1N9300922

BARC/1992/E/032

BARC/1992/E/032

सत्यमेव जयते

SEISMIC ANALYSIS OF TWO 1050 mm DIAMETER HEAVY WATER UPGRADING TOWERS FOR 235 MWe KAIGA ATOMIC POWER PLANT SITE

by

R. S. Soni, H. S. Kushwaha, S. C. Mahajan and A. Kakodkar
Reactor Engineering Division
and
Suresh Narwaria, T. G. Vardarajan and H. K. Sadhukhan
Heavy Water Division

1992

BARC/1992/E/032

BARC/1992/E/032

GOVERNMENT OF INDIA
ATOMIC ENERGY COMMISSION

SEISMIC ANALYSIS OF TWO 1050 mm DIAMETER HEAVY WATER UPGRADING TOWERS FOR 235 MWe KAIGA ATOMIC POWER PLANT SITE

by

R.S. Soni, H.S. Kushwaha, S.C. Mahajan and A. Kakodkar
Reactor Engineering Division

and

Suresh Narwaria, T.G. Vardarajan and H.K. Sadhukhan
Heavy Water Division

BHABHA ATOMIC RESEARCH CENTRE
BOMBAY, INDIA

1992

BARC/1992/E/032

BIBLIOGRAPHY DESCRIPTION SHEET FOR TECHNICAL REPORT

(as per IS : 9400 - 1980)

01	Security classication :	Unclassified
02	Distribution :	External
03	Report status :	New
04	Series :	BARC External
05	Report type :	Technical Report
06	Report No. :	BARC/1992/E/032
07	Part No. or Volume No. :	
08	Contract No. :	
10	Title and subtitle :	Seismic analysis of two 1050 mm diameter heavy water upgrading towers for 235 MWe Kaiga Atomic Power Plant site
11	Collation :	52 p., 12 figs., 18 tabs.
13	Project No. :	
20	Personal author (s) :	(1) R.S., Soni; H.S. Kushwaha; S.C. Mahajan; A. Kakodkar; (2) Suresh Narwaria; T.G. Vardarajan; H.K. Sadhukhan
21	Affiliation of author (s) :	(1) Reactor Engineering Division, Bhabha Atomic Research Centre, Bombay; (2) Heavy Water Division, Bhabha Atomic Research Centre, Bombay
22	Corporate author(s) :	Bhabha Atomic Research Centre, Bombay - 400 085
23	Originating unit :	Reactor Engineering Division, BARC, Bombay
24	Sponsor(s) Name :	Department of Atomic Energy
	Type :	Government
30	Date of submission :	September 1992
31	Publication/Issue date	October 1992

contd...(ii)

40 Publisher/Distributor : Head, Library and Information Division, Bhabha Atomic Research Centre, Bombay

42 Form of distribution : Hard Copy

50 Language of text : English

51 Language of summary : English

52 No. of references : 14 refs.

53 Given data on :

60 Abstract : This report deals with the analysis carried out for the evaluation of earthquake induced stresses and deflections in two 1050 mm diameter heavy water upgrading towers for Kaiga Atomic Power Plant Site. The analysis of upgrading tower has been carried out for two mutually perpendicular horizontal excitations and one vertical excitation applied simultaenously. The upgrading towers have been analysed using beam model taking into account soil-structure interaction. Response spectrum analysis has been carried out using site spectra for 235 MWe Kaiga site. The seismic analysis has been performed for both the towers with supporting structure alongwith concrete pedestals and raft foundation. The towers have been checked for its stability due to compressive stresses to avoid buckling so that the nearby safety related structures are not geopardised in the event of SSE loading.

70 Keywords/Descriptors : HEAVY WATER PLANTS; EARTHQUAKES; STRESSES; STRESS ANALYSIS; SOIL-STRUCTURE INTERACTIONS; SEISMIC EFFECTS; MECHANICAL STRUCTURES; MECHANICAL VIBRATIONS; MATHEMATICAL MODELS; FINITE ELEMENT METHOD; S CODES; STABILITY; THREE-DIMENSIONAL CALCULATIONS; DEFORMATIONS; DYNAMIC LOAD; WIND

71 Class No. : INIS Subject Category : E1300; D1200

99 Supplementary elements :

CONTENTS

ABSTRACT

This report deals with the analysis carried out for the evaluation of earthquake induced stresses and deflections in two 1050 mm diameter heavy water upgrading towers for Kaiga Atomic Power Plant Site.

The analysis of upgrading tower has been carried out for two mutually perpendicular horizontal exitations and one vertical excitation applied simultaneously. The upgrading towers have been analysed using beam model taking into account Soil-structure interaction. Response spetrum analysis has been carried out using site spectra for 235 MWe Kaiga site.

The seismic analysis has been performed for both the towers with supporting structure alongwith concrete pedestals and raft foundation. The towers have been checked for its stability due to compressive stresses to avoid buckling so that the nearby safety related structures are not geopardised in the event of SSE loading.

Seismic Analysis of Two 1050 mm Diameter Heavy Water Upgrading Towers For 235 MWe Kaiga Atomic Power Plant site

By

R.S.Soni, H.S.Kushwaha, S.C.Mahajan, A.Kakodkar

Reactor Engineering Division

And

Suresh Narwaria, T.G.Vardarajan, H.K.Sadhukhan

Heavy Water Division

Bhabha Atomic Research Centre

Bombay-400 085

1.0 Introduction

This report deals with the seismic analysis of two identical single Heavy Water upgrading towers of 235 MWe Kaiga Atomic Power Plant. Each Upgrading tower is made up of a column sump, 14 column sections, a reflux condenser, a vent condenser and a reboiler. The total height of the single tower is 56 meters and its weight is 70 Tons (approx.). Weight of the structure which supports both the towers is 264.2 tons (approx.) and the weight of concrete pedestals and raft taken together is 1143 tons (approx.). The upgrading towers are required to be designed for OBE (Operating Basis Earthquake) and shall be checked against total collapse under SSE (Safe Shutdown Earthquake) considering the height of the distillation columns so that the nearby safety related structures do not suffer any kind of damage (Ref.10).During the course of analysis, it was agreed that the design of distillation columns (e.g. thickness of column sections) would not be altered , and it would be qualified seismically by evolving the supporting structure of the towers in such a way that the induced seismic stresses are within the allowable limits. This is an economical proposition because the towers are made up of SS 304L. Analysis of the towers was performed in various steps which are as follows:

(1) Modelling of towers

(2) Analysis Of Supporting Structure

(3) Evaluation of equivalent model of supporting structure,

(4) Modelling of raft, concrete pedestals & soil structure interaction

(5) Analysis of towers with supporting structure,

(6) Checking the stability of tower and

(7) Checking the effect of wind load on tower

(8) Calculation of stresses.

This report gives details of all these steps, results of analysis and the conclusions arrived at.

2.0 Description of The Upgrading Plant

The upgrading plant at Kaiga consists of two numbers of distillation towers along with its accessories and the supporting structure. The distillation columns and the supporting structure rest on concrete pedestals which in turn rest on a common raft foundation .Both the distillation columns at their bottom have a sump which gets supported on a skirt. Part of the sump support (Skirt) is made out of S.S 304 L and the rest is made out of IS2002 Gr 2A material.

Each distillation tower consists of 14 number of column sections containing tower internals. Each column section is of 1050 mm I.D. and 3250 mm height fabricated out of S.S 304 L plates (Ref. Fig.1). The first three column sections are made out of 18 mm thick plate while the remaining eleven column sections are made out of 12 mm thick S.S. plate. Each tower has its own column sump ,packed sections, reboiler,reflux condenser and vent condenser(Ref. 12). Each column section weighs about 4.0 Tons. The column section are of flanged type which are seal welded after erection. The column sump is of 1400 mm O.D.,14 mm thick and 2000 mm in height. Column sump and skirt assembly weighs around 8.2 Tons. The reboiler is connected at the bottom and condensers are located on the top of the distillation tower. The weight of the reflux condenser is 5.0 Tons. The sump water while passing through the vertical thermo-syphon reboiler,gets vapourised due to steam heating . All the vapours get condensed in the condensers and are returned as reflux. Both the vapour and the liquid reflux flowing counter currently get exchanged their heavier and lighter isotopes . Thus D2O concentration is enriched to the required grade in the sump. The tower is supported at the bottom and in addition it is provided with five numbers of lateral supports at various elevations of the supporting structure with the reqired gap between the support and the column section.

3.0 Mathematical Modelling

3.1 Modelling of Towers

Lumped mass beam modelling has been used in the analysis.The first step in lumped mass beam modelling is to convert the real structure into a system of lumped masses and segments suitable for mathematical analysis. The response of tower to a given excitation is determined by its mass, stiffness and its interaction with the foundation.

Finite element model of distillation columns along with supporting structure was made so as to represent mass and stiffness properties from base to top. An assemblage of beam elements with straight centroidal axis and of uniform cross section connected at structural joints with masses concentrated at selected structural joints designated as nodes has been used for discretising the towers. The characteristics of a lumped mass model are described by the masses lumped at nodal points, segment height, moment of inertia, area and shape factor at the mid section of each segment. The portion between the lumped masses is assumed to have uniform cross-section.

The FEM model of both the towers and their supporting structure consists of 216 nodes and 218 elements (segments) as shown in Fig.2. Nodes 1 to 85 have been used to discretise the tower-1 and nodes 130 to 214 have been used to discretise the tower-2, nodes 86 to 129 have been used to represent framed supporting structure and the rest of the nodes are used for discretising the civil structure.

3.2 Modelling of Supporting Structure

The supporting structure has got 4 built up columns. Each column is a built up section made of two ISMC 400 with 10 to 25 mm thick plates welded to their flanges (Ref.13). In the plan, these columns are connected by four horizontal ISMB 300 beams at various levels as shown in Fig.3. In the elevation these beams are connected by cross bracings made up of angles ISA 150 x 150 x 12, ISA 150 x 150 x 10 , ISA 130 x 130 x 10 and ISA 100 x 100 x 8 . The complete structure rests on 4 concrete pedestals which in turn are resting on raft foundation as shown in Fig.4. These four concrete pedestals and tower pedestals are joined by rectangular concrete beams in the plan at EL. -1000. , EL. -4350. and EL. -7650.0 . The height of supporting structure is 57.927 meters.

The supporting structure, therefore, is a 3-D space frame type of structure which consists of a number of beams and columns and supports both the towers. Modelling of such a space frame type of structure alongwith the tower for carrying out the dynamic analysis is not feasible because the problem would then become too complex requiring large computer memory space and computer time. In order to solve this problem , an equivalent beam model of the supporting structure has been evolved which has been used alongwith the towers for the purpose of dynamic analysis. A 3-D space frame model of supporting structure (Fig.5) has been subjected to static loading in three directions at its top elevation. The deflections & rotations caused by these loadings at various levels have been subsequently used for arriving at the geometrical properties of the equivalent beam model. Computer code SAP-IV (Ref.1) has been used for the analysis .For the evaluation of properties of the equivalent beam model, x-direction loads of 100 kg each have been applied at nodes 1, 3, 4, 5, 7 and 8 respectively (Fig.5) For Y-direction, the loads of 100 kg each have been applied at nodes 1, 2, 3, 5, 6 and 7 respectively. For evaluating the properties in vertical direction, load of 100 kg each has been applied at nodes 1, 3, 5 and 7. For finding out the polar moment of inertia of the equivalent beam, a torque of 400 kg-mm has been applied at the center of the structure at top level.

In order to validate the properties , the equivalent beam model as shown in Fig. 6.0 has been analysed for the same static loadings as the 3-D space frame model of the supporting structure and the results of both the models have been compared at some selected elevations as shown in Table 1.0. This ensures that the equivalent beam model truly represents the behaviour of the actual supporting structure.

3.3 Modelling of Raft, Concrete Pedestals and Soil-Structure Interaction

Having established the mathematical model for the tower and its supporting structure, it is necessary to include the modelling of concrete pedestals and the raft on which they rest. The concrete pedestals and the raft have been modelled as beam elements with appropriate geometrical properties. To evaluate the properties of concrete pedestals, it is modelled as a 3-D space frame model has been used (Fig.7). This 3-D space frame model has been subjected to static loading in three direction at its top elevation. The deflections and rotations caused by these loadings at various levels have been used to find out geometrical properties of equivalent beam model of concrete pedestals (Fig.8). Table-2 shows the comparision between 3-D space frame model and equivalent beam model for deflections and rotations for the same static loadings. Masses of concrete pedestals and raft have been lumped suitably at various nodes. For this purpose, the weight density of concrete has been assumed to be 2.54 T/cu.m.

The foundation conditions may have considerable effect on the response depending upon the nature of the soil and extent of its interaction. The soil-structure interaction has been given due credence in the formulation of the mathematical model by introducing the frequency independent linear soil springs at the base . The soil at the base has been represented by rotational springs KTX and KTY , transtational springs KX and KY, vertical spring KZ and torsional spring KT whose stiffnesses have been evaluated on the basis of half space theory (Ref. 2 and Ref. 3) as given below:

$$KX \text{ or } KY = 2(1+\nu)\, G\, \beta_x \sqrt{BL},$$

$$KTX \text{ or } KTY = G/(1-\nu)\, \beta_\psi \, . \, BL^2$$

$$KZ = G/(1-\nu)\, \beta_z \sqrt{BL}, \quad \text{and}$$

$$KT = 16\, GR^3/3,$$

where,

ν = Poisson's ratio of foundation medium,

G = Shear modulus of soil, $= \frac{\rho}{g} V_s^2$

R = $\sqrt[4]{BL\,(B^2 + L^2)/6\pi}$ = radius of equivalent circular raft,

ρ = Weight density of soil,

B = width of basemat perpendicular to the direction of horizontal excitation,

L = length of basemat in the direction of horizontal excitation.

β_x, β_ψ, β_z = constants that are function of dimensional ratio, L/B, which are found using Fig.3300-3 of Ref.2

The foundation properties used for 235 MWe Kaiga site are given below:

V_s = Shear wave velocity of rock = 1600 m/sec

ν = Poisson's ratio of rock = 0.15

ρ = Weight density of rock = 2.5 T/cu.m

The effect of side soil has been neglected in the analysis because the back fill soil is loose in nature (Ref.11) . Table 3.0 shows the values of soil spring stiffnesses used in the model. Boundary elements of code SAP-IV (Ref.1) have been used for modelling these stiffnesses as shown in Fig. 2. The soil damping for translational ,rocking and torsional modes has been assumed to be that of the super-structure which would give conservative results

4.0 Method of Analysis

Finite element computer code SAP-IV(Ref.1) has been used for the analysis. The analysis was carried out for both the distillation towers alongwith their supporting structure. Response spectrum method of analysis has been used to qualify the towers during SSE and OBE events. The ground motion response spectra used for 235 MWe Kaiga site (Ref.11) are as shown in Fig.9.0 and Fig. 10.0 . The OBE acceleration values at various frequencies have been taken as half of the SSE values. The structural damping value of 3% of the critical damping has been used for SSE conditions and 2% of critical damping has been used for OBE conditions for towers ,steel supporting structure,concrete pedestals and soil (Ref.4). For the purpose of dynamic analysis, static value of modulus of elasticity has been used as given in ASCE standard (Ref.2). All the six degrees of freedom viz. δx, δy, δz , Θx, Θy and Θz are allowed at each node. The connections between the tower and its supporting structure at various levels have been established by coupling the two horizontal translational degrees of freedom (x and y) of the two structures through the use of end release code option available in the computre code SAP- IV.

In order to carry out the response spectrum analysis, the programme first calculates the undamped natural frequencies and mode shapes of the structure. Having obtained the frequencies and mode shapes, it does the response calculation for each mode using modal superposition method. Material damping has been assumed to be same in all the modes. The total response is then obtained by combining the responses from various modes either using square root of sum of squares (SRSS) method or ten percent method depending upon the closeness of the modes (Ref.5). First sixty modes have been selected for the purpose of analysis so as to excite all the modes upto 33 Hz. The total mass participation in various directions upto 60 modes is as shown in Table- 5 The rigid body mode response due to rest of the mass (called missing mass) has been evaluated using equivalent static method and has been combined with the dynamic response in an SRSS manner. The spatial combination has been performed using SRSS method (Ref.5).

5.0 Check for Stability of the Tower

The D2O upgrading towers would be subjected to axial compressive stresses due to its self weight. In addition to this, the seismic loading would also give rise to axial and bending type compressive stresses. The tower, therefore, needs to be checked for its stability due to these compressive stresses so as to avoid its buckling. The stability check for the tower is required to ensure that the nearby located safety related structures are not damaged in the event of an SSE loading. In order to carry out this check, it is first necessary to evaluate the buckling load for the tower under various modes of failures and then perform the check as per the relevant design code.

The tower is designed as per ASME Code Section VIII, Div.1 (Ref.6). This code, although, requires the evaluation of seismic stresses in the vessels, it does not give a clear cut methodology for performing the stability check. A survey of various available design codes to arrive at a suitable decision was therefore made. As per the design practices given in various codes, a thin cylinder under axial compression may fail in one of the following three ways.

(i) By plastic yielding when the stress in the material reaches the yield stress i.e. fc = fy , where fc is the critical buckling stress and fy is the yield strength of material.

(ii) By buckling of the complete cylinder as a strut (Eulers buckling). This would happen when induced stress reaches the critical stress given by:

$$fc = (k1E \pi^2 r^2) / (2 L^2) ,$$

where, k1 = critical stress factor which depends upon the supporting conditions of the vessel,

E = Young's modulus,

r = external radius of vessel &

L = length of vessel.

(iii) By local buckling with the formation of axial and circumferential stress waves on the surface. This would occur when the induced stresses reach the critical buckling stress given by:

$$fc = k2\ E\ t\ /\ \sqrt{3(1 - \nu^2)}\ .\ r ,$$

where, k2 = a constant determined from chart C-47 as given in the code.(Ref.7)

In the case of combined bending and direct axial stresses, the highest compressive stress occuring in a cylinder subjected to an axial load W and a bending moment M is given by:

$$fm = W / (2 \pi r_m . t) + M / (\pi r_m^2 t)$$

The maximum stress fm should be limited to the lowest critical stress found as above divided by an appropriate safety factor. As per ASME code VIII (Ref.9), the allowable compressive stresses shall be the least of

(i) 25 % of critical buckling stress with a factor of 50 % for tolerance

(ii) 50 % of specified minimum Sy and at the temperature , where Sy is the yield strength of the material

(iii) 100 % of avarage stress to produce a creep rate of 0.01 % per 1000 hours

(iv) 100 %of the allowable stress in tension

The failure modes due to Euler's buckling and local buckling normally require very high compressive stresses to be developed in the tower. This fact was studied as given in Ref. 8 . It was concluded from this study that the buckling mode caused due to plastic yielding is the one which governs the design and therefore the allowable compressive stress values corresponding to SY/2 has been used for checking the tower against buckling.

6.0 Effect Of Wind Load On Distillation Tower

Analysis for wind load has been performed by M/S Chemicon. M/S Chemicon has evaluated the distribution of wind pressure along the height of steel supporting structure and has designed it for this loading. Since the distillation towers are connected to the steel supporting structure at five locations, there would be a transfer of loads to the towers due to wind loading. This effect could not be simulated by M/S Chemicon because their model includes only steel supporting structure and does not include mathematical representation of distillation towers. It was therefore, decided to use the equivalent forces generated due to wind loading at various levels of steel supporting structure in present model and evaluate the effect of these forces on the distillation towers only. The design adeqacy of steel supporting structure for wind loading has been checked by M/S Chemicon. The wind forces used in the analysis are given in Table-15 (Ref. 14).

7.0 Results

Table- 4 shows the frequencies & modal participation factor of first sixty modes of vibration for two towers along with supporting structures. In order to reduce the stresses in the tower, analysis with five supports was carried out. Table- 7 and Table- 6 show the deflection and stresses at salient locations of tower for SSE loadings respectively. Table- 9 and Table- 8 shows deflection and stresses at salient locations of tower for OBE loadings respectively. Table- 5 shows the mass participation in various directions, Table- 10 and Table-11 show the lateral forces transferred on the supporting structure at the five support locations for SSE and OBE loadings respectively. Table-13 and Table-14 respectively show the seismic forces and moments in the steel structure concrete pedestals and raft for SSE and OBE loading. Fig.11 and Fig.12 respectively show the mode shape for first two modes of vibration for towers supported at 5 locations to the supporting structure. Table- 12 shows the reaction forces/moments at the base of each tower. Table-16 and Table-17 respectively show the stresses and deflection in the tower for wind loading.

7.0 Discussions

(1) Appropriate modelling of tower and its supporting structure plays a key role in the seismic qualification of the towers. Modelling of complete structure alongwith the towers is rather time consuming and uneconomical from the point of view of computer memory and time. An equivalent model of the structure has been, therefc ` evolved whose behaviour is a true representation of the actual structure, as shown by results given in Table 1.0. This is a very important step in this anlaysis which decides the accuracy of the analysis performed. Modelling of soil-structure interaction alongwith the concrete pedestals and raft also are important in this respect.

(2) The tower has been qualified with five lateral supports without altering its design which is desirable from the point of economy and at the same time ensure the safety of other safety related structures located nearby by making sure that the tower alongwith its supporting structure is quite stable.

9.0 Conclusions

(1) From the various studies conducted above it is concluded that in order to have minimum induced stresses in the towers in the event of SSE earthquake, they shall be supported at five locations along their height from the supporting structure. It is, therefore, recommended to use the five-supports tower design as a standard one.

(2) A close look at the mode shapes of the tower alongwith its supporting structure reveals that the tower and the supporting structure both behave as a single cantilever beam in the first two modes. The first frequency of the system is 0.7043 Hz which is a bending type of mode. Response of the complete system is predominantly governed by the first ten modes. In order to ensure the convergence of response and excite all the modes up to 33 Hertzs, sixty modes have been considered for the analysis

(3) The maximum deflections in X, Y and Z directions at the top of tower (with five supports) are 14.13 cm, 13.83 cm and 0.2 cm respectively whereas at the top of supporting structure are 14.67 cm, 15.1 cm and 0.30 cm respectively.

(4) The maximum stress levels as in various sections of the tower are kept well below the allowable limits so as to avoid failure due to instability. The maximum induced stresses due to weight and seismic loading(SSE) and missing mass in the tower shell is 9420 psi for the case

of five supports along the height of the tower at the bottom of section -14 . However, it is worth mentioning here that the stresses induced due to all other loadings such as pressure, temperature etc shall also be combined with these stresses to check the tower for stability.

(5) The maximum stress in the tower for the case of wind load is 5369.0 psi and the maximum deflection at the top of tower is 20.34 cm ,22.04 cm and 0.25 cm in X , Y and Z direction respectively. Table-18 gives forces /moments at the base of steel structure for wind loadings

(6) The base shears and base moments induced due to seismic event for each tower are as given in Table-12. The seismic forces and moments in steel tower,concrete pedestals and raft are given in Table-13 and Table-14 respectively. It is worth mentioning here that the civil structures shall be designed for these forces and moments.

(7) The reaction forces transferred at the five lateral supports to the supporting structure are as given in Table-10 and Table-11 for SSE and OBE respectively. The design of supporting structure shall take care of these forces

10. References

(1) SAP-IV: Structural Analysis Program-IV, K.J.Bathe & Wilson.

(2) Standard for the Seismic Analysis of Safety Related Nuclear Structures, American Society of Civil Engineers, 1986.

(3) Whitman, R.V., Soil-Structure Interaction, Seismic Design of Nuclear Power Plants, R.J. Hansen, MIT, England, 1972.

(4) ASME Code Section III Division 1, Appendices, Appendix N, Dynamic Analysis Method, 1986.

(5) USNRC Regulatory Guide 1.92, Combining Modal Responses and Spatial Components in Seismic Response Analysis, 1976.

(6) ASME Boiler and Pressure Vessel Code Section VIII, Div.1, 1986.

(7) IS-2825:1969: Design of Unfired Pressure Vessels, 1969.

(8) Seismic Analysis Of Single 1500 MM Dia. Heavy Water Upgrading Tower For 500 MWe & Kaiga Site

(9) Structural Analysis and Design of Process Equipments, Mann H. Jawad and James R. Farr, 1984.

(10) Letter No. KAIGA/34930/92/B/1050 April 7, 1992 by Shri T.B. Nandwani ACE (WM and UP) 235 MWe NPC.

(11) Letter No Kaiga- 1,2/34930/92/B/1708 minutes of meeting on D2O upgrading plant and distillation tower,dated June 9, 1992.

(12) Drawing Nos.KAPP-1/34930/2008/GA Distributor Rev.0 , KAPP-1/34930/2008/GA Details & Assly. of column sump Rev.0 , KAPP- /34930/2560/DD Top vent cond. Rev-0 , KAPP-/34930/2559/DD Reboiler Rev-0, KAPP-1&2/34930/2557/DD Reflux cond. Rev-0, KAPP-1&2/34930/2558/DD Details of Reflux Cond. Rev-0, KAPP-1&2/34930/2575/DD Details of Column section Rev-0, KAPP-1/34930/2005/GA Vapour Hood Rev-0,

KAPP-1/34930/2009/GA Liquid Collector Rev-0, KAPP-1/34930/2007/GA Bottom & Top support Ring Rev-0.

(13) Drawing No. Kaiga 1,2 /29950/800/GA Structural Steel Work Elevation and Plan General Arrangement

(14) Letter from M/S Chemicon to shri D. K. Gandhi, Head, Upgrading Plant Section NPC, Annexure-1 ,dated Feb. 17,1992

TABLE-1

COMPARISION OF DEFLECTION FOR 3-D MODEL AND EQUIVALENT BEAM MODEL FOR SUPPORTING STRUCTURE

ELEVA-TION MM	FX=600.0 KG				FY=600.0 KG				FZ=400.0 KG		MZ=400.0 KG-MM	
	3-D FEM		EQU-BEAM		3-D FEM		EQU-BEAM		3-D FEM	EQU-BEAM	3-D FEM	EQU-BEAM
	DFL-X MM	ROT-Y	DFL-XMM	ROT-Y	DFL-Y MM	ROT-X	DFL-Y MM	ROT-X	DFL-Z MM	DFL-Z MM	ROT-Z	ROT-Z
57827.0	0.1675E1	0.419E-4	0.162E1	0.4292E-4	0.1620E1	0.4266E-4	0.1615E1	0.4292E-4	0.2068E-1	0.2063E-1	0.2412E-7	0.2473E-7
36075.0	0.56971	0.2980E-4	0.5659	0.2885E-4	0.56945	0.2981E-4	0.56599	0.2885E-4	0.6001E-2	0.6052E-2	0.4031E-8	0.4056E-8
23057.0	0.24718	0.189E-4	0.25051	0.1720E-4	0.24720	0.1890E-4	0.24051	0.1720E-4	0.2848E-2	0.2882E-2	0.2438E-8	0.2446E-8

TABLE-2

COMPARISION OF DEFLECTION FOR 3-D MODEL AND EQUIVALENT BEAM MODEL FOR CONCRETE PEDESTALS & RAFT

ELEVA-TION MM	FX=600.0 KG				FY=600.0 KG				FZ=600.0 KG		MZ=414.4 KG-MM	
	3-D FEM		EQU-BEAM		3-D FEM		EQU-BEAM		3-D FEM	EQU-BEAM	3-D FEM	EQU-BEAM
	DFL-X MM	ROT-Y	DFL-XMM	ROT-Y	DFL-Y MM	ROT-X	DFL-Y MM	ROT-X	DFL-Z MM	DFL-Z MM	ROT-Z	ROT-Z
10500.0	0.105E-1	0.1355E-5	0.10E-1	0.135E-5	0.9591E-2	0.1158E-5	0.959E-2	0.115E-5	0.2358E-3	0.2358E-3	0.2981E-9	0.2950E-9
6150.0	0.462E-2	0.1080E-5	0.46E-1	0.112E-5	0.4385E-2	0.9405E-6	0.422E-2	0.959E-6	0.136E-3	0.1381E-3	0.1927E-9	0.1727E-9
2850.0	0.128E-2	0.683E-6	0.12E-2	0.635E-6	0.1242E-2	0.6176E-6	0.127E-2	0.5435E-6	0.6314E-4	0.6401E-4	0.7796E-10	0.8000E-10

TABLE-3

SOIL SPRINGS USED TO ACCOUNT FOR SOIL STRUCTURE INTERACTION

NODE	KX	KY	KZ	KTX	KTY	KTZ
91	1.686E7	1.775E7	1.979E7	4.835E14	8.274E14	1.111E15

KX= TRANSLATIONAL SPRING IN X-DIRECTION (KGF/MM)

KY= TRANSLATIONAL SPRING IN Y-DIRECTION (KGF/MM)

KZ= TRANSLATIONAL SPRING IN Z-DIRECTION (KGF/MM)

KTX= ROCKING SPRING ABOUT X-AXIS (KGF MM/RAD)

KTY= ROCKING SPRING ABOUT Y-AXIS (KGF MM/RAD)

KTZ= TORTIONAL SPRING ABOUT Z-AXIS (KGF MM/RAD)

TABLE-4

NATURAL FREQUENCIES AND MODAL PARTICIPATION FACTORS FOR

FOR VARIOUS MODES FOR TOWER WITH 5-SUPPORTS

MODE	FREQUENCY HERTZ	MODE PARTICIPATION FACTOR		
		X	Y	Z
1	0.7043	0.3710E-4	0.5720E1	-0.1115E-2
2	0.7282	0.5664E1	-0.3485E-4	0.2040E-2
3	2.799	-0.1260E-1	0.2202	-0.1291E-3
4	2.930	-0.2488E1	-0.1405E-2	0.2495E-2
5	3.115	0.1416E-2	0.3688E1	-0.3215E-2
6	3.376	-0.2762E1	0.1565E-3	0.96[illegible]2E-2
7	6.879	0.3004E1	0.6463	0.5220E-1
8	6.932	0.6613	-0.1927E1	0.1583E-1
9	7.190	0.6500	-0.3522E-1	0.7962E-1
10	7.279	-0.1[illegible]26	-0.4639	-0.3060E-2
11	7.544	0.7770E-1	0.3263E-2	0.6804E-2
12	7.56[illegible]	0.8543E-1	-0.3929	-0.1059E-3
13	7.792	0.3216E-1	-0.1882E1	0.5656E-2
14	8.142	-0.8368E-2	0.8741E-3	0.4621E1
15	9.326	0.1836E1	-0.3929E-1	-0.8738E-1
16	9.715	0.1836	-0.8689	-0.3908E-2
17	9.848	0.2941E1	-0.4044E-1	0.1619
18	10.30	0.6777E-1	0.3469E1	-0.2090E-1
19	10.69	0.6938	0.3438	-0.6334E-1
20	11.39	0.3089	0.2180E-1	0.3716E1
21	11.54	0.1337E-2	0.9521	0.1742E-3
22	12.42	0.8884E-1	-0.4062	-0.8828E-2
23	12.55	-0.2862E1	-0.1356E-1	0.2548
24	13.91	-0.3494E-1	-0.2657E1	0.2203E-2
25	14.57	-0.1437E1	-0.2545E-1	-0.1380
26	14.84	0.1590E-2	0.1282	-0.3307E-2
27	15.40	0.4007E-1	-0.6132E-1	0.3754E-2
28	15.49	-0.1400	0.3941	0.6460
29	15.59	-0.1051E1	0.2872E-1	-0.6017E-1
30	15.84	-0.1790	0.1568E-1	-0.4740E-1

CONTD. TABLE-4

MODE	FREQUENCY HERTZ	MODE PARTICIPATION FACTOR		
		X	Y	Z
31	16.48	0.2467E1	-0.5232E-1	-0.1402
32	17.17	-0.6014E-1	-0.2643E1	-0.4593E-1
33	18.01	0.2824E-2	0.1392	0.4893E-2
34	18.74	-0.5875E-1	-0.6449E-3	-0.3255E-1
35	19.23	-0.6233E-1	0.9910E-2	-0.3313E1
36	19.80	-0.9719	0.4607E-2	-0.2831E-1
37	20.19	-0.1228	0.1472E-2	-0.2870E-1
38	21.56	0.2126E-2	-0.4786E-1	-0.1135E-1
39	23.33	0.9034E-5	-0.7475E-1	0.2108E-2
40	23.78	-0.4912	-0.5629E-2	0.4377
41	23.86	0.2292E-1	-0.7023E-1	-0.1941E-1
42	23.87	0.1000E-1	0.4701	-0.1813E-2
43	24.68	-0.7193	0.7520E-2	-0.3526E-1
44	25.40	-0.2276E-1	-0.4529	0.4336E-1
45	26.72	0.2449	-0.5384E-1	0.6468
46	26.89	0.3232E-1	0.7012	0.2594E-1
47	28.77	-0.6382E-1	0.4754E-1	-0.1945E1
48	28.85	0.3538E-1	-0.4450E-1	0.9454
49	29.04	0.1970E-2	-0.2451	-0.1190
50	30.25	-0.1429E-1	-0.4019E-1	0.4956E-2
51	30.49	-0.4506E-1	0.4749	0.2044E-1
52	31.10	-0.6365E-1	0.1699	0.5391E-1
53	31.25	-0.4557	0.1284E-3	0.1623E1
54	31.31	0.3342E-1	-0.1605E-1	0.2179E1
55	31.39	0.3271E-1	0.4114E-2	0.9940E-1
56	31.83	-0.1486E1	0.1764E-1	-0.1693
57	33.39	-0.1059E1	0.9468E-1	-0.5243
58	34.22	0.8689E-1	-0.5421E-1	-0.1094E-1
59	34.50	0.6949E-2	0.1439	-0.1036E-1
60	35.22	-0.1509	-0.5512	0.2374E1

TABLE-5

MASS PARTICIPATION IN VARIOUS DIRECTIONS FOR THE TOWER WITH 5-SUPPORTS
(FOR 60 MODES)

CASE	% MASS PARTICIPATION IN VARIOUS DIRECTION		
	X	Y	Z
TOWER WITH 5-SUPPORTS	57.30	55.40	40.70

TABLE-6

STRESSES AT SALIENT POINTS OF TOWER WITH 5-SUPPORTS FOR SSE LOADING

LOCATION	TOWER-1			TOWER-2		
	EL.NO	STRESS KG/SQ.MM	STRESS PSI	EL.NO	STRESS KG/SQ.MM	STRESS PSI
SKIRT BASE	1	4.92	7239.00	116	4.86	7148.00
COLUMN SUMP NECK	8	5.90	8681.00	123	5.88	8648.00
REBOILER SUPPORT I-BEAM	11	15.13	22242.00	126	12.64	18593.00
MITRE BEND	13	4.10	6039.00	128	3.60	5292.00
MITRE BEND	16	4.26	6266.00	131	3.67	5403.00
CHANNEL OF REBOILER	18	1.32	1942.00	133	1.15	1701.00
SHELL OF REBOILER	20	0.99	1463.00	135	0.88	1305.00
COLUMN SECTION NO.1 (16.MM THK.)	24	4.51	6643.00	139	4.50	6623.00
COLUMN SECTION NO.2 (16.MM THK.)	26	4.24	6236.00	141	4.24	6242.00
COLUMN SECTION NO.3 (16.MM THK.)	28	3.86	5679.00	143	3.87	5702.00
COLUMN SECTION NO.4 (12.MM THK.)	30	5.36	7882.00	145	5.37	7906.00
COLUMN SECTION NO.5 (12.MM THK.)	32	5.42	7961.00	147	5.43	7995.00
COLUMN SECTION NO.6 (12.MM THK.)	34	5.06	7443.00	149	5.08	7471.00
COLUMN SECTION NO.7 (12.MM THK.)	36	4.74	6981.00	151	4.75	6995.00
COLUMN SECTION NO.8 (12.MM THK.)	38	4.98	7323.00	153	4.99	7343.00
COLUMN SECTION NO.9 (12.MM THK.)	40	4.78	7037.00	155	4.79	7053.00
COLUMN SECTION NO.10 (12.MM THK.)	42	4.04	5944.00	157	4.05	5954.00
COLUMN SECTION NO.11 (12.MM THK.)	44	4.30	6327.00	159	4.30	6323.00
COLUMN SECTION NO.12 (12.MM THK.)	46	3.99	5876.00	161	4.01	5902.00
COLUMN SECTION NO.13 (12.MM THK.)	48	4.35	6407.00	163	4.38	6450.00
COLUMN SECTION NO.14 (12.MM THK.)	50	6.27	9225.00	165	6.40	9420.00
VAPOUR HOOD	52	3.68	5414.00	167	3.79	5586.00
REFLUX CONDENSER NECK	55	4.96	7293.00	170	5.14	7558.00
SHELL OF REFLUX COND.	75	5.55	8170.00	190	5.65	8307.00
CHANNEL OF REFLUX COND.	80	0.24	362.00	195	0.25	375.00
NOZZLE CONNECTION	82	8.56	12586.00	197	8.57	12611.00
SHELL OF VENT COND.	88	1.77	2614.00	203	1.77	2616.00
CHANNEL OF VENT COND.	90	0.125	184.00	205	0.127	187.00

SUPPORTS AT ELEVATIONS: EL-13237.00 MM (SECTION-3)
EL-22987.00 MM (SECTION-6)
EL-32737.00 MM (SECTION-9)
EL-42487.00 MM (SECTION-12)
EL-48987.00 MM (SECTION-14)

TABLE-7

DEFLECTIONS AT SALIENT LOCATIONS FOR THE TOWER WITH 5-SUPPORTS FOR SSE LOADING

LOCATION	HEIGHT MM	TOWER-1				TOWER-2			
		NODE	DFL-X MM	DFL-Y MM	DFL-Z MM	NODE	DFL-X MM	DFL-Y MM	DFL-Z MM
TOWER BASE	0.00	1	5.94	6.75	1.60	130	5.94	6.75	1.60
SUMP TOP	5299.0	9	12.83	14.10	1.62	138	12.82	14.10	1.62
SECTION -3 TOP	15049.0	28	30.9	31.75	1.66	157	30.9	31.75	1.66
SECTION -6 TOP	24799.0	34	52.53	52.21	1.77	163	52.55	52.21	1.77
SECTION -9 TOP	34549.0	40	78.23	76.6	1.88	169	78.23	76.60	1.88
SECTION-12 TOP	44299.0	46	109.0	106.4	1.95	175	109.05	106.56	1.97
SECTION-14 TOP	50799.0	50	131.6	128.69	1.99	179	131.6	128.6	1.99
REFLUX COND CENTRE	52586.0	69	138.5	135.3	2.002	198	138.53	135.3	2.002
VENT COND CENTRE	53397.0	82	141.3	135.39	2.006	211	141.4	138.0	2.006
STEEL STRUCTURE TOP	57927.0	129	146.6	151.25	3.07				

TABLE-8

STRESSES AT SALIENT POINTS OF TOWER WITH 5-SUPPORTS FOR OBE LOADING

LOCATION	TOWER-1			TOWER-2		
	EL.NO	STRESS KG/SQ.MM	STRESS PSI	EL.NO	STRESS KG/SQ.MM	STRESS PSI
SKIRT BASE	1	3.27	4817.00	116	3.25	4779.00
COLUMN SUMP NECK	8	3.92	5774.00	123	3.90	5735.00
REBOILER SUPPORT I-BEAM	11	9.33	13716.00	126	7.86	11566.00
MITRE BEND	13	2.54	3738.00	128	2.24	3301.00
MITRE BEND	16	2.57	3779.00	131	2.23	3279.00
CHANNEL OF REBOILER	18	0.81	1202.00	133	0.72	1060.00
SHELL OF REBOILER	20	0.60	896.00	135	0.54	803.00
COLUMN SECTION NO.1 (18.MM THK.)	24	2.99	4407.00	139	2.79	4377.00
COLUMN SECTION NO.2 (18.MM THK.)	26	2.80	4127.00	141	2.80	4124.00
COLUMN SECTION NO.3 (18.MM THK.)	28	2.56	3767.00	143	2.57	3779.00
COLUMN SECTION NO.4 (12.MM THK.)	30	3.55	5220.00	145	3.558	5231.00
COLUMN SECTION NO.5 (12.MM THK.)	32	3.54	5207.00	147	3.557	5221.00
COLUMN SECTION NO.6 (12.MM THK.)	34	3.29	4844.00	149	3.30	4861.00
COLUMN SECTION NO.7 (12.MM THK.)	36	3.07	4524.00	151	3.08	4532.00
COLUMN SECTION NO.8 (12.MM THK.)	38	3.16	4658.00	153	3.17	4669.00
COLUMN SECTION NO.9 (12.MM THK.)	40	3.01	4430.00	155	3.02	4439.00
COLUMN SECTION NO.10 (12.MM THK.)	42	2.55	3753.00	157	2.55	3755.00
COLUMN SECTION NO.11 (12.MM THK.)	44	2.66	3924.00	159	2.66	3920.00
COLUMN SECTION NO.12 (12.MM THK.)	46	2.45	3610.00	161	2.46	3623.00
COLUMN SECTION NO.13 (12.MM THK.)	48	2.64	3888.00	163	2.66	3913.00
COLUMN SECTION NO.14 (12.MM THK.)	50	3.71	5459.00	165	3.79	5573.00
VAPOUR HOOD	52	2.19	3230.00	167	2.26	3329.00
REFLUX CONDENSER NECK	55	2.89	4257.00	170	2.99	4403.00
SHELL OF REFLUX COND.	75	3.23	4752.00	190	3.28	4830.00
CHANNEL OF REFLUX COND.	80	0.15	224.00	195	0.157	232.00
NOZZLE CONNECTION	82	5.01	7378.00	197	5.02	7393.00
SHELL OF VENT COND.	88	1.13	1662.00	203	1.13	1663.00
CHANNEL OF VENT COND.	90	0.07	115.00	205	0.07	117.00

SUPPORTS AT ELEVATIONS: EL-13237.00 MM (SECTION-3)
EL-22987.00 MM (SECTION-6)
EL-32737.00 MM (SECTION-9)
EL-42487.00 MM (SECTION-12)
EL-48887.00 MM (SECTION-14)

TABLE-9

DEFLECTIONS AT SALIENT LOCATIONS FOR THE TOWER WITH 5-SUPPORTS FOR OBE LOADING

LOCATION	HEIGHT MM	TOWER-1				TOWER-2			
		NODE	DFL-X MM	DFL-Y MM	DFL-Z MM	NODE	DFL-X MM	DFL-Y MM	DFL-Z MM
TOWER BASE	0.00	1	3.27	3.72	0.88	130	3.28	3.72	0.88
SUMP TOP	5299.0	9	7.07	7.77	0.89	138	7.07	7.77	0.89
SECTION -3 TOP	15049.0	28	17.06	17.47	0.98	157	17.06	17.47	0.98
SECTION -6 TOP	24799.0	34	28.8	28.6	1.01	163	28.8	28.8	1.01
SECTION -9 TOP	34549.0	40	42.9	42.0	1.04	169	42.9	42.0	1.04
SECTION-12 TOP	44299.0	46	59.8	58.39	1.10	175	59.8	58.3	1.10
SECTION-14 TOP	50799.0	50	72.2	70.56	1.111	179	72.29	70.56	1.111
REFLUX COND CENTRE	52586.0	69	76.1	74.2	1.112	198	76.1	74.2	1.112
VENT COND CENTRE	53287.0	82	77.65	57.7	1.116	211	77.67	75.79	1.116
STEEL STRUCTURE TOP	57927.0	129	80.49	82.99	1.56				

TABLE-10

SEISMIC FORCES TRANSFERED AT LATERAL SUPPORTS ON SUPPORTING

STRUCTURE DUE TO SSE LOADING

ELEVATION mm	LOCATION	Fx Kgf	Fy Kgf
13237.	SECTION-3	0.133E5	0.789E4
22987.	SECTION-6	0.129E5	0.852E4
32737.	SECTION-9	0.113E5	0.770E4
42487.	SECTION-12	0.140E5	0.105E5
48987.	SECTION-14	0.170E5	0.226E5

TABLE-11

SEISMIC FORCES TRANSFERED AT LATERAL SUPPORTS ON SUPPORTING

STRUCTURE DUE TO OBE LOADING

ELEVATION mm	LOCATION	Fx Kgf	Fy Kgf
13237.	SECTION-3	0.547E4	0.450E4
22987.	SECTION-6	0.739E4	0.484E4
32737.	SECTION-9	0.642E4	0.437E4
42487.	SECTION-12	0.809E4	0.599E4
48987.	SECTION-14	0.937E4	0.129E5

TABLE-12

SEISMIC REACTION FORCES AND MOMENTS AT THE BASE OF EACH UPGRADING TOWER
(ELEMENT NO.1 AND ELEMENT NO. 116)

FORCE COMPONENT	VALUE FOR SSE LOADING	VALUE FOR OBE LOADING
Fx	0.103E5 Kgf	0.588E4 Kgf
Fy	0.776E4 Kgf	0.437E4 Kgf
Fz	0.279E5 Kgf	0.168E5 Kgf
Mx	0.342E8 Kgf mm	0.191E8 Kgf mm
My	0.478E8 Kgf mm	0.270E8 Kgf mm
Mz	0.418E8 Kgf mm	0.237E8 Kgf mm

NOTE : X , Y & Z ARE GLOBAL DIRECTION (REF. FIG-2)

TABLE-13

SEISMIC FORCES AND MOMENTS IN STEEL STRUCTURE ,CONCRETE PEDESTALS AND RAFT
DUE TO SSE LOADING

ELEVATION mm	LOCATION	Fx Kgf	Fy Kgf	Fz Kgf	Mx Kgf mm	My Kgf mm	MZ Kgf mm
57927.	ROOF	0.2929E5	0.4120E5	0.2746E5	0.	0.	0.
51857.	STEEL STR	0.293E5	0.412E5	0.275E5	0.260E9	0.178E9	0.
48987.	STEEL STR	0.323E5	0.443E5	0.381E5	0.376E9	0.270E9	0.
45807.	STEEL STR	0.461E5	0.465E5	0.473E5	0.483E9	0.393E9	0.
42487.	STEEL STR	0.437E5	0.475E5	0.558E5	0.559E9	0.515E9	0.
39307.	STEEL STR	0.427E5	0.481E5	0.633E5	0.697E9	0.594E9	0.
36057.	STEEL STR	0.471E5	0.505E5	0.696E5	0.803E9	0.683E9	0.
32737.	STEEL STR	0.502E5	0.504E5	0.746E5	0.918E9	0.788E9	0.
29557.	STEEL STR	0.617E5	0.587E5	0.793E5	0.102E10	0.907E9	0.
26307.	STEEL STR	0.639E5	0.599E5	0.835E5	0.114E10	0.105E10	0.
22887.	STEEL STR	0.647E5	0.604E5	0.870E5	0.127E10	0.122E10	0.
19807.	STEEL STR	0.627E5	0.626E5	0.901E5	0.138E10	0.136E10	0.
16557.	STEEL STR	0.631E5	0.649E5	0.930E5	0.150E10	0.152E10	0.
13237.	STEEL STR	0.637E5	0.672E5	0.956E5	0.164E10	0.169E10	0.
10057.	STEEL STR	0.659E5	0.732E5	0.977E5	0.176E10	0.183E10	0.
6807.	STEEL STR	0.697E5	0.770E5	0.995E5	0.190E10	0.198E10	0.
3770.	STEEL STR	0.734E5	0.810E5	0.101E6	0.205E10	0.213E10	0.
0.	GR. LEVEL	0.767E5	0.842E5	0.102E6	0.225E10	0.234E10	0.
-1000.	CON PEDES	0.855E5	0.101E6	0.117E6	0.236E10	0.245E10	0.256E8
-4350.	CON PEDES	0.125E6	0.129E6	0.121E6	0.261E10	0.268E10	0.256E8
-7650.	CON PEDES	0.151E6	0.155E6	0.128E6	0.291E10	0.297E10	0.256E8
-10500.	RAFT TOP	0.163E6	0.166E6	0.137E6	0.323E10	0.327E10	0.256E8
-12000.	RAFT BOT	0.187E6	0.192E6	0.162E6	0.341E10	0.345E10	0.256E8

NOTE: X AND Y ARE TWO HORIZONTAL DIRECTIONS AND Z IS A VERTICAL DIRECTION
(REFER FIG. NO. 2)

TABLE-14

SEISMIC FORCES AND MOMENTS IN STEEL STRUCTURE ,CONCRETE PEDESTALS AND RAFT
DUE TO OBE LOADING

ELEVATION mm	LOCATION	Fx Kgf	Fy Kgf	Fz Kgf	Mx Kgf mm	My Kgf mm	MZ Kgf mm
57827.	ROOF	0.1677E5	0.236E5	0.140	0.	0.	0.
51857.	STEEL STR	0.168E5	0.237E5	0.151E5	0.144E9	0.102E9	0.
48987.	STEEL STR	0.184E5	0.234E5	0.210E5	0.216E9	0.154E9	0.
45807.	STEEL STR	0.262E5	0.263E5	0.260E5	0.296E9	0.224E9	0.
42487.	STEEL STR	0.246E5	0.268E5	0.307E5	0.340E9	0.292E9	0.
39307.	STEEL STR	0.283E5	0.270E5	0.348E5	0.394E9	0.334E9	0.
36057.	STEEL STR	0.264E5	0.248E5	0.382E5	0.452E9	0.382E9	0.
32737.	STEEL STR	0.281E5	0.283E5	0.409E5	0.515E9	0.439E9	0.
29557.	STEEL STR	0.347E5	0.330E5	0.435E5	0.571E9	0.503E9	0.
26307.	STEEL STR	0.360E5	0.337E5	0.458E5	0.636E9	0.583E9	0.
22987.	STEEL STR	0.364E5	0.338E5	0.478E5	0.708E9	0.673E9	0.
19807.	STEEL STR	0.349E5	0.349E5	0.495E5	0.766E9	0.754E9	0.
16557.	STEEL STR	0.350E5	0.361E5	0.511E5	0.831E9	0.840E9	0.
13237.	STEEL STR	0.354E5	0.374E5	0.525E5	0.904E9	0.931E9	0.
10057.	STEEL STR	0.366E5	0.404E5	0.537E5	0.968E9	0.101E10	0.
6807.	STEEL STR	0.389E5	0.431E5	0.547E5	0.140E10	0.109E10	0.
3770.	STEEL STR	0.411E5	0.454E5	0.554E5	0.112E10	0.117E10	0.
0.	GR. LEVEL	0.430E5	0.473E5	0.556E5	0.124E10	0.128E10	0.
-1000.	CON PEDES	0.539E5	0.571E5	0.655E5	0.130E10	0.135E10	0.148E8
-4350.	CON PEDES	0.711E5	0.735E5	0.679E5	0.143E10	0.147E10	0.148E8
-7650.	CON PEDES	0.862E5	0.878E5	0.714E5	0.161E10	0.164E10	0.148E8
-10500.	RAFT TOP	0.925E5	0.941E5	0.759E5	0.179E10	0.181E10	0.148E8
-12000.	RAFT BOT	0.104E6	0.106E6	0.881E5	0.189E10	0.191E10	0.148E8

NOTE: X AND Y ARE TWO HORIZONTAL DIRECTIONS AND Z IS A VERTICAL DIRECTION
(REFER FIG. NO. 2)

TABLE-15

WIND LOADS APPLIED ALONG THE

HEIGHT OF STEEL STRUCTURE

ELEVATION mm	LOAD Kg
57927.	10600.
51857.	10497.
48987.	7019.
45807.	7480.
42487.	7353.
39307.	7222.
36057.	7086.
32737.	6953.
29557.	6821.
26307.	6652.
22987.	6485.
19807.	6332.
16557.	6082.
13237.	5812.
10051.	5806.
6807.	0.
3770.	0.
0.	0.

TABLE-16

STRESSES AT SALIENT POINTS OF TOWER FOR

WIND LOADING

LOCATION	STRESS Kg/Sq.mm	STRESS PSI
SKIRT BASE	3.65	5369.
SECT-1 (18 Thk.)	3.23	4753.
SECT-4 (12 Thk.)	1.90	2802.

TABLE-17

DEFLECTIONS AT SALIENT LOCATIONS FOR THE TOWER WITH 5-SUPPORTS FOR WIND LOADING

LOCATION	HEIGHT MM	TOWER-1				TOWER-2			
		NODE	DFL-X MM	DFL-Y MM	DFL-Z MM	NODE	DFL-X MM	DFL-Y MM	DFL-Z MM
TOWER BASE	0.00	1	9.202	10.72	2.586	130	9.202	10.72	2.586
VENT COND CENTRE	53297.0	82	203.44	220.46	2.586	211	203.43	220.4	2.586
STRUCTURE BASE	0.00	86	9.191	10.72	0.000				
STRUCTURE TOP	57927.0	129	224.4	242.25	0.00				

TABLE-18

FORCES AND MOMENTS AT THE BASE OF

STEEL STRUCTURE FOR WIND LOADING
(ELEMENT NO. 98)

FORCE COMP.	VALUES
Fx	103400. Kgf
Fy	102900. Kgf
Fz	0. Kgf
Mx	3.735E9 Kgf mm
My	3.349E9 Kgf mm
Mz	0. Kgf mm

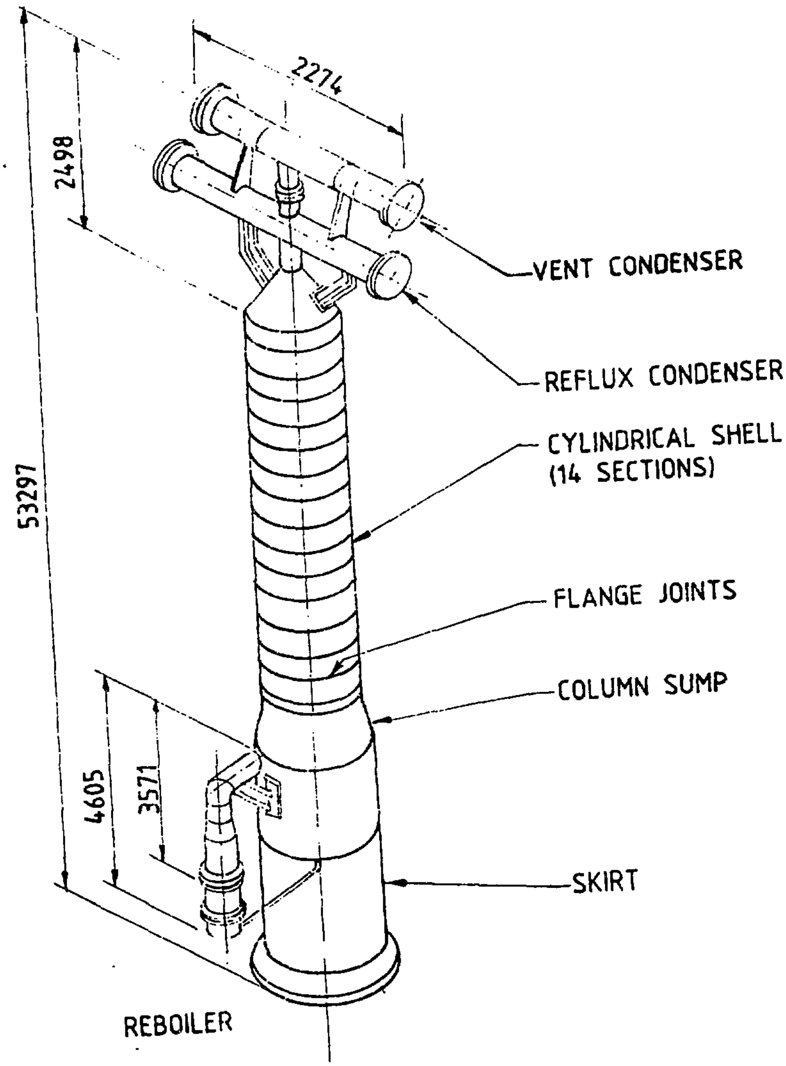


Fig. 1 ISOMETRIC SKETCH OF SINGLE D_2O UPGRADING TOWER

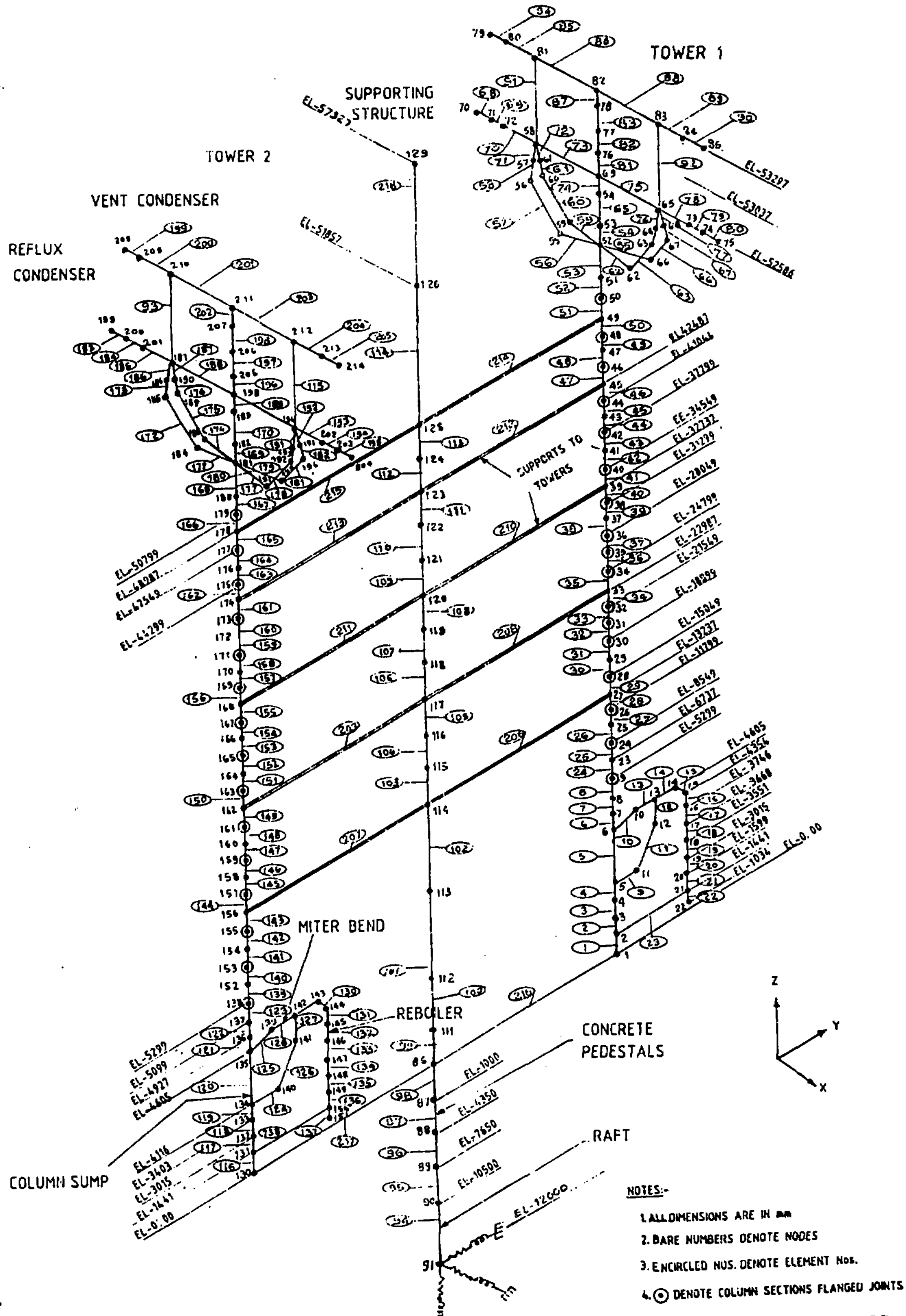


Fig. 2 F.E.M. ' ,UEL OF TWO DISTILLATION TOWERS WITH SUPPORTING STRUCTURE

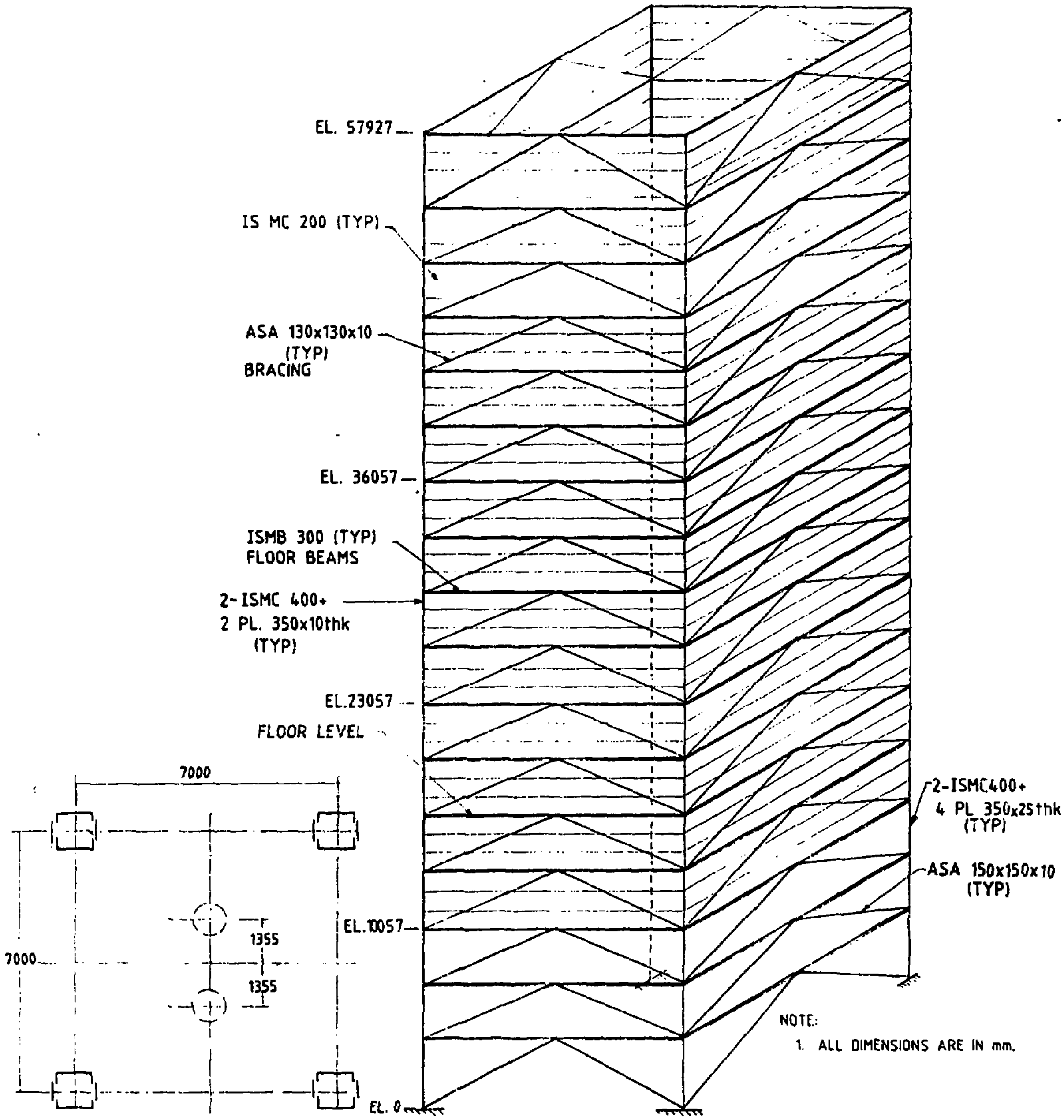
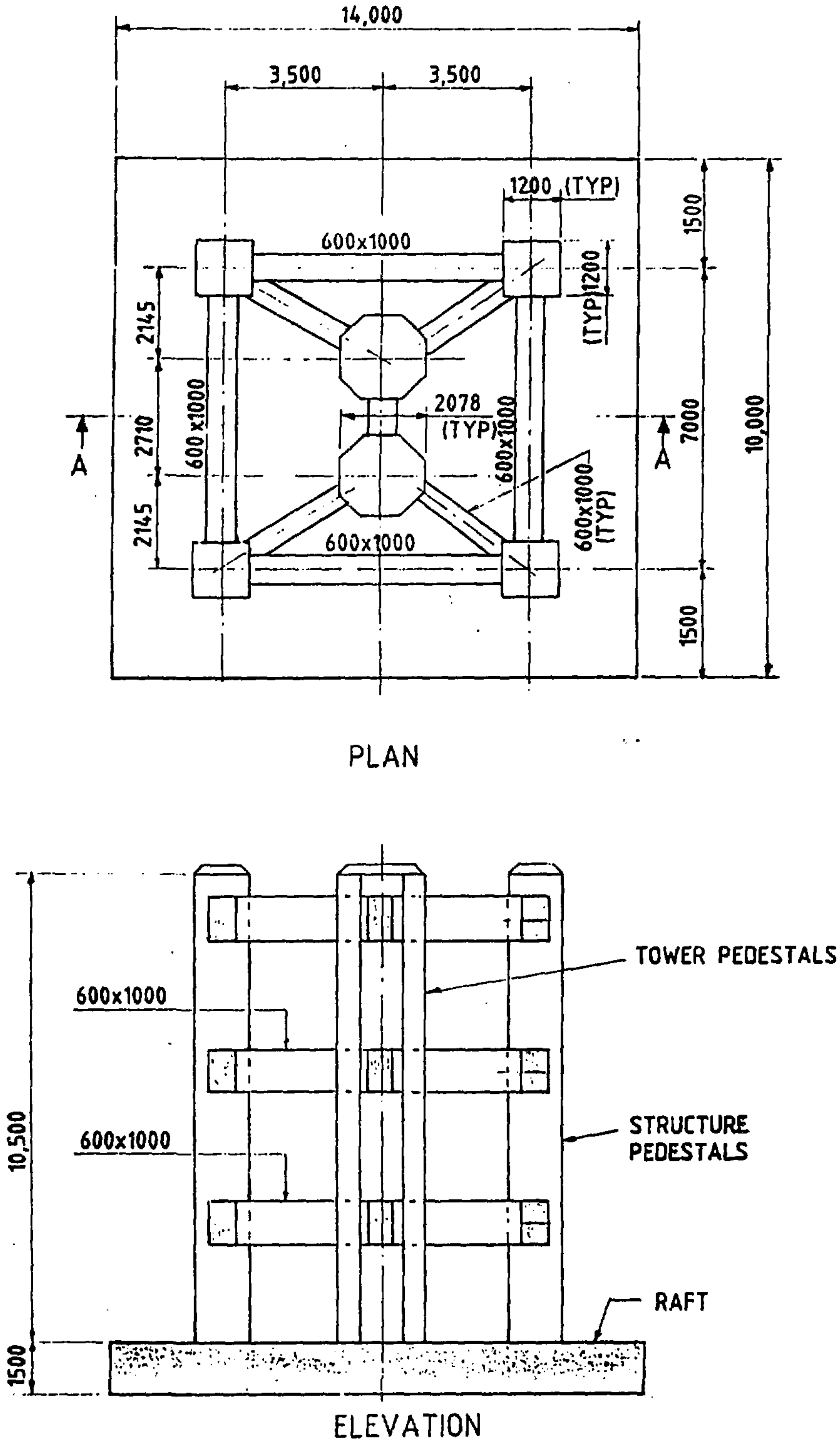
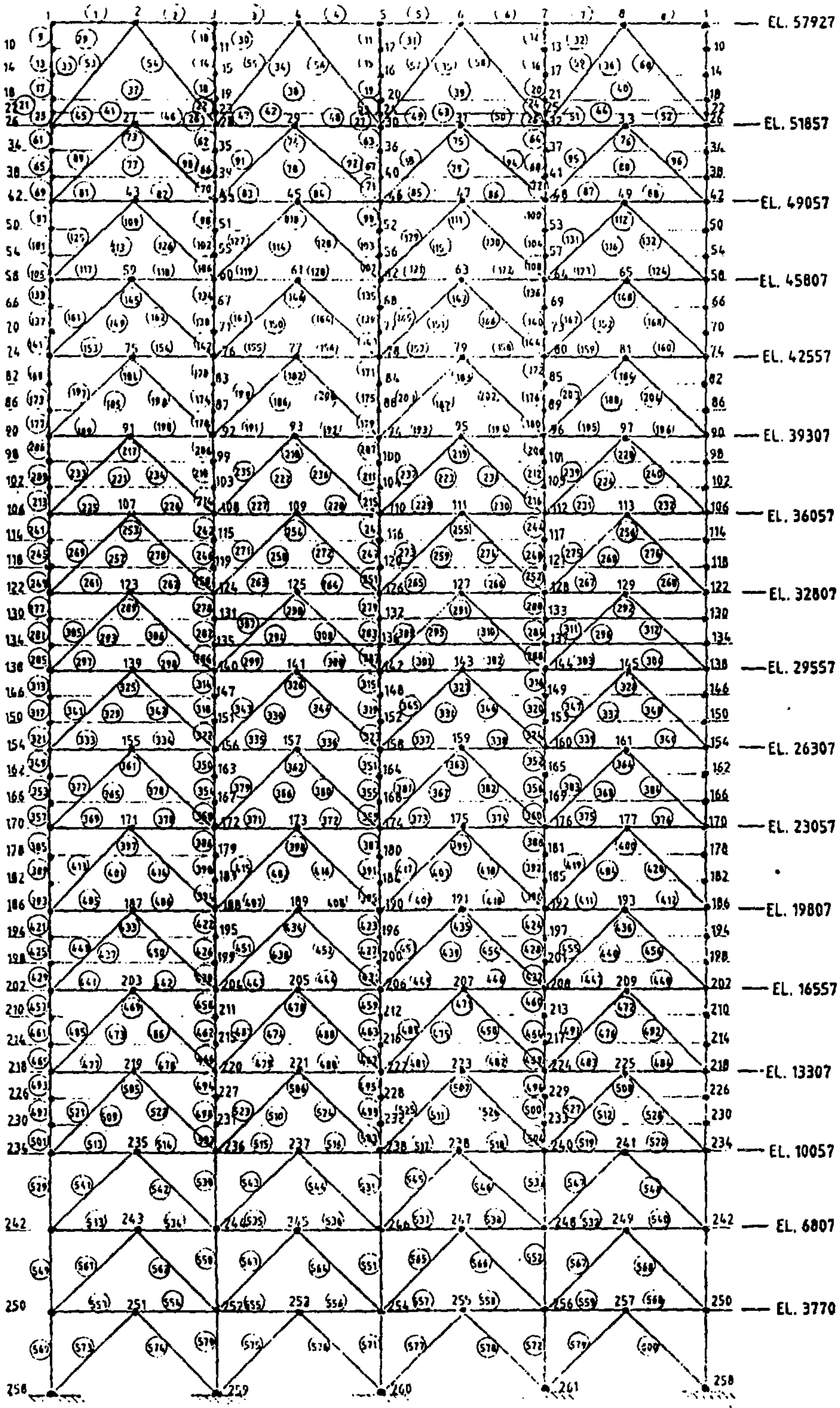


Fig. 3 3-DIMENSIONAL SKETCH OF SUPPORTING STRUCTURE (PLAN AND ELEVATION)



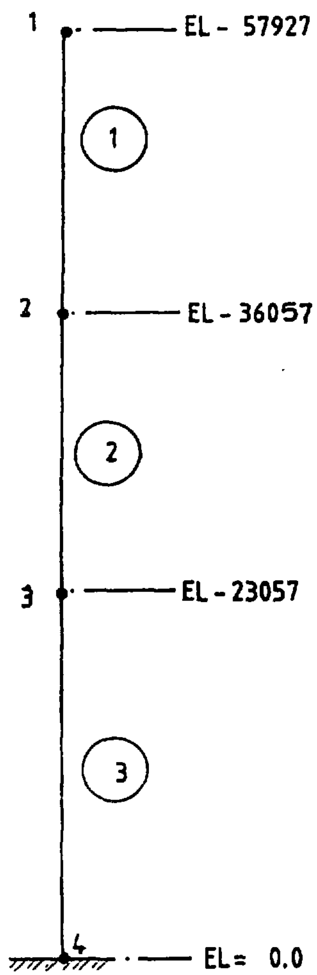
NOTE : ALL DIMENSIONS ARE IN mm.

Fig. 4 CONCRETE FOUNDATION DETAILS OF TOWERS AND THEIR SUPPORTING STRUCTURE



NOTES: 1) BARE NUMBERS DENOTE NODES. (2) ENCIRCLED NUMBERS DENOTE ELEMENT NUMBERS (3) ALL DIMENSIONS ARE IN mm.

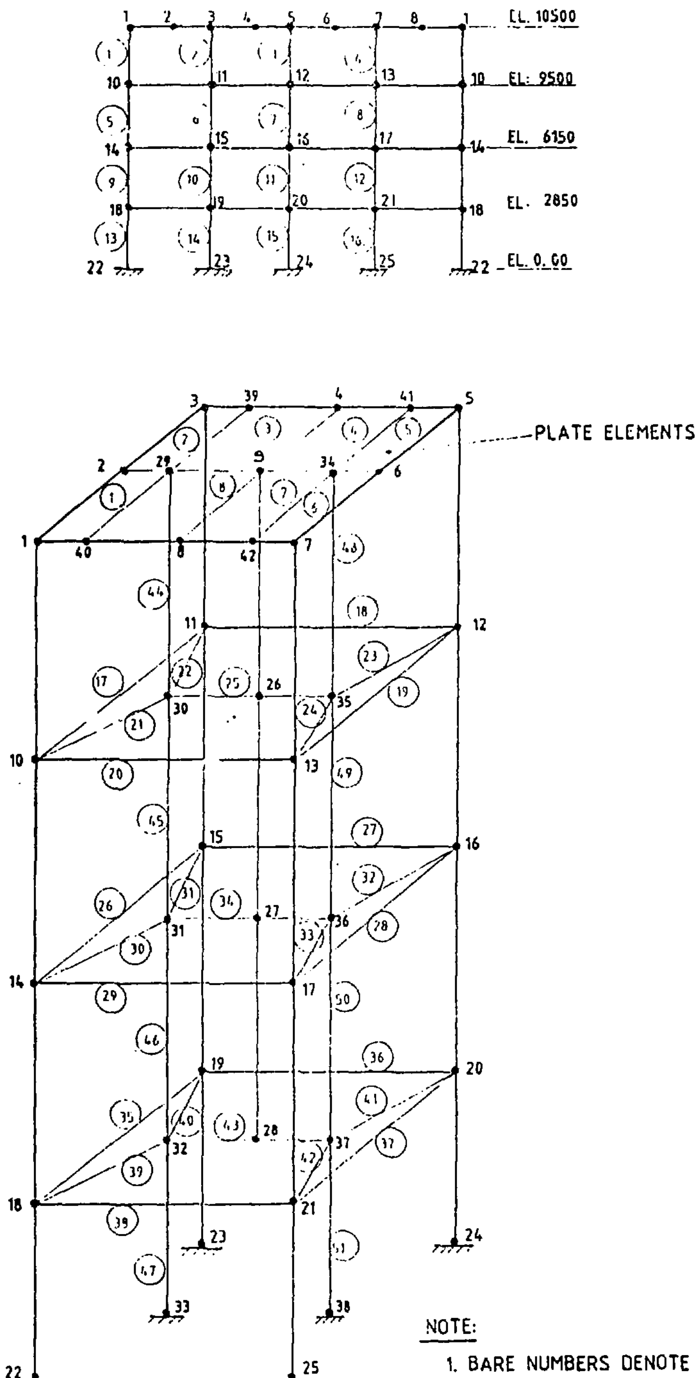
Fig. 5 3-D F E M MODEL OF SUPPORTING STRUCTURE (Developed view)



NOTES:-

1. BARE NUMBERS DENOTE NODES
2. ENCIRCLED NUMBERS DENOTE ELEMENT NUMBERS
3. ALL DIMENSIONS ARE IN mm

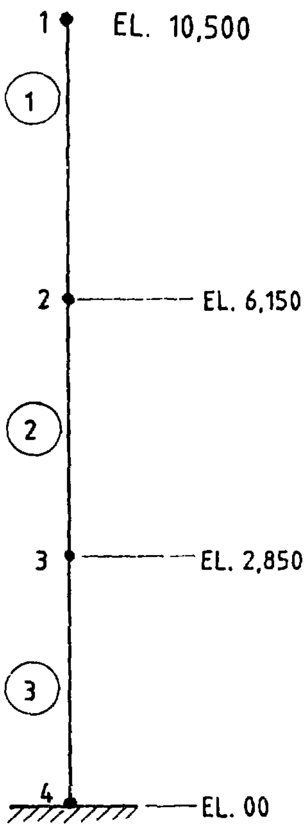
Fig. 6 EQUIVALENT BEAM FOR SUPPORTING STRUCTURE



NOTE:

1. BARE NUMBERS DENOTE NODES
2. ENCIRCLED NUMBERS DENOTE ELEMNTS Nos.
3. ALL DIMENSIONS ARE IN mm.

Fig. 7 F. E. M. MODEL CONCRETE PEDESTALS



NOTES:

(1) BARE NUMBERS DENOTE NODES
(2) ENCIRCLED NUMBERS DENOTE ELEMENT NUMBERS
(3) ALL DIMENSIONS ARE IN mm.

Fig. 8 EQUIVALENT BEAM FOR CONCRETE PEDESTALS

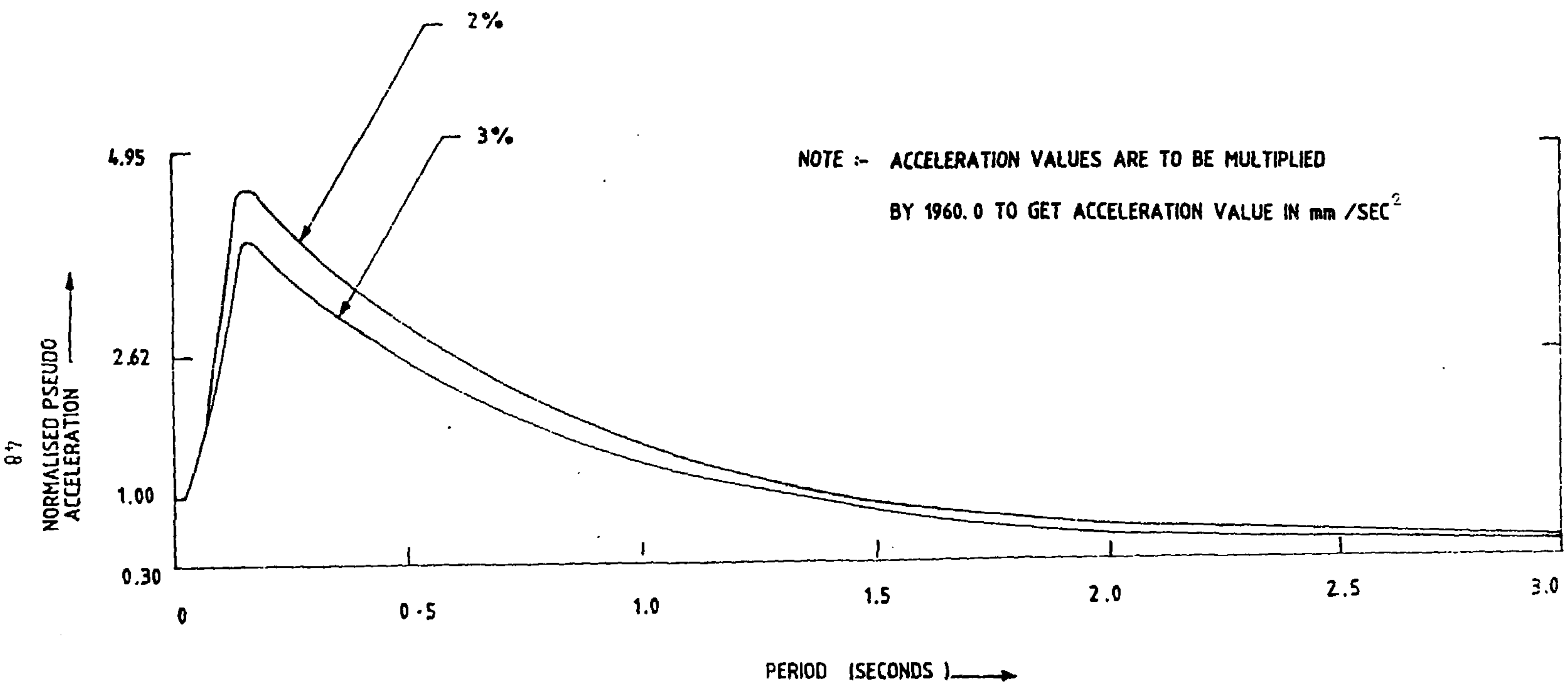


FIG. 9 SMOOTHENED DESIGN REPONSE SPECTRA OF HORIZONTAL MOTION FOR KAIGA SITE FOR SSE

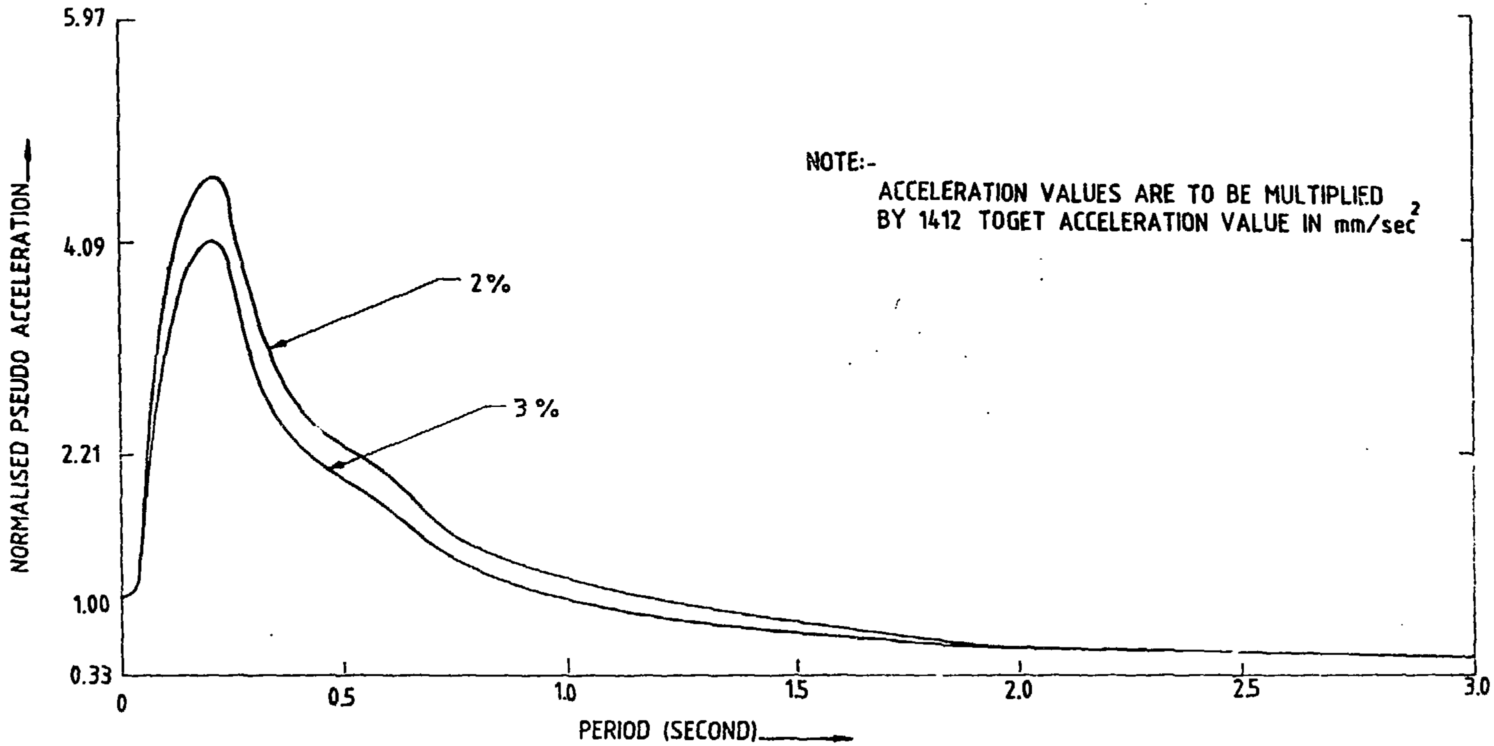


FIG. 10 SMOOTHENED DESIGN RESPONSE SPECTRA OF VERTICAL MOTION FOR KAIGA SITE FOR SAFE SHUT-DOWN EARTHQUAKE

SUPPORTING STRUCTURE

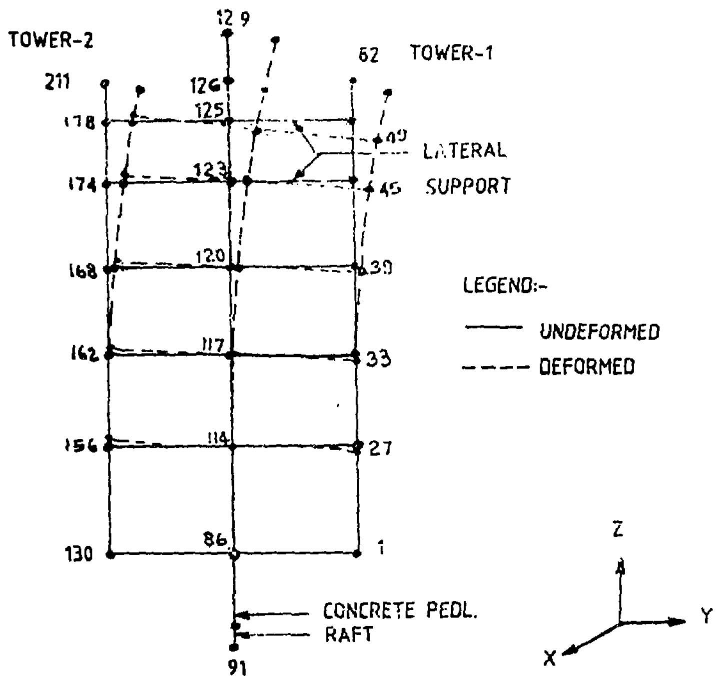


FIG.11 FIRST MODE SHAPE OF TOWER WITH SUPPORTING STRUCTURE (Y-Z PLANE)

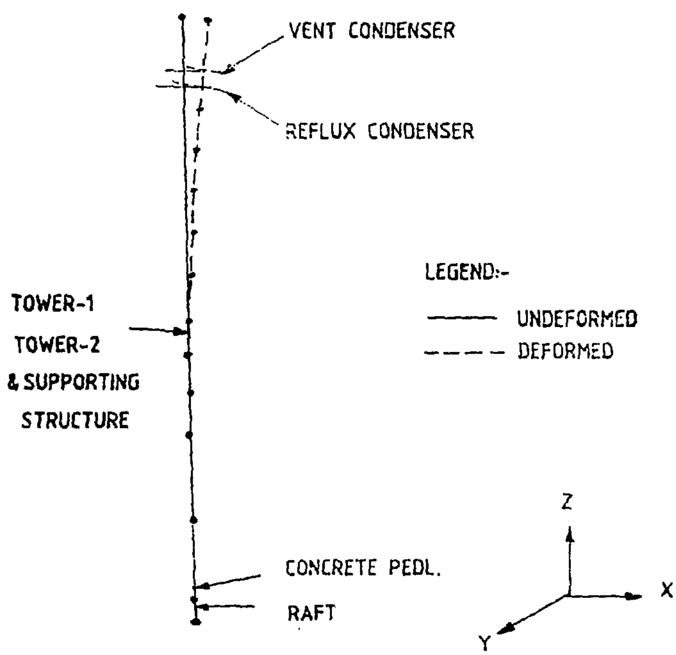


FIG.12 SECOND MODE SHAPE OF TOWER WITH SUPPORTING STRUCTURE (X-Z PLANE)

Published by : M. R. Balakrishnan Head, Library & Information Services Division
Bhabha Atomic Research Centre Bombay 400 085