determination of seismic triggering levels for automaticaly shutdown of reactors in function of seismic safety of respective units;

- planning and organization of post earthkuake walk down inspection and taking decisions for starting the shut down reactors or their upgrading;

Some of the results from establishing and verifying the "as built" models, seismic response and seismic margins analyses are given in annexes 1 and 2.

Defining of Three Dimensional Models for 440 MW NPP Kozloduy Units and Their Verification by Experiments and Real Earthquake

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1. PRINCIPLES IN DEFINING THE 3D MODELS

Three dimensional mathematical models for NPP Kozloduy 440MW units 1 - 4 consisting of (fig1.1) 5 substructures (turbine hall, reactor, intermediate and auxilary buildings, with water tower reservoir for units 3 and 4 only) have been defined on the basis of Finite Element Metod (FEM) analysed by SAP-90 computing code [5]. Nonsimetric design in plan and elevation, excentricity between center of rigidities and masses (fig.1.1) leads to difficult modelling and necessity of model verification by series of experiments. The models were established by introducing the folowing structural elements (fig.1.2 and 1.3) :
- 861 columns floor and roof beams;
- 864 shear concrete and masonry walls including : reactor, foundation, stairs and elevator towers, diagonal and other walls;
- 625 floor and roof slabs;
- 198 three-axial foundation springs for modelling soil-structure interaction;
- 68 two-axial springs for modelling the interaction between substructures and units;
- 1200 nodes and masses concentrated in nodal points - each with 6 degree of freedom, or total 6300 degrees of freedom sistem.

The number of model modifications were 35 with variation of :
- ground conditions representation;
- connections between substructures and units;
- structural component representation;
- seismic excitation - design and real earthquakes.

2. VERIFICATION OF 3D MODELS BY EXPERIMENTS

Two dimensinal mathematical models, used for analyses of 440 MW units in the periods1980 - 1988 have been improved by experiments [1]. Experimental data were used for verification of 3D models and planning additional experiments for investigations of interaction between the substructures and neighbouring units.
Special ambient and force vibration testing (resonanse metod) were performed in 1991 by two dinamic actuators fixed on the roof of the intermediate building [6].
The excitations were performed in the two main building axes (X and Y) and rotation with measurment of the displacements and accelerations on different levels. The space modes of vibration and interaction of the tested unit 3 with neighbouring units 4 and 2, as well as with water tower reservoir were recorded.
The calculated natural frequencies by the 3D models are compared with the experimental ones (table 1). There is very good correspondence between analytical and experimental data. During the experiments some of the local natural frequencies

were not actuated and measured. This is the reason the experimental frequency No 5 to corespond to the analitical frequency No 15 etc.

Table 1.1 Comparision of experimental and analitical frequencies

direction	freq No	f1	f2	f3	f4	f5	f6	f7	f8	f9
N-S	experim	2.02	2.66	2.85	3.05	3.60	4.15	4.45	5.45	5.75
	freq No	f1	f2	f3	f4	f15	f20	f23	f24	f25
	analitical	2.01	2.64	2.95	2.96	3.48	4.13	4.72	5.46	5.74

direction	freq No				f1	f2	f3	f4	f5	f6
E-W	experim				3.04	3.40	4.10	4.30	5.10	5.65
	freq No				f5	fi4	f19	f21	f22	f26
	analitical				2.97	3.47	3.98	4.53	4.69	5.93

The analitical and experimental horizontal roof mode shapes are compared on fig.1.4. There is a good correspondence and it can be seen that the roofs are not very rigid in plan even for small vibrations at 0.00226 g.
The correspondence between analitical and experimental mode shapes in transversal direction (N-S frame 17) are also very good - fig.1.5 and 1.6.
Some of the space mode shapes are given on fig. 1.8,1.9 and 1.10. It's clear, that: a.) mode No 1 (fig.1.8) is a local resonance in turbine hall - part of structure between axes 1 and 10 , which is the most flexibal part of the complete structure;
b.) mode No 4 (fig.1.9) is a resonanse of reactor roof;
c.) mode No 5 (fig.1.10) is a local resonance of columns of row A.
The experimental results shows similar character of mode shapes.

3. VERIFICATION OF 3D MODELS BY VRANCHEA 1977 REAL EARTHQUAKE

The three Vranchea accelerograms registered on 4 -th of March 1977 in Bucharest were modified by deconvolution and convolution for geological conditions and distance to NPP Kozloduy site. The verified 3D model and 3-component accelerograms were used for determination of relative displacements in brick masonry walls between axes B and C dameged during the earthquake. The bearing capacity and the horizontal relative displacements in masonry walls, for appearing of first cracks have been determined by experimentaly established relations [7]. The horizontal seismic force (H) which causethe first crack in the masonry is :

$$H = 3 I d (B - 0.5B^2) R / F$$

at relative horizontal deformations :

$$Dh = 0.2 h^2 H^2 C / (d^2 r E I D^2)$$

where : l, h, D, d - lenght, height, diagonal and thickness of masonry wall resp.;
R, E - tensile strenght of masonry and modulus of elasticity resp;
F, C - factors of dimensions l and h ; B = l / h.

The calculated by 3D model relative displacements in masonry walls corespond to those, calculated by the above relation. This is indication that the 3D model and assumptions for calculation correspond to the actual structural behaviour during 1977 earthquake. The cracks in the reactor reinforced concrete wall during the earthquake weren't analysed because of lack of data concerning steel reinforcement and location of the cranes which maybe had localy increased seismic effects.

4. CONCLUSIONS

Simplified 2D and 3D (beam/stick) models are preferable for design and analyses of simmetrical structures. The 440 MW units in NPP Kozloduy are nonsimmetric with big differencies in rigidity in plan and elevation, which leads to big excentricity between centers of rigidities and masses and significant torsional effects. There are many technological connections between substructures and between neighbouring units, which significantly affects the response. For considering all these pecularities a detail space "as built" mathematical model with 2350 elements and 6300 degrees of freedom was established.

Data from full scale ambient and force vibration (resonanse method) tests were used for verification of the "as built" model. 35 modifications of the model were analysed and good correspondances between experimental and analitical natural frequencies and mode shapes were obtained.

The model was verified also by real earthquake - Vranchea 1977. From aveilable data for cracks in masonry walls caused by Vranchea 1977 earthquake the relative displacements between the floors were calculated and compared with those, determined analiticaly by the model. There were good correspondances between analitical and real data.

The verified model was used for detail investigations described in other presentations.

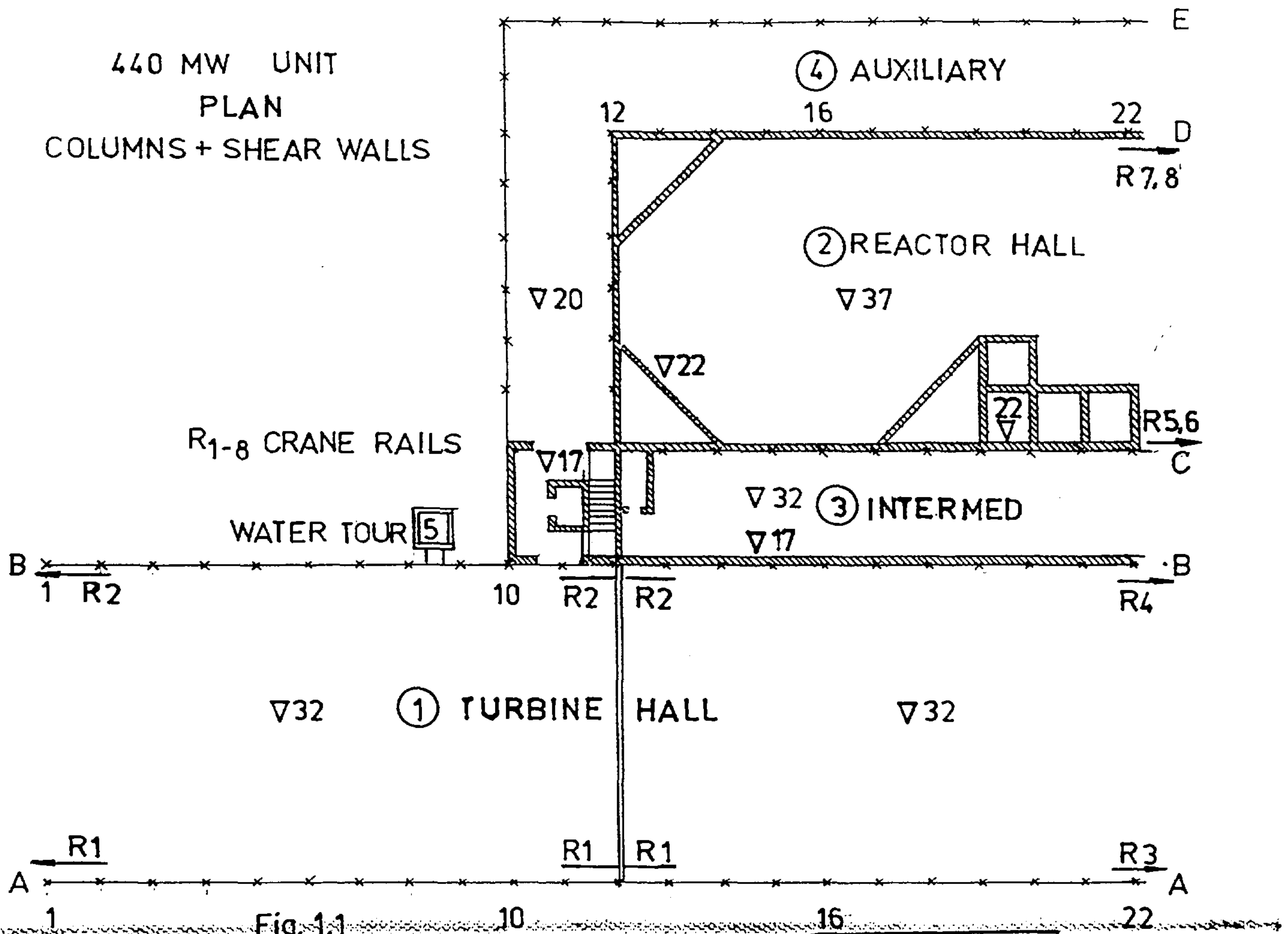

440 MW UNIT
PLAN
COLUMNS + SHEAR WALLS
④ AUXILIARY
② REACTOR HALL
③ INTERMED
① TURBINE HALL
⑤
WATER TOUR
R1-8 CRANE RAILS
∇20
∇37
∇22
∇17
∇32
∇22
∇32
∇32
∇17
R7,8
R5,6
R4
R3
R2
R2
R2
R1
R1
R1
A
A
B
B
C
D
E
1
10
12
16
22

Fig. 1.1

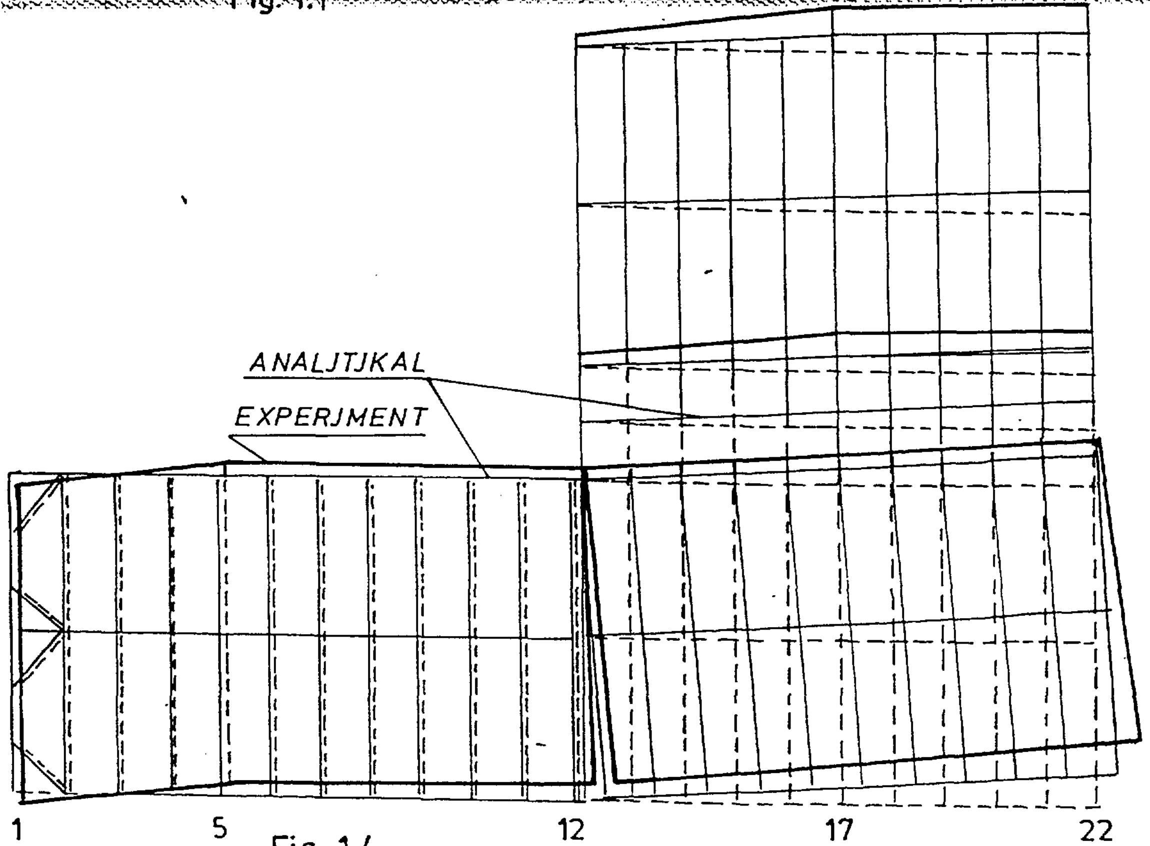

ANALJTJKAL
EXPERJMENT
1
5
12
17
22

Fig. 1.4

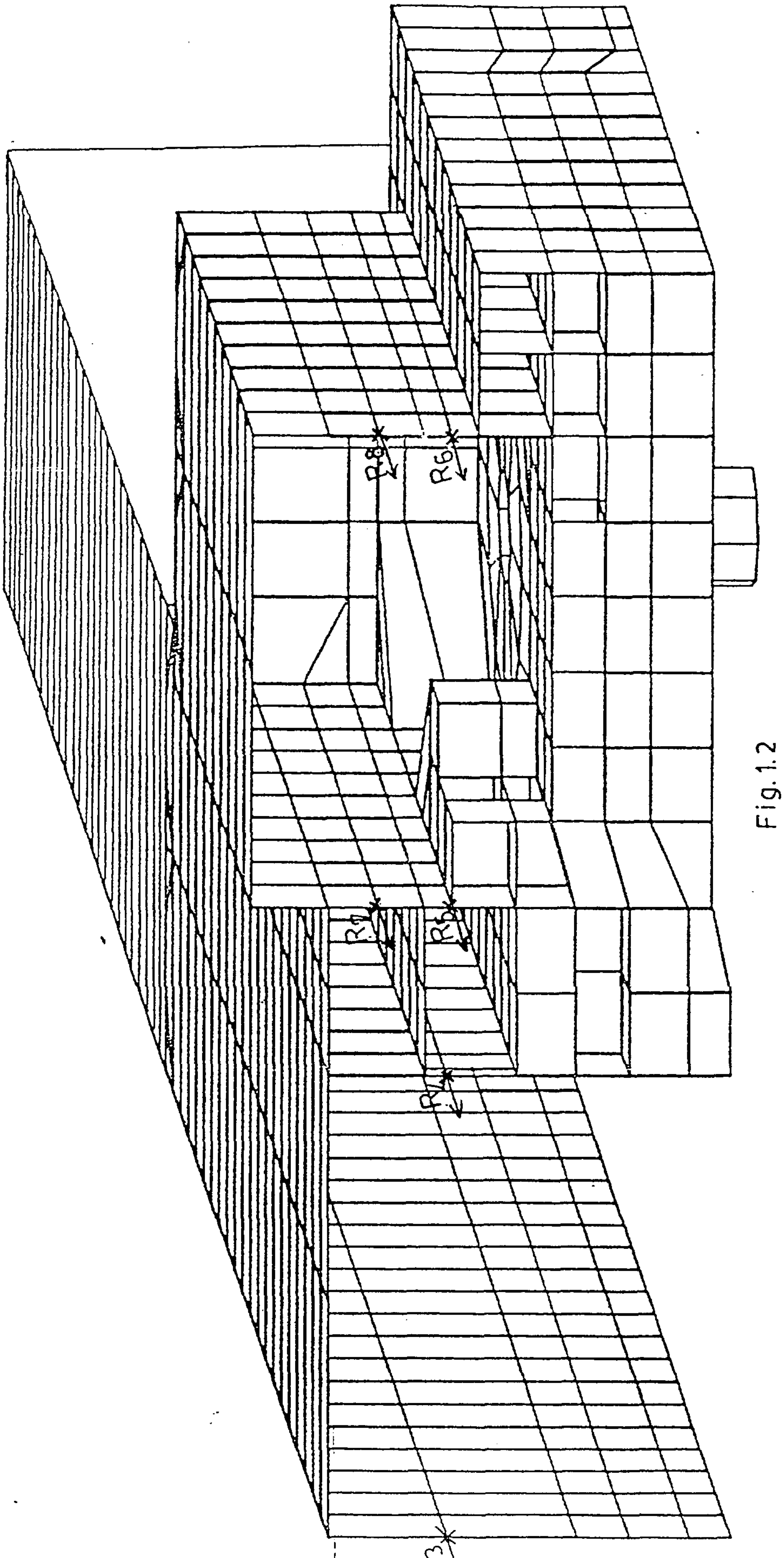


Fig.1.2

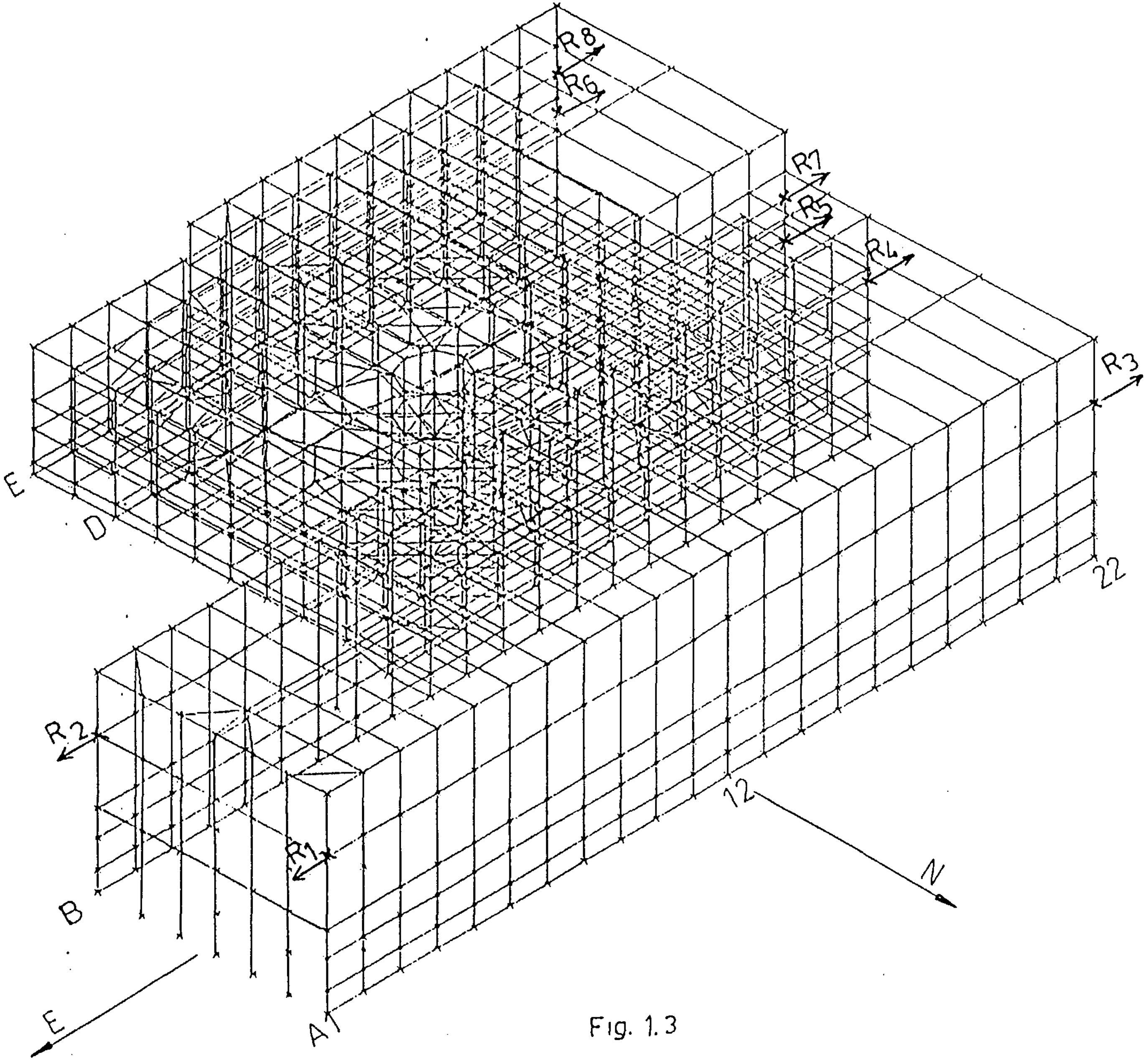
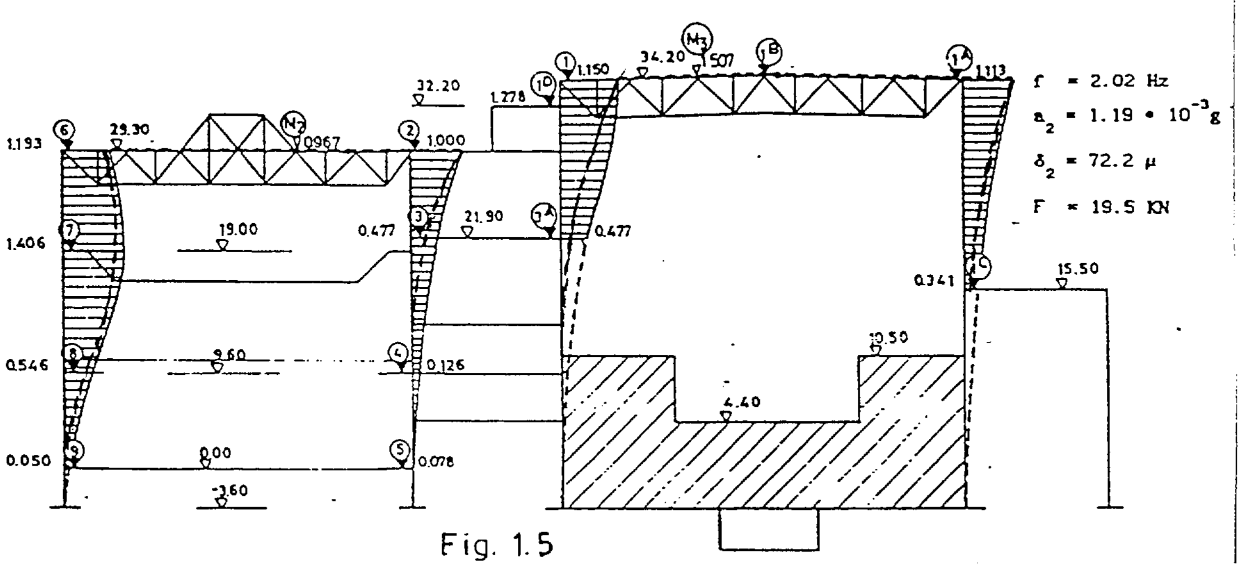


Fig. 1.3



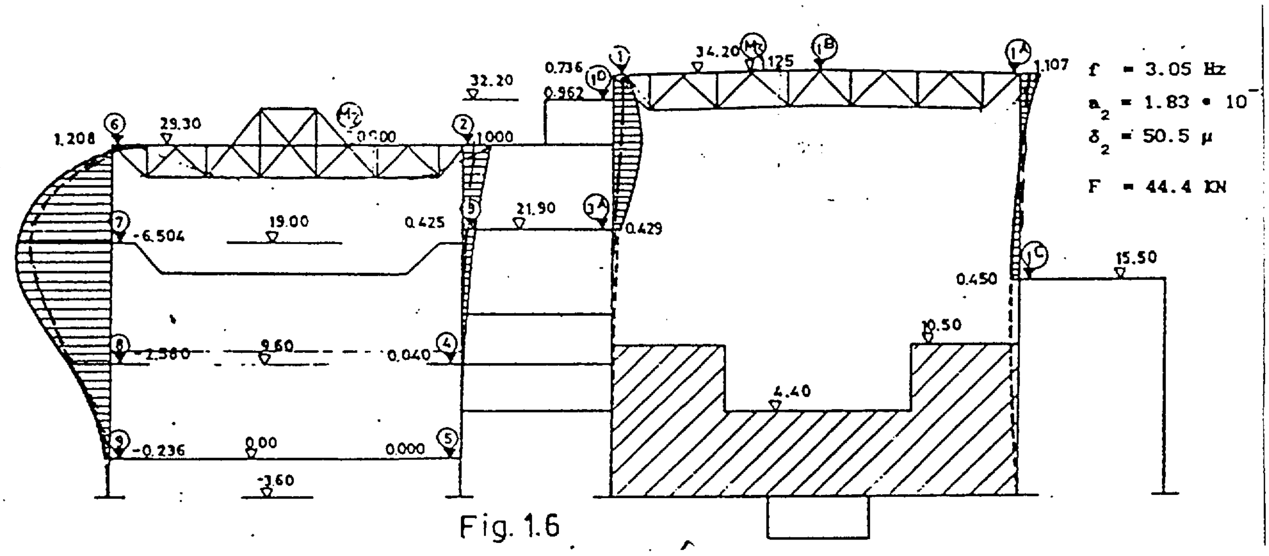
f = 2.02 Hz

$a_2 = 1.19 \cdot 10^{-3}g$

δ_2 = 72.2 μ

F = 19.5 KN

Fig. 1.5



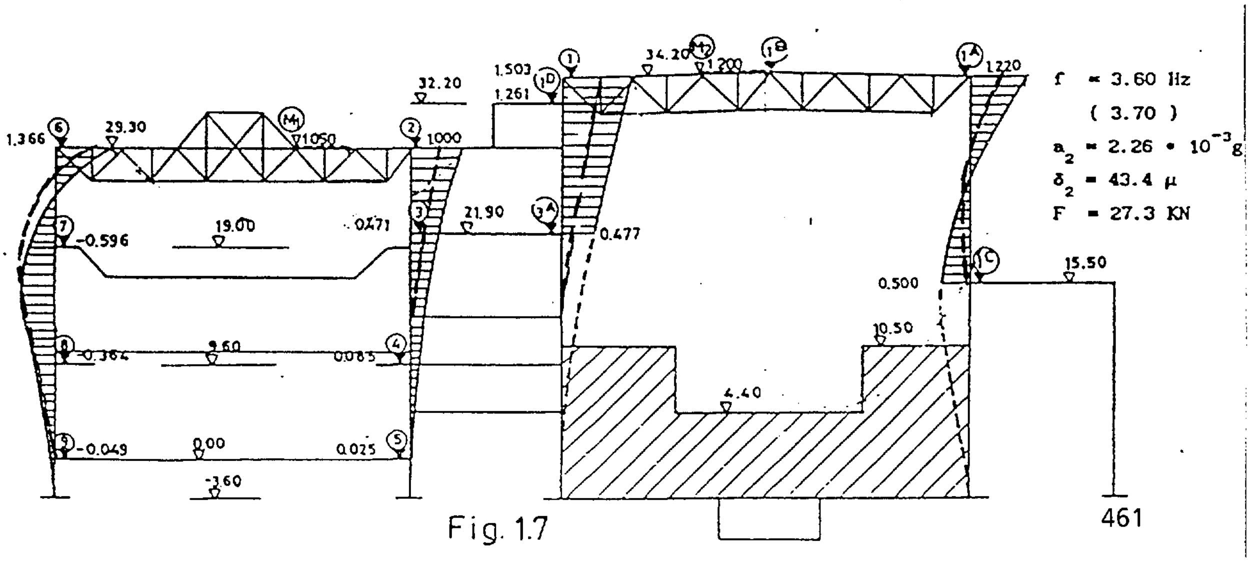
f = 3.05 Hz

$a_2 = 1.83 \cdot 10^{-}$

δ_2 = 50.5 μ

F = 44.4 KN

Fig. 1.6



f = 3.60 Hz

(3.70)

$a_2 = 2.26 \cdot 10^{-3}g$

δ_2 = 43.4 μ

F = 27.3 KN

Fig. 1.7

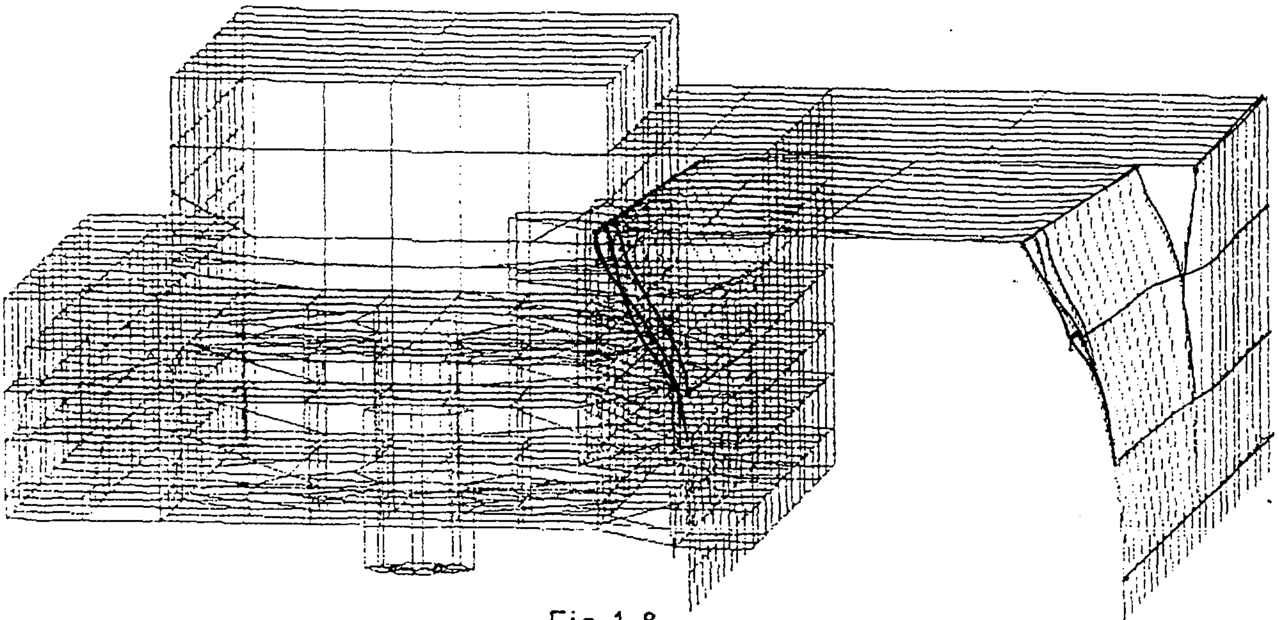


Fig. 1. 8

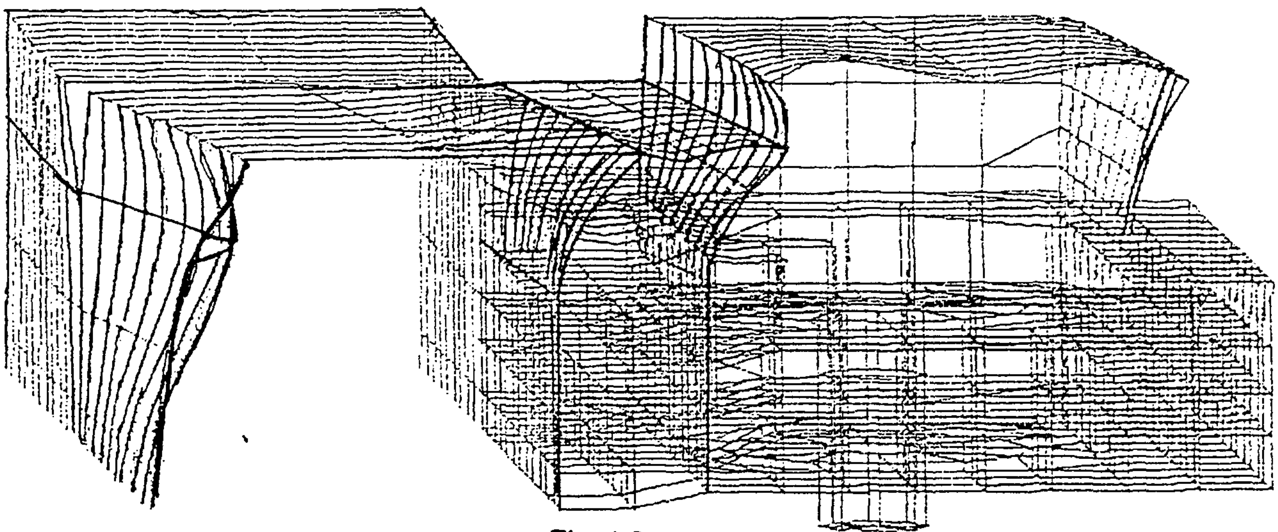


Fig. 1.9

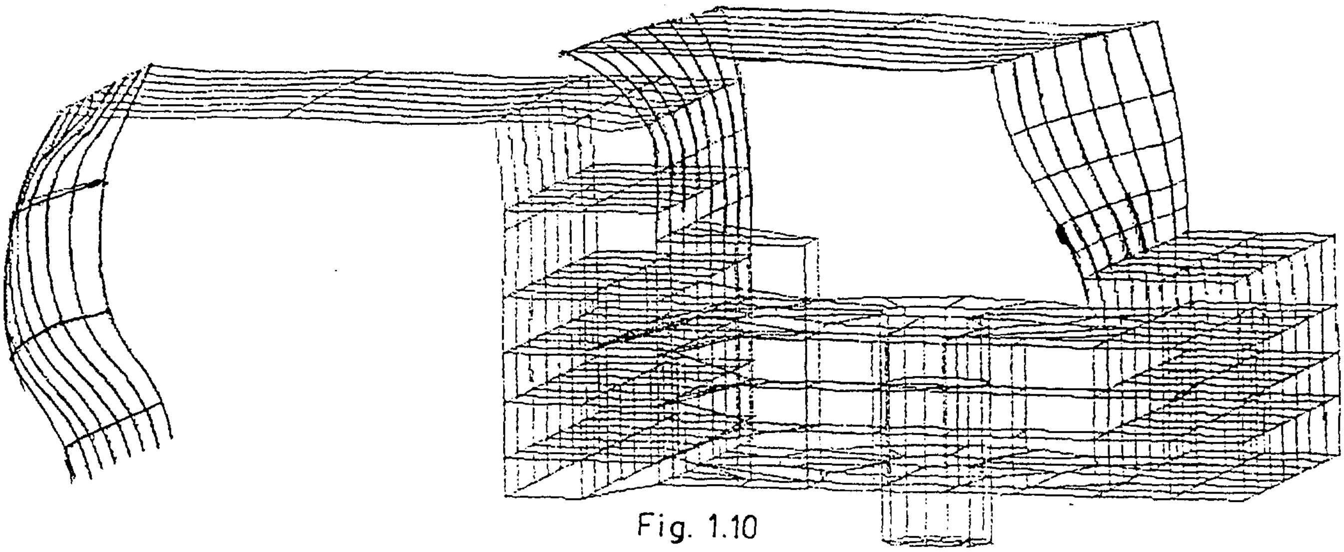


Fig. 1.10

Annex 2

Seismic Response and Resistance Capacity of NPP Kozloduy 440 MW Units

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1. INTRODUCTION

On the base of the defined and verified by experiments 3D matematical models of 440 MW NPP Kozloduy units [Annex 1], and the established 3 - component design accelerograms scaled to 0.204g, the seismic response, seismic safety margins in structure and resistance capacity of 440 MW units were analysed.

2. RESPONSE OF 440 MW UNITS

The response of 3D models were analysed by SAP-90 computing code for investigation of :
a.) influence of irregularities in rigidities (in plane and elevation) on response of different frames and on torsional effects;
b.) influence of technological connections between substructures and neighbouring units on total response;
c.) influence of soil-structure interaction on response.
Output data for absolute and relative accelerations, velocities, displacements and respective response spectra of each nodal point, as well as load and seismic forces - normal, shear, bending and torsional moments for each element can be derived in function of time durring the earthquake.

2.1. Influence of irregularities in rigidities on response

From analyses the following results were obtained :
a.) High frequency mode of vibration influence significantly the response of particular parts of structure. For example, for point 1099 maximum response in N-S direction is formed from modes No 4, 30 and 53. The response of point 1147 (reactor slab) is influenced significantly from 17 Hz frequency which corespond to natural frequency of reactor foundation determined experimentaly. 2D models can't reflect those phenomena.
b.) Spectral contain of floor response spectra for given level is different for frames with different rigidities : A comparision between axis 12 and 16 of 5% spectrum of accelerations (SA) in transversal direction for the roof is given on fig.2.1. On fig 2.2 is given a comparision between rows A and B of 5% SA in longitudenal direction for crane level.
c.) Influence of different rigidities in transversal (T) and longitudenal (L) directions on zero response accelerations (ZRA) is significant. The differances between ZRA of frames on axes 12 and 22 is near to twice. This is result from combined effects of different rigidity and rotation. An example of differencies in ZRA for axes 12 and 16 in transversal direction is given on fig. 2.6.

2.2. Influence of the technological connections between units on response

Response of two types 3D models were investigated :
- "designed" (D) model - without connections in delatational gaps;
- "as built" (B) model - with connections in delatational gaps.

8 crane rails connecting unit 3 and neighbouring units 2 and 4 were modeled by two-component spring constants determined from reaction of neighbouring units at loading with single forses.

From analyses the following conclusions were made :

a.) Technological connections changes the first 9 natural frequancies of the complete structure and decreeses the response;

b.) Connections reduce the maximum accelerations specialy in turbine hall axes 1 - 11. An example for axis 3 is given in table 2.1;

Table 2.1 Influence of Connections on Maximum Acceleration

axis/level	0.00	3.25	7.97	14.10	22.10	31.90
A free	0.207	0.215	0.298	0.348	0.500	0.405
A conn.	0.207	0.208	0.270	0.379	0.347	0.348
B free	0.207	0.227	0.375	0.472	0.583	0.405
B conn.	0.207	0.210	0.305	0.435	0.499	0.348

c.) Connections modifies the response spectra for some parts of structure : On fig.2.3 are compared the response spectra of reactor building at level of rail connections with unit 4 calculated with "B" and "D" models. On fig.2.4 are compared the response spectra in turbine hall at connections with unit 2.

d.) The most significant influence of the connections on response and bearing capacity is between the water tour reservoir, turbine hall and deareators. This problem is under detailed analysis.

e.) In table 2.2 are given the maximal relative displacements of points at two sides of joint in axis 12 for two levels, and comparisoin between the two variants.

Table 2.2 Relative Displacements In Joints

joint	level 31.90 - roof				level 22.10 - rails			
	axis A		axis B		axis A		axis B	
	tran	long	tran	long	tran	long	tran	long
NO Conn. [sm]	8.06	6.97	8.11	5.87	5.01	5.96	0.63	1.18
Rails [sm]	6.49	3.82	6.41	3.86	0.16	0.004	0.06	0.004
Ratio	1.24	1.82	1.26	1.52	31.3		10.5	

It's seen, that for the roof the rails decreeses relative displacements 25% in transversal and 50 to 80 % in longitudenal directions. There weren't any damages in the rails connections during Vrancha '77 earthquake with maximum acceleration 0.1g. From the analyses we calculated that they will not be destroid from earthquake with A_{max}= 0.2g.

2.3 Influense of soil-structure interaction on response

The influence of site soil conditions on response were investigated by :

1. Modifying the input 3 axial accelerogram through the soil profil from free field to the foundation level using computer program SHAKE. The 5% SA for free field and for foundation level (-5.50) are given on fig.2.5;
2. Three-axial spring constants for the foundation nodes;
3. Equivalent damping in structure.

To analise the influence of soil-structure interaction on response were compared Model "B" on rigid base, with design 3 axial accelerogram, and Model "S" - base on springs, with modified 3 axial accelerogram.

Soil-structure interaction decreeses the complete response of structure about 5 to 25 % acording to the relative rigidity - more significetly the response of relatively rigid parts of structure - up to elevation 14.10 and axis 12. On fig.2.6 are compared the maximum accelerations of axis 12 and axis 16 for Model "B" and "S".

The 5% SA in transversal direction on elevation 14.10 (plate of reactor), and on reactor roof for both models, are compared on figure 2.7. On fig.2.8 are compared those, for elevation 22.10 and reactor roof. There are significant changes in frequancy content, espesialy for level 14.10 due to SSI. For the roof the frequency content is similar, but the values of main peeks are decreesed.

3. ANALYSES OF SEISMIC SAFETY MARGINS

Seismic safety margins were investigated for defining seismic level for automatic shutdown of reactors. This is the reason the refferance level of earthquke excitation to be taken SL2 = 0.20g for structural seismic safety margins. For equipment qualification the results from walkdown missions HCLPF criteria and fragility curves analyses were used [4]. During Vranchea '77 earthquake the maximum acceleration was near to 0.1g without demages in equipment. From analises HCLPF for some component is 0.04g, wich shows high conservatism in analyses.

Seismic safety margins for structural critical components - columns in turbine hall were investigated with 3D models and 3-axial accelerogram (model "B").

The limit curves of column bearing capacity are determined in function of normal force P and bending moments (Mx and My). The seismic forces are determined in function of time during the excitation taking into consideration the dead load :

$$P(t) = Pdl + Peq(t) \qquad Mx(t) = Mxdl + Mxeq(t) \qquad My(t) = Myeq(t)$$

From the analysis we calculated that for the columns up to level 22.10 during the excitation the limit curves were not exceeded. The water tour reserwoir significantly increese seismic forces in columns of row B axis 1 to 10 and at crane level (22.10) the limit curves in longitudenal direction were exceeded . On fig 2.9 are shown the time history of seismic margins in one element of axis B level 22.10.

It can be seen, that in some moments of earthquake the bending moments are over the limit curves and columns are in nonelastic stage.

4. CONCLUSIONS

From the analyses of verified 3D models the following conclusions were made :
a.) The irregularities in structural rigidities in plane and elevation leads to significant torsion and differencies in response on given level. Only 3D model can refflect this pecularities;
b.) The technological connections between substurctures and neighbouring units reduce the complete response and have to be considered;
c.) The soil-structure interaction significantly influence the response of structures and their components. More detailed analyses of this problem are in progress.
d.) The analyses of seismic safety margins were performed in time domain, during the excitation. At crane level row B seismic forces exceed the limit curves of column bearing capacity. Additional reinforcement of some critical components is needed.

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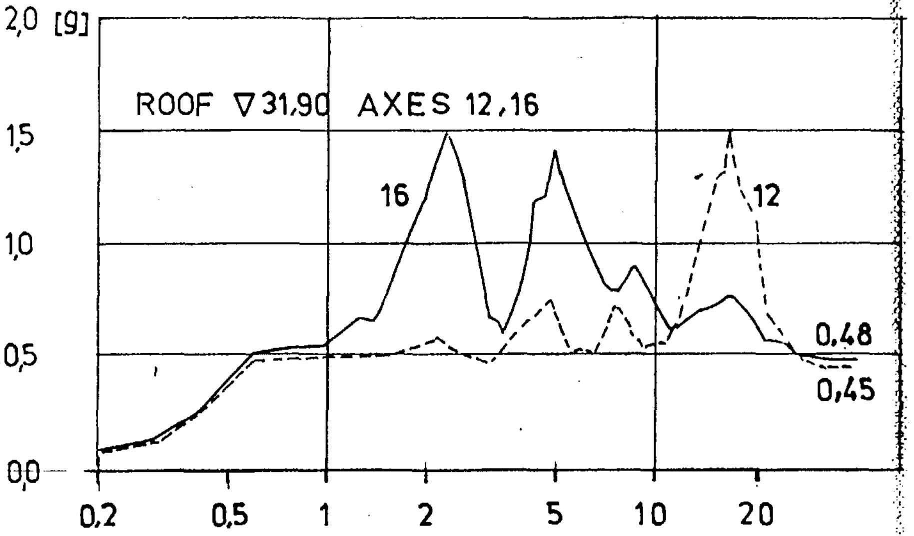

Fig. 2.1

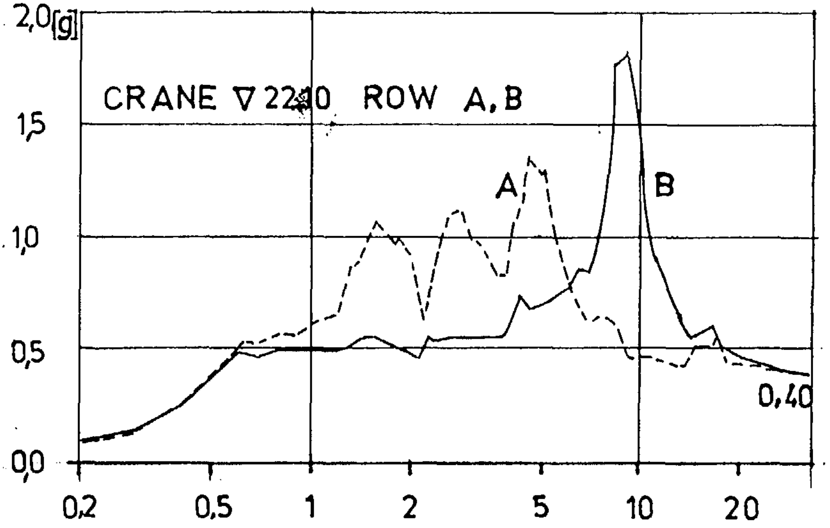

Fig. 2.2

RESPONSE SPECTRA

SA – 5 %

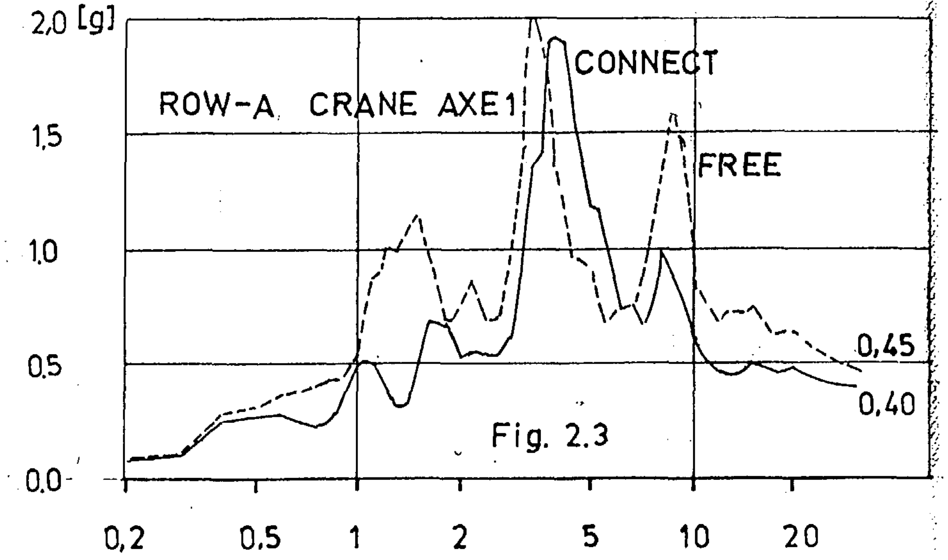

Fig. 2.3

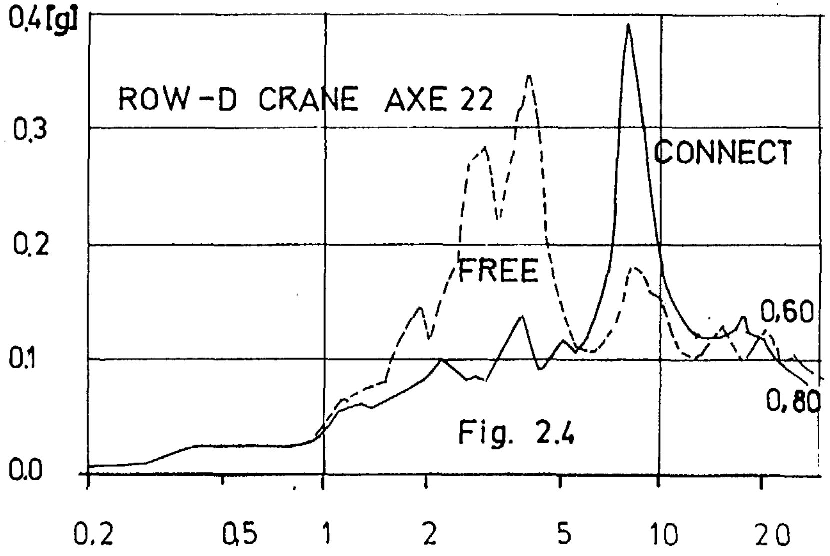

Fig. 2.4

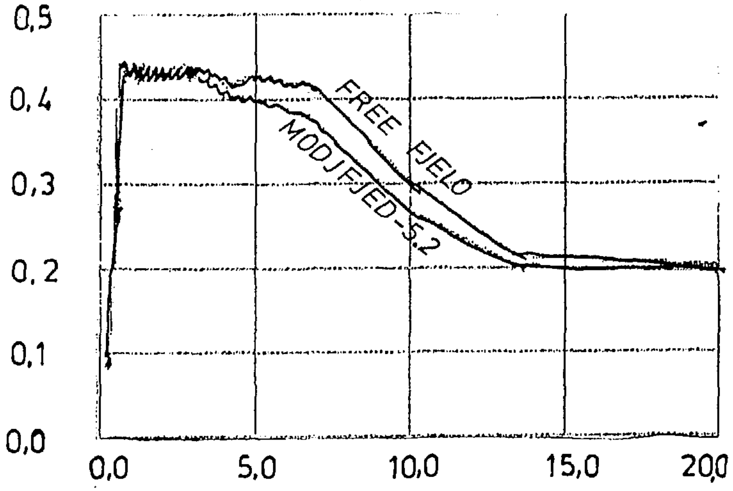

Fig. 2.5

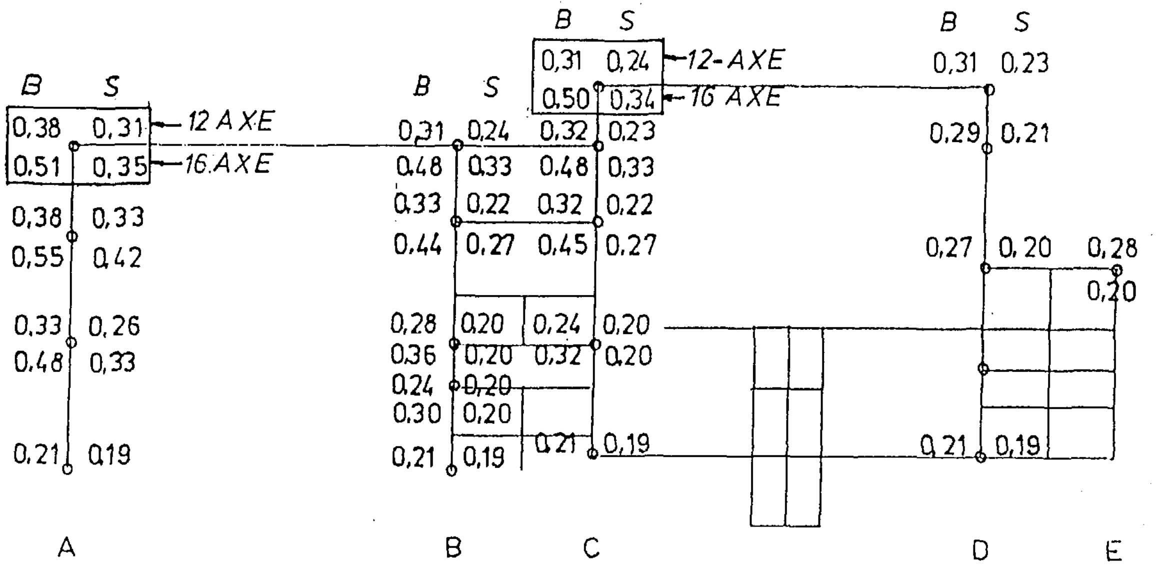

Fig. 2.6

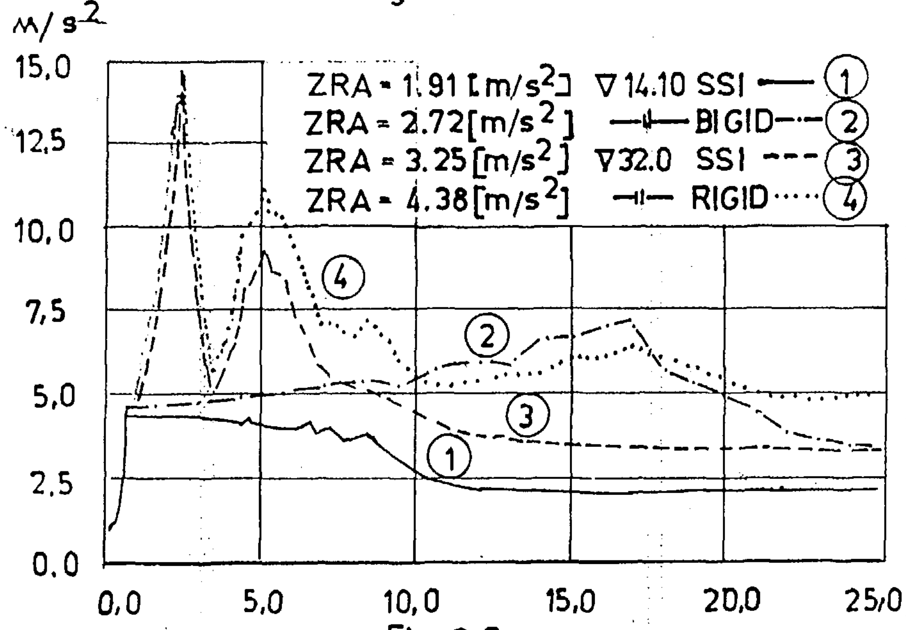

Fig. 2.7

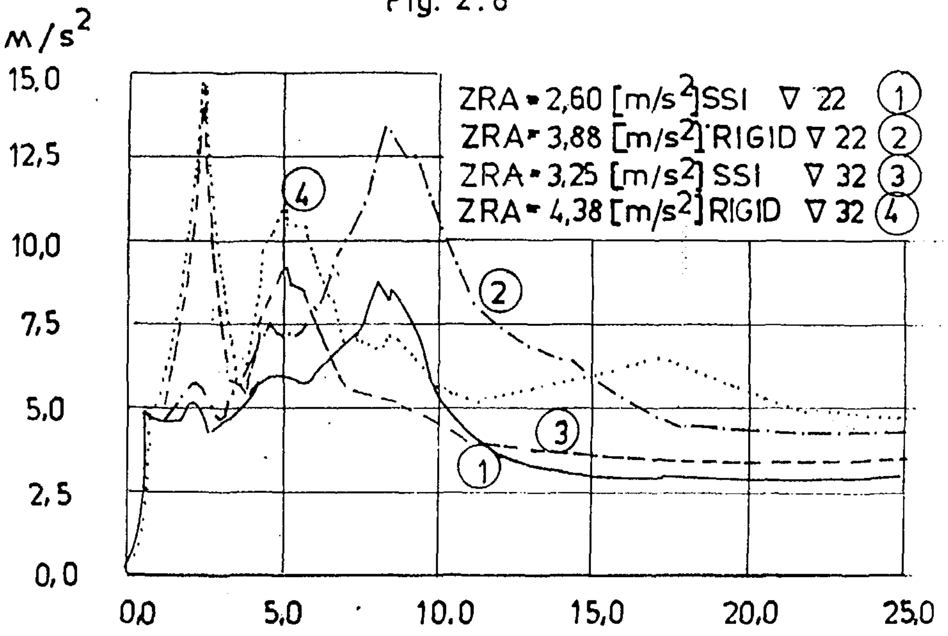

Fig. 2.8

TIME HISTORY OF SEISMIC MARGINS

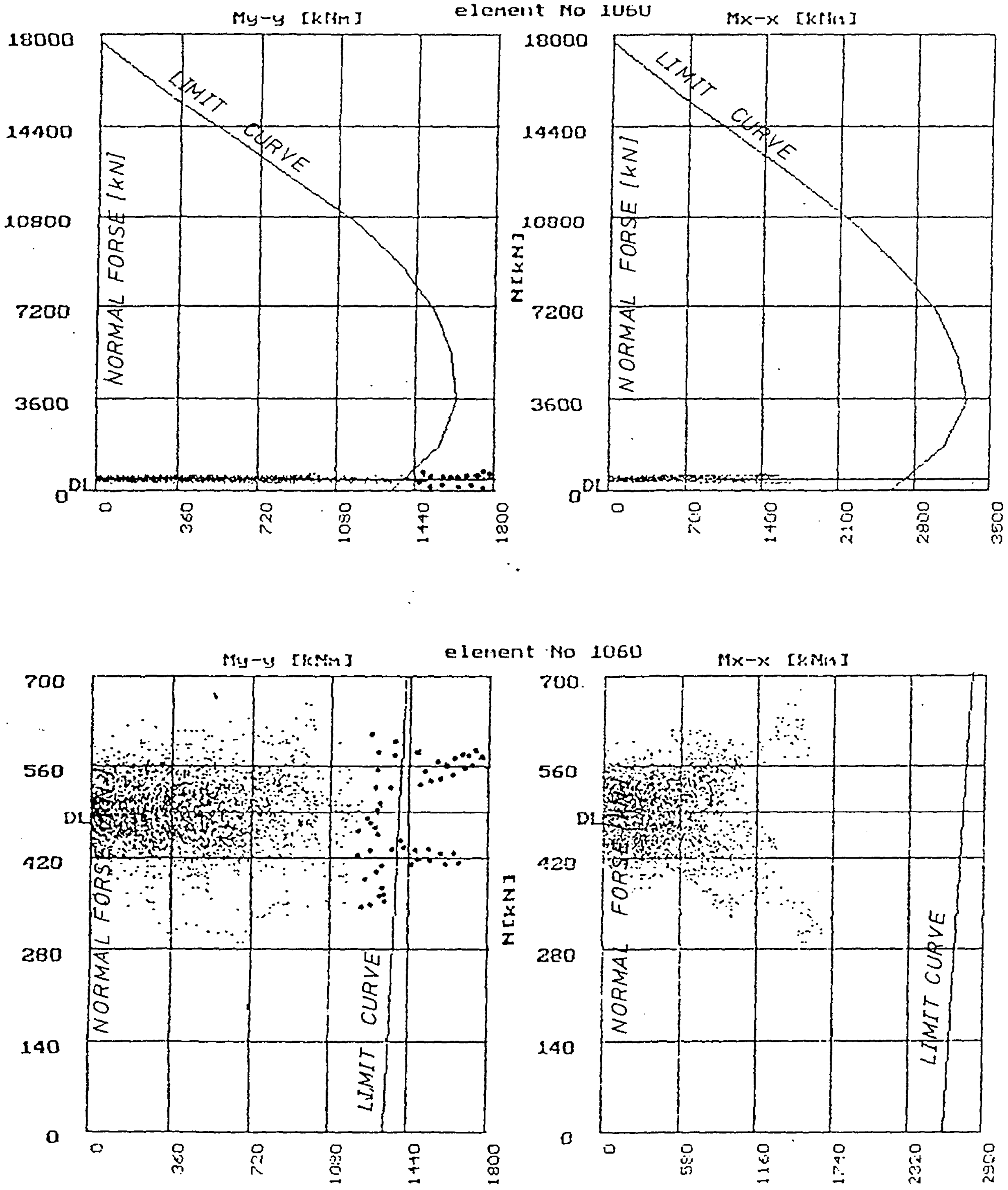

Fig. 2.9