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Some Issues in Aseismic Design of Nuclear Structures and Components

A. Kakodkar

Director
Reactor Design and Development Group
Bhabha Atomic Research Centre
Trombay, Bombay - 400085

1. INTRODUCTION

It is an honour for me to deliver the ISET annual lecture this year at Roorkee. Indeed Roorkee has nurtured the discipline of earthquake engineering and has made a name for itself on the international scene. We in atomic energy, just as many others in other areas, have benefited by the knowledge, advise and consultancy support provided by the Roorkee school. So in some way this lecture provides me an oppurtunity to pay my respect to pioneers in this field.

During the last few years, we have seen some damaging earthquakes in our country such as the ones at Uttarkashi and Latur. These earthquakes have not only resulted in heavy loss of life and damage to property, but have also created a psychological fear in the minds of people. The damage has been mostly for non-engineered constructions. Such buildings make use of locally available materials and they are based on simple structural systems developed from age old practices of building construction in the region. Such buildings will continue to be constructed as they are percieved to be economical. The number of such buildings is naturally very large and this warrants the development of simple and cost effective backfits and incremental modifications to ensure their safety from collapse during earthquakes.

On the other hand, constructions such as the historic Taj Mahal and Kutub Minar and many ancient monuments, which are manifestations of India's architectural and artistic skills, have

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survived earthquakes and reflect some insight in constructing durable structures with a degree of earthquake resistance. It is prudent that knowledge base and awareness of earthquake engineering and technology grows with large scale development that is taking place in our country. Buildings in major cities, roads, bridges, railway networks, ports and harbours, dams and canals, major industrial projects, various hydel and thermal power projects etc. represent major investments which must be protected from damage due to earthquakes. Department of Earthquake Engineering, University of Roorkee has been taking initiatives in this direction and has done pioneering work in development of national expertise in this vital area.

Aseismic design has a special importance for installations where one is concerned about protection of nearby population and environment against hazards posed by damage of the installation. Large dams and installations dealing with hazardous substances belong to such a category. Nuclear Power Plants also belong to this category. We in DAE thus have been very conscious about developments in this discipline and its applications to nuclear structures and components right from the beginning of our programme. I intend sharing with you our understanding and perceptions in this area , in the present lecture. In doing so, I will be drawing upon the work of a number of my colleagues, particularly from Reactor Structure Section of RED,BARC under the able leadership of Shri H.S.Kushwaha.

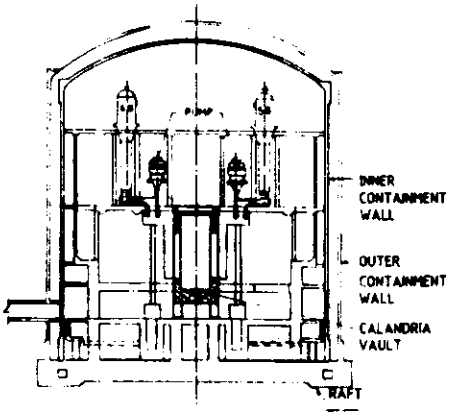
2. ASEISMIC DESIGN OF NUCLEAR SYSTEMS

Safety plays a key role in the design of nuclear structures and components under various normal as well as abnormal conditions. Qualification of nuclear systems under seismic loading has to be given special attention because of the requirement to maintain functional ability of many components/equipments during and after the occurrence of a seismic event. Apart from forming an important design input, seismic aspects also constitute one of the essential siting consideration which are addressed to in detail at the time of site selection for a Nuclear power Plant (NPP). I do not intend to go into siting related issues here but rather concentrate on some of the related topics. Over the years we have gone through a large number of assessments. These assessments have been subjected to internal and external reviews and as a result several additional investigations were required to be carried out to resolve issues that arose and to fill in the knowledge gaps. These exercises have been very rewarding and I would like to present to you a glimpse of these activities and the important insights they provide.

2.1 Seismic Motion

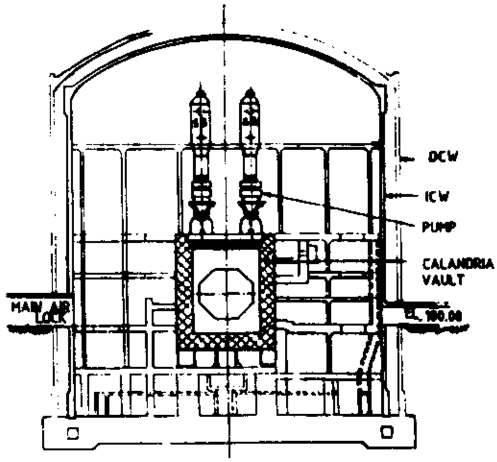
The assessment of design basis earthquake for a nuclear power plant site is based on detailed geological and seismological investigations covering an area of 300 kms. around the site. Inputs for generating design basis earthquake level at the site are themselves statistical in nature and call for careful judgement while arriving at a design basis earthquake based on the perception of potential damage due to various capable faults in the region. Most of the time, due to lack of data, it becomes extremely difficult to assess the activity of the fault and its potential . This is because the data required for making such an assessment necessitates collecting information about micro-earthquakes, ground movements, geophysical investigation etc. over a fairly long period of time and are mostly not available. Therefore, many times, it leads to significant debate among concerned agencies in this field before a judgement can be arrived at.

While defining the design response spectrum for a site, it is important to know the contents of various frequency components in it so that it does not differ from the real earthquake ground motion. This difference may arise due to use of inappropriate attenuation laws, variations in soil properties etc.. Hence, it is normal practice to define the design seismic input on a conservative basis so as to account for these uncertainties. The Design Response Spectrum (DRS) and the design time history are chosen such that the Time history Generated Response Spectrum (TGRS) is compatible to the design response spectrum. However, difficulties do arise when defining the seismic motion using a single time history, because the TGRS of this time history may not match with the design response spectrum at all the frequencies of interest. Alternatively, one may have to define the time history in such a way that the TGRS envelopes the DRS at all the frequencies. However, it may become too conservative in that case.



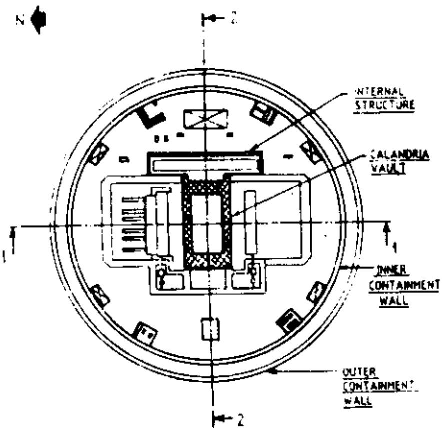
SECTION 1-1

REACTOR BUILDING SECTION



SECTION 2-2

REACTOR BUILDING SECTION



PLAN AT FL.EL.100 M.

REACTOR BUILDING FLOOR PLAN

STANDARDISED DESIGN

SITE	MAGNITUDE	ZPA	EPICENTRAL DISTANCE	FOCAL DEPTH
* 1	5.0	0.1g.	10 km.	10 km
2	6.5	0.2g.	14 km	20 km.
3	6.5	0.2g.	30 km	20 km.

* ADVANTAGE
- ECONOMICAL
- SPEEDY EXECUTION
- NO REPEATED DESIGN EFFORTS

FIG. 1 : TYPICAL NUCLEAR REACTOR BUILDING FOR 500 MWe PHWR

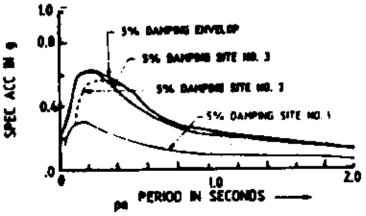
The current regulatory practice (ASCE 4-86) thus recommends the use of multiple time histories which are compatible to the given design response spectrum to reduce this conservatism. To illustrate, three independent time histories have been used for defining the seismic motion in each direction for 500 MWe PHWR (Fig 1). The response due to these three time histories is almost identical as shown in Fig.2 which shows that there are no holes in defining these motions. In addition, the floor response spectra generated using multiple time histories are much smoother and broader with less sharp peaks and thus help in reducing the seismic response in components and systems.

Another acceptable way of defining seismic input which is gaining importance is to express it in the form of Power Spectral Density Function (PSDF). Use of this method of defining seismic input eliminates the use of multiple time histories and it is also free from the various deficiencies in defining the DRS because a properly defined PSDF accounts for the seismic energy content at various frequencies in a correct manner. Use of PSDF approach for the 500 MWe PHWR reactor building has given identical results for both the deterministic and stochastic approaches (Fig..2) and thus signifies that both the inputs have been defined properly.

2.2 Mathematical Modelling

Conversion of a real life structure into a suitable mathematical model is a highly skilled task and requires an in-depth understanding of the behavior of structural systems under dynamic loading. The model should be simple enough to depict the physical behavior of the system. More often than not, complex models involving a large number of degrees of freedoms have problems of ill-conditioning of solution matrices. Moreover, dynamic analysis of structures demands a coarser model only so as to reflect the overall dynamic behavior of the system.

Out of the various modelling techniques available, lumped mass beam models are the most popular ones because of their inherent conservative response predictions (Reddy et al.,1987). It is a cumbersome task to evaluate the properties of lumped mass beam models of complex structural geometries which have length to width ratio of almost unity. Structural features such as the elbow



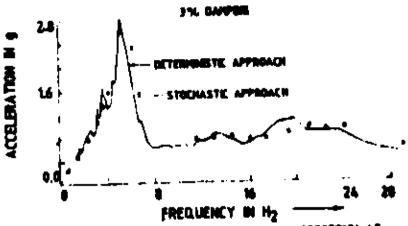
ACCELERATION SPECTRA FOR SAFE SHUTDOWN EARTHQUAKE FOR SITE NO. 1, 2 & 3 ALONG - WITH ENVELOPE SPECTRA.

KIND OF ANALYSIS	TOTAL BASE SHEAR (T)	TOTAL BASE MOMENTS (T-M)
COUPLED RESPONSE SPECTRUM ANALYSIS	31710	11,657,00
COUPLED T. H ANALYSIS # 1	35610	11,794,50
COUPLED T. H ANALYSIS # 2	35100	12,106,90
COUPLED T. H ANALYSIS # 3	35503	12,577,50

COMPARISON RESPONSE IN PRIMARY STRUCTURE

(ANALYSIS USING 3-TIME HISTORIES)

(N-S)



COMPARISON OF FLOOR RESPONSE SPECTRUM AT CENTER OF CALANDRIA (N-S)

COMPARISON OF DEFLECTIONS FOR LUMPED MASS BEAM MODEL USING DETERMINISTIC & STOCHASTIC APPROACHES
(IN mm)

1 = DETERMINISTIC
2 = STOCHASTIC

DIRECTION OF VIBRATION → / LOCATION ↓	N-S		E-W		VERTICAL	
	1	2	1	2	1	2
TOP OCW (1)	0.57	0.57	7.71	7.13	0.97	0.92
TOP LCW (4)	13.05	14.63	16.41	17.10	0.88	0.83
EL-130.0 M (13)	11.23	11.69	13.46	13.95	0.97	0.91
TOP CAL VAULT (2A)	06.21	05.52	04.15	03.67	0.46	0.43

COMPARISON OF BASE MOMENTS & SHEARS FOR LUMPED MASS BEAM MODEL USING DETERMINISTIC & STOCHASTIC APPROACHES
(IN mm)

1 = DETERMINISTIC
2 = STOCHASTIC

DIRECTION OF VIBRATION → / LOCATION ↓	N-S		E-W		VERTICAL	
	1	2	1	2	1	2
BASE SHEAR O. C. W. (T)	6605	6487	5707	5303	5406	5006
BASE SHEAR I C. W. (T)	10590	11260	12540	13160	5146	4863
BASE SHEAR INTERNALS (T)	14910	11540	15460	13470	15500	14100
BASE MOMENT O C W (T-M)	309400	312100	269600	259200	—	—
BASE MOMENT I C W (T-M)	504600	495400	593600	628100	—	—
BASE MOMENT INTERNAL (T-M)	360400	303400	363900	344400	—	—

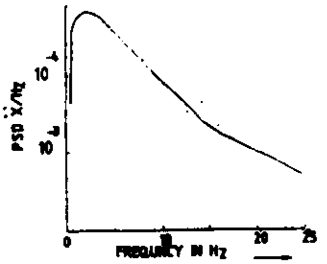


FIG. 8: POWER SPECTRAL DENSITY FUNCTION COMPATIBLE TO THE 5% DAMPING ENVELOPE GROUND SPECTRUM

FIG. 2: ISSUES RELATED TO SEISMIC GROUND MOTION

effect (Fig.3.) may even prohibit the use of simple analytical evaluation of geometrical properties (such as moment of inertia and shear area) of the beam model. In such a case, one has to take recourse to Finite Element Method (FEM) for modelling the structure using plate/shell elements and perform a static analysis. The deflections and rotations so obtained are then used to arrive at stiffness properties of the representative beam model. This is done by matching the strain energies of the FEM model and the beam model (Soni et al.,1987).

On many occasions, use of FEM models are also warranted for the dynamic analysis of important structures such as the containment building. This enables the generation of the detailed stress picture. As shown in Fig.3, the two containments have been modelled using axi-symmetric elements and that the non- axisymmetric reactor internals have been converted into an equivalent axi-symmetric system by matching their dynamic characteristics for the first mode of vibration using the modal mass concept. Fig. 3 also shows a comparison of the results obtained by 2-D beam model and the axi-symmetric model for the two containments. It can be observed from this figure that the beam model predicts conservative results because in this model, the total response due to input seismic motion is concentrated in first few modes only whereas the same is distributed over various other modes in an FEM model (Reddy et.al.,1987).

Nuclear structures consist of structural systems which are usually symmetric about the two horizontal axes, so that the torsional effects are minimized. However, the torsional effects do arise at various floor levels (due to location of various equipment masses) where the mass centre and the shear centre do not coincide with each other. These effects are accounted by using a 3-D beam model and specifying the eccentricity between the shear centre and the mass centre (Fig.3)(Reddy et al.,1994).

2.3 Soil-Structure Interaction

Soil-structure interaction can play a major role in altering the seismic response of a structure. This phenomenon not only introduces flexibility in the overall system, it also introduces additional

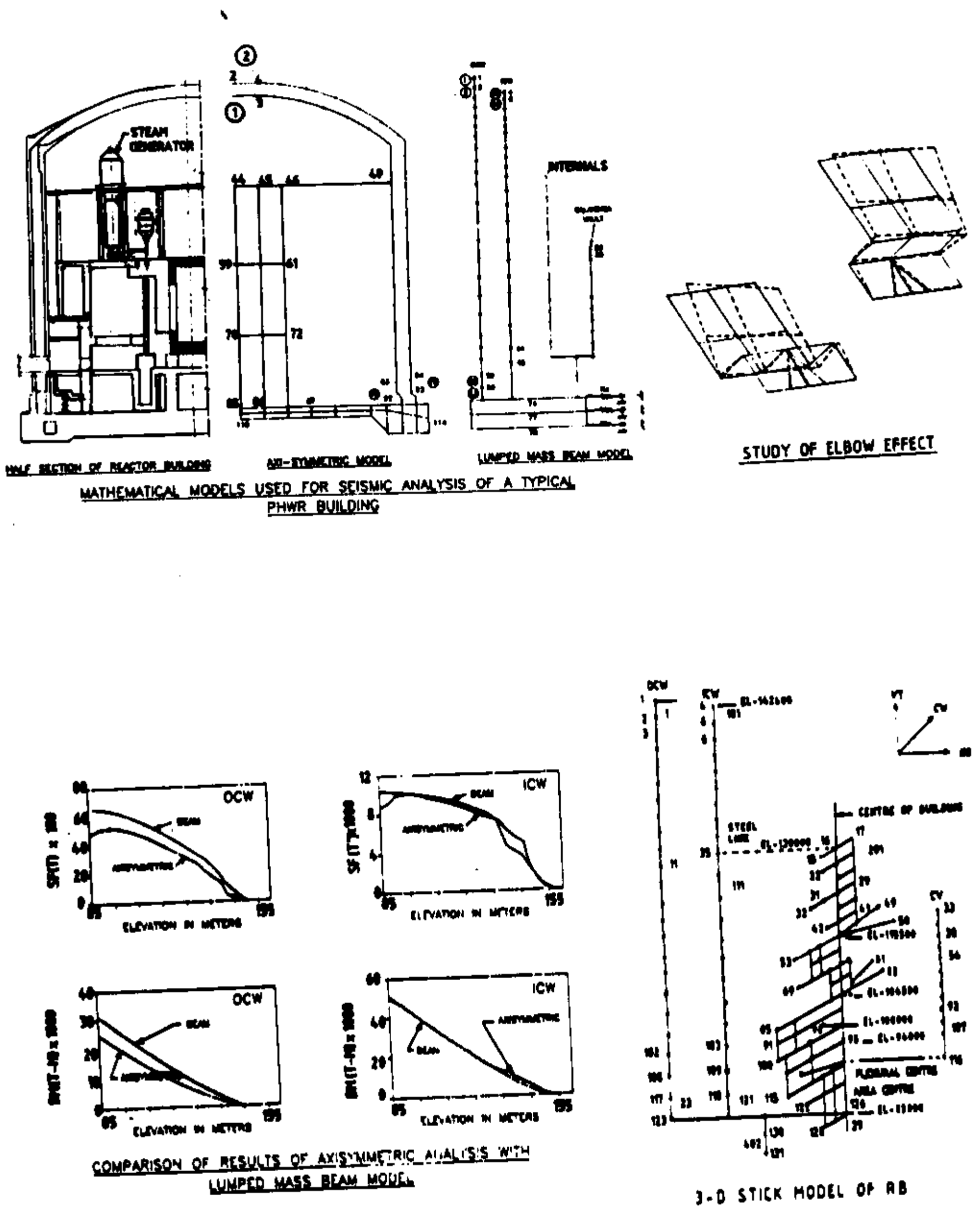


FIG. 3: ISSUES RELATED TO MATHEMATICAL MODELLING

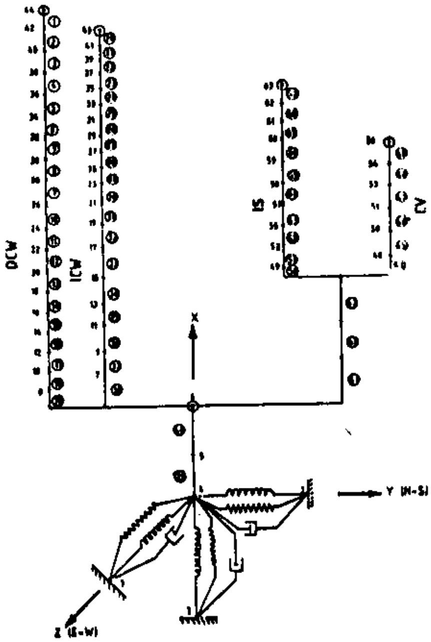
damping in the form of radiation damping. Analysis for soil-structure interaction effects are normally performed by either using detailed FEM model of the soil-strata or by using the soil impedences derived based on half-space theory. Standards usually specify as to when this interaction effect is significant and should be considered. The ASCE standard specifies that if the shear wave velocity of soil is more than 1100 m/sec and the corresponding shear strain in the soil medium is less than 10E-3 percent, a fixed base condition may be assumed. However, use of such assumption solely based on the limit on shear wave velocity may sometimes lead to erroneous results.

This situation mostly arises on account of superstructure stiffness which may at times become comparable or even more than the soil stiffness for a particular direction of vibration (Ghosh at al.,1994). One such example is shown in Fig 4. for a 220 MWe PHWR reactor building wherein it has been observed that for a shear wave velocity of 1600 m/sec for the soil medium, the building does not behave as a fixed base case. The seismic response is underpredicted if one performs a fixed base analysis (Fig.4). The codal provisions should thus be used with caution and with full understanding of the subject matter.

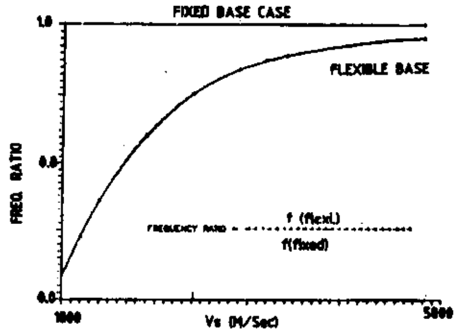
2.4 Compensation for uncertainties

Seismic analysis of secondary systems such as components, piping etc. is normally performed by using the floor motions generated at various levels of the primary structure i.e. the civil structure. These floor motions are obtained as a by- product from the analysis of civil structure and can be either in the form of Floor Response Spectra (FRS) or Floor Time Histories (FTH). These floor motions may differ depending on the variations in the various parameters which enter into the calculations and which have a direct effect on the structural frequencies.

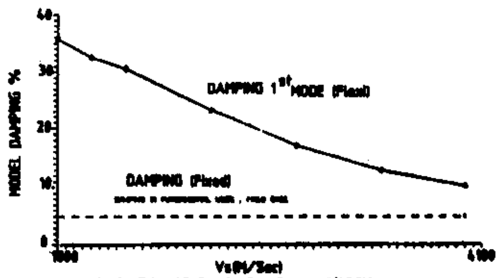
To account for the various uncertainties in the structural frequencies owing to variations in parameters such as the material properties of the structure and soil, damping values, soil-structure interaction effects, and the approximations involved in modelling techniques, the computed FRS from the FTH is smoothened and the peaks associated with each of the structural frequencies



LUMPED MASS BEAM MODEL OF REACTOR BUILDING



VARIATION OF FREQUENCY RATIO WITH Vs (VERTICAL DIRECTION)



VARIATION OF DAMPING WITH Vs (VERTICAL)

Comparison of the Frequencies(Hz) and Mass Participation factor(%) in Vertical and Horizontal direction for Flexible base and Fixed base case

	Vertical direction		Horizontal direction	
Mode No	Fixed Base	Flexible Base	Fixed Base	Flexible Base
1	16.26(14.99%)	10.29(95.82%)	3.90(12.00%)	3.20(44.40%)
2	17.03(14.84%)	16.47(0.186%)	6.44(12.37%)	6.10(4.16%)
3	17.83(45.40%)	17.34(0.107%)	6.57(34.29%)	6.49(0.912%)
4	36.88(6.70%)	22.34(4.34%)	10.07(2.64%)	9.98(2.179%)

Comparison of the Forces at the Top of Raft for different Substructure

	Vertical direction		Horizontal direction			
	Flexible base. Force(t)	Fixed base Force(t)	Flexible base Force(t)	Flexible base Mom.(t-m)	Fixed base Force(t)	Fixed base Mom.(t-m)
O.C.W	1640.0	1186.0	3439.0	141300.0	2927.0	113900.0
I.C.W	1738.0	1248.0	4949.0	207900.0	3130.0	123900.0
INT. STR.	4991.0	3312.0	7637.0	309300.0	7221.0	290200.0

FIG. 4 : ROLE OF SUPERSTRUCTURAL STIFFNESS IN SOIL-STRUCTURE INTERACTION

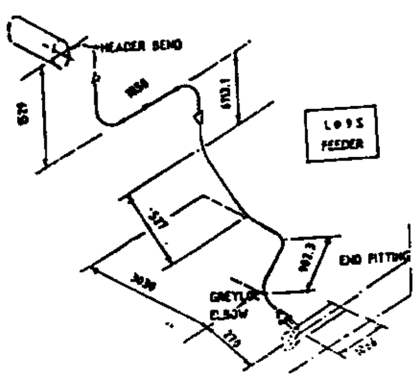
should be broadened based on the variations in frequencies. In the absence of detailed parametric study, it is recommended that this variation should be taken as + 15% around the peaks which are associated with the structural frequencies (USNRC RG.1.122).

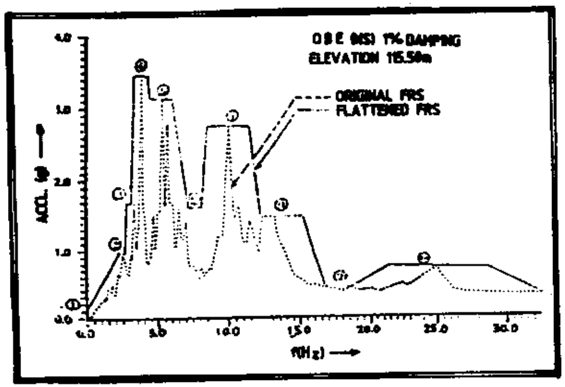
While the procedure to account for these uncertainties is simple in case of FRS, to account for the similar peak broadening effect in FTH, one must be very careful. This is because to incorporate this effect in FTH, the time history has to be expanded or compressed by upto 15%. This not only changes the total time duration of the FTH, it also affects the frequency contents of the FTH by similar amount. Study on a simple feeder pipe shows that merely expanding or compressing the FTH by 15% does not give the governing design response(Fig.5). Instead, the analysis using FTH with broadening effect corresponding to the second frequency of the pipe (expanding and compressing by 9.2%) gives higher response which governs the design (Soni et al.,1992). It has been observed that it is also close to the flattened FRS results. Thus, it can be observed from this exercise that flattened FRS or flattened FTH give almost identical results if the design practice is adopted in a correct manner.

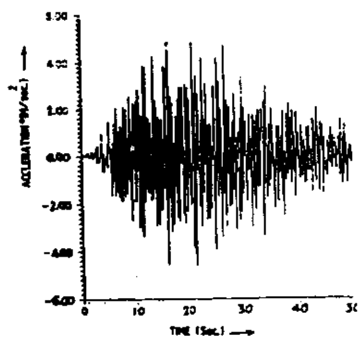
2.5 Heavy Components

Aseismic design has to envelope both passive as well as active components. While the design of former is relatively easy and straight forward, the design of active components requires special efforts to confirm their functional capability. Design of passive components is performed either by decoupling them from the civil structure if the interaction effects are negligible or by coupling them with the civil structure if the interaction effects are found to be significant. The criteria as to whether any component can be decoupled from the civil structure is given by the decoupling criteria as given below: (USNRC SRP 3.72)

(i) If Rm < 0.01, the component can be decoupled for any Rf.
(ii) If 0.01 <Rm < 0.1, then it can be decoupled only if 0.8 > Rf >1.25
(iii) If Rm > 0.1, it should be coupled with the primary structure.







RESULTS OF ANALYSIS
L-09 FEEDER

Number/ Method	Resultant Moment(Kgf-mm)* Greyloc elbow	Header bend
1. URS	80497	112385
2. FRS	193943	332129
3. UTH	78169	103310
4. ETH(15%)	114203	104387
5. CTH(15%)	51644	64214
6. ETH(9.2%)	203218	318915
7. CTH(9.2%)	56532	81942
8. ETH(6.3%)	196754	282966
9. CTH(8.3%)	55347	82981
10.Envelop URS of cases 3,4,5	111396	162583
11.RS with 15% peak reduction	164933	282367

$*\sqrt{Mx^2+My^2+Mz^2}$

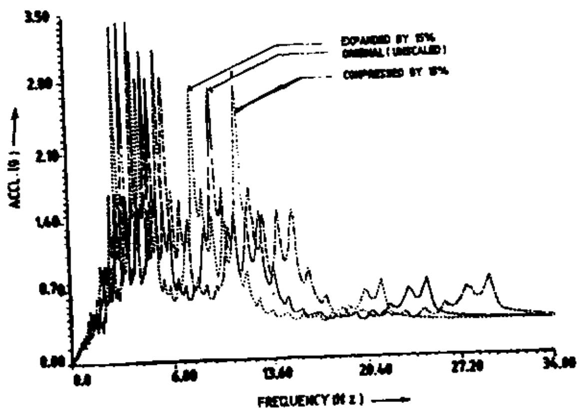


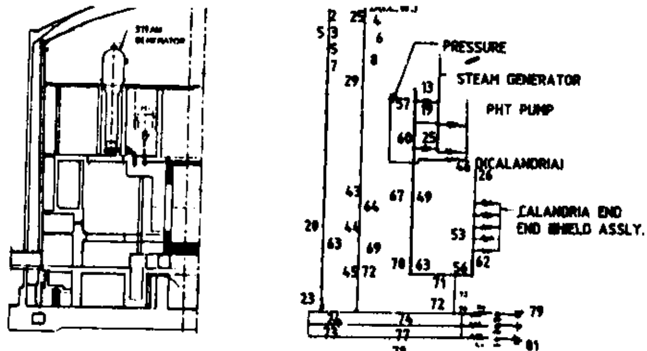
FIG. 5: PEAK BROADENING EFFECT IN FLOOR RESPONSE SPECTRA AND FLOOR TIME HISTORY

Here Rm and Rf are the ratio of modal masses and modal frequencies of the secondary system to the primary structure respectively. Use of this criteria for heavy equipments which have dynamic interaction with the civil structure, requires that they should be modelled with the civil structure itself. One such study is depicted in Fig.6 wherein heavy equipment like steam generators, primary pumps, pressuriser and the calandria-end shield assembly have been modelled with the reactor building. Results of this study as shown in Fig.6. show that a decoupled analysis for such components may lead to erroneous results (Soni et al.,1989).

2.6 Multiply Supported Systems

Components which are supported at more than one elevations get excited by different support motions at different levels. This kind of excitation by a multiply supported system generates not only inertial response but also the response due to Seismic Anchor Movements (SAM) which is generated due to the differential motion of various supports. For the components/systems which can be decoupled from the civil structure, there are two approaches available for the design of such components. These are the use of multiple support excitation technique and the use of envelope spectrum approach. In the envelope spectrum approach, analysis is performed by using a single spectrum which envelopes the individual support point response spectra. This approach, thus, results in applying a conservative upper bound support motion at all the supports of the components and hence it usually yields conservative seismic response(ASME III , App.N,1986). On the other hand, in the multiple support excitation technique different support groups are subjected to different support motions either in the form of FRS or FTH and the overall response of the system is then obtained by combining the responses due to various support groups appropriately.

In most of the instances, the envelope spectrum approach gives conservative response prediction as compared to the multiple support excitation technique. However, in case of equipment with large overhang, it has been observed that the envelope spectrum approach gives unconservative results as shown in Fig.7. This is on account of out-of-phase motion of supports which gives rise to additional response if the multiple support excitation technique is followed



HALF SECTION OF REACTOR BUILDING

MATHEMATICAL MODEL FOR COUPLED ANALYSIS

MATHEMATICAL MODEL OF REACTOR BUILDING FOR COUPLED ANALYSIS

COMPARISON OF FORCES AT EQUIPMENT SUPPORT (N-S DIRECTION)

METHOD OF ANALYSIS	FORCE IN K1 (T)	FORCE IN K2 (T)	FORCE IN K3 (T)	FORCE IN K4 (T)	FORCE IN K5 (T)	BASE SHEAR PRESSURE
COUPLED R'S ANALYSIS	670	240	261	93	1150	226
COUPLED TH ANAL #1	669	264	220	61	735	119
COUPLED TH ANAL #2	621	209	222	50	774	122
COUPLED TH ANAL #3	630	210	206	53	791	111
DECOUPLED F. R. S. ANAL	600	640	300	120	1363	120

DECOUPLING CRITERIA

(i) IF $Rm < 0.01$, DECOUPLE FOR ALL R_f.

(ii) IF $0.01 < Rm < 0.1$, DECOUPLE ONLY IF $0.8 \geq R_f \geq 1.25$

(iii) IF $Rm > 0.1$, COUPLE THE COMPONENT.

$$Rm = \frac{\text{MODAL MASS OF SECONDARY SYSTEM}}{\text{MODAL MASS OF PRIMARY SYSTEM}}$$

$$R_f = \frac{\text{FREQUENCY OF SECONDARY SYSTEM}}{\text{FREQUENCY OF PRIMARY SYSTEM}}$$

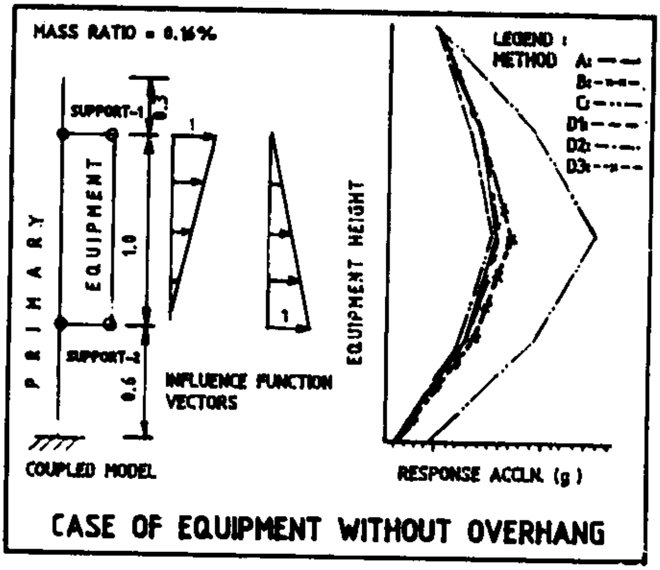
FIG. 6: COUPLED ANALYSIS FOR SEISMIC DESIGN OF HEAVY EQUIPMENTS

which otherwise is not possible to pick up following the envelope spectrum approach. This is because the envelope spectrum approach assumes uniform motion for all the support points. Fig.7. shows results for an equipment with and without overhang wherein it has been observed that the envelope spectrum approach gives conservative results for the equipment without overhang but it gives unconservative results for the equipment with overhang(Neelwarne et al.,1991). In this figure, case A represents the time history analysis of the component coupled with civil structure, case B represents the response spectrum analysis of component coupled to civil structure, case C represents the envelope spectrum analysis of the component alone; case D1, D2 and D3 respectively represent the multiple support technique analyses for the component alone with algebraic, SRSS and absolute support group combination methods. These results thus show that one must be very careful while deciding on the method of analysis for such components.

2.7 Piping Systems

Nuclear piping systems handle radioactive fluids and therefore, they must be designed in such a way that there is minimum maintenance work required on them so as to reduce the man-rem consumption. The normal operating loads such as pressure and temperature require that the system should be as flexible as possible. On the other hand, the seismic loading demands that the system should be rigid enough to attract low acceleration. Thus, the designer has to confront with these two mutually conflicting requirements.

As far as possible, the designer tries to meet these challenges by the use of simple supports. However, in many cases, it becomes unavoidable to use snubbers which act as seismic restraints but permit slow thermal movement. For example, many PWRs and BWRs have around 300 - 400 snubbers in their piping systems. This is a very high number looking at the problems associated with these supports such as their inadvertent locking, requirements for regular maintenance and in-service inspection etc. An endeavor has, therefore, been made to limit the use of snubbers in our systems as far as possible. This has been achieved by designing the layouts in such a way that the operating stresses are low and thus a comfortable margin is available for accommodating the seismic stresses. The system is then permitted to reach its maximum allowable stress range during



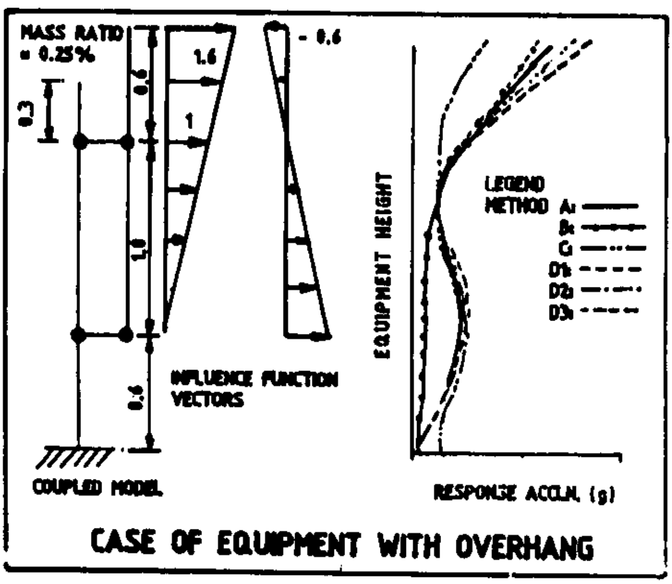


FIG. 7 : ANALYSIS OF MULTIPLY SUPPORTED SYSTEMS - A COMPARISION OF DIFFERENT APPROACHES

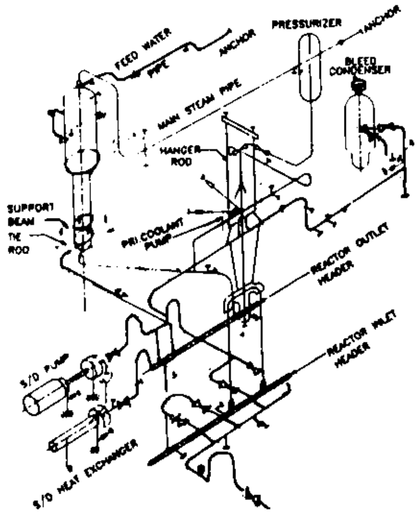
a seismic event since it is probably a one time loading in its life time. This helps in reducing the number of supports and snubbers in the piping system and also improves its fatigue life. The Primary Heat Transport (PHT) system of a 500 MWe PHWR has been designed with only 36 snubbers in its entire loop which is a very low number as compared to what is being used elsewhere in the world (Fig.8).

In view of the issues associated with active snubbers, the present day technology advocates the use of non-linear restraints in place of snubbers. These restraints can be either hysteretic type or friction type or some other type (Fig. 8) and absorb the seismic energy when they undergo non-linear deformation while the entire piping system remains linear elastic. The hysteretic type supports absorb seismic energy by taking the elements into their plastic regime during seismic event. In addition to these, use of isolated damping elements at some places in the system also have been found helpful in controlling the seismic response. Use of these kind of supports for our plant would require a lot of research and development activity within the country. In view of their passive nature, these type of supports/dampers offer considerable advantage and should be developed. These are cerainly within the available national capability. Many other sectors such as petrochemicals, fertilisers,thermal power etc. would also benefit from such a development.

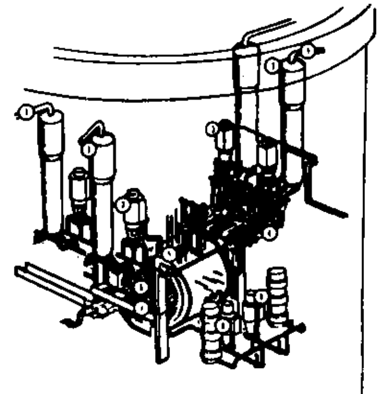
2.8 Active Equipment

Active equipment, whose function depends on the actuation of some parts within the system, cannot be qualified by analytical means with full guarantee. It is, therefore, always desirable to qualify such components by testing only. Active components can be of two types e.g. rotating equipment such as pumps, motors, diesel generators etc. and actuating type such as valves, relays, switches etc. While for the actuating type components which are usually small, it is easier to perform seismic testing, for the rotating equipment which are generally massive, it is not possible to qualify them by testing on account of lack of test facilities and test set-ups etc.

For large active equipment, resorting to analytical means for their qualification becomes inevitable. One such example is shown in Fig.9 for the moderator-system pump motor assembly of

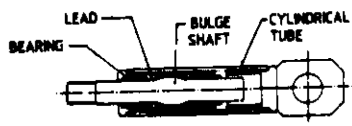


MATHEMATICAL MODEL OF PHT PIPING FOR 500 MWe PHWR



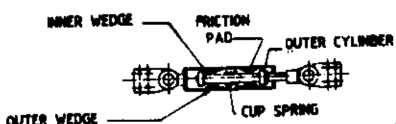
1 MAIN STEAM SUPPLY PIPING
2 STEAM GENERATORS
3 MAIN PRIMARY SYSTEM PUMP
4 FEEDERS
5 CALANDRIA ASSEMBLY
6 FUEL CHANNEL ASSEMBLY
7 FUELLING MACHINE BRIDGE
8 MODERATOR CIRCULATION SYSTEM

NUCLEAR CORE SYSTEM & STEAM GENERATING PLANT

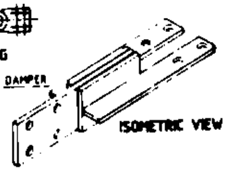


CYLINDER TYPE L.E.D

ENERGY ABSORBER

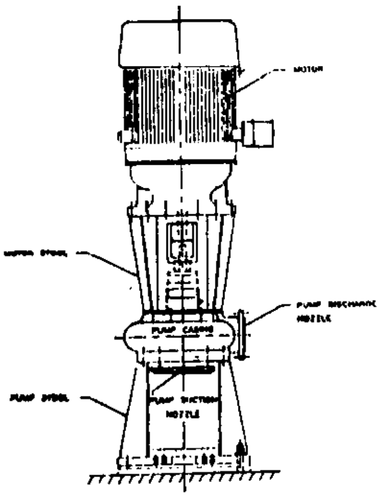


CROSS SECTION OF FRICTION DAMPER

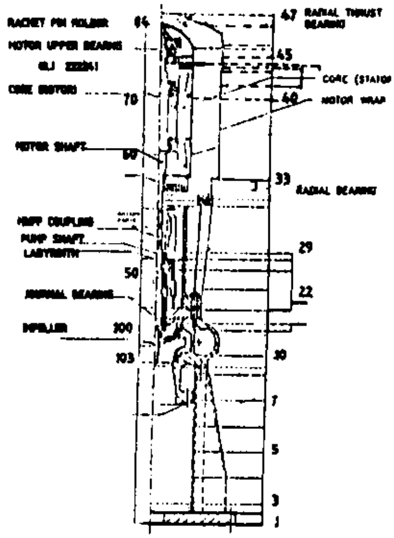


CONSTRAINED LAYER VISCOELASTIC SHEAR DAMPER

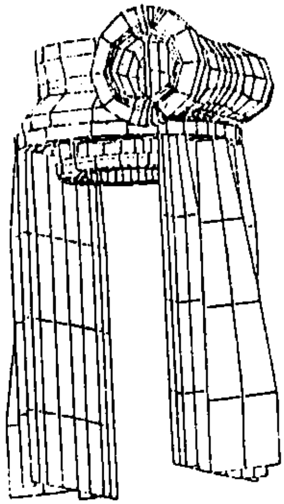
FIG. 8: A SEISMIC DESIGN OF PIPING SYSTEMS



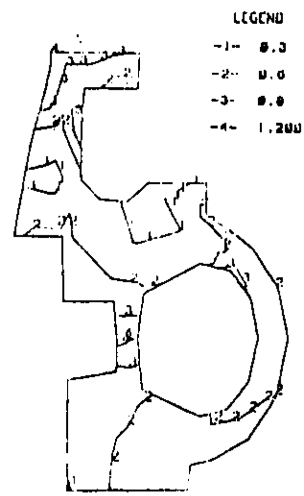
MODERATOR SYSTEM PUMP MOTOR ASSEMBLY



SECTIONAL VIEW OF PUMP - MOTOR ASSEMBLY



3-D MODEL OF PUMP CASING



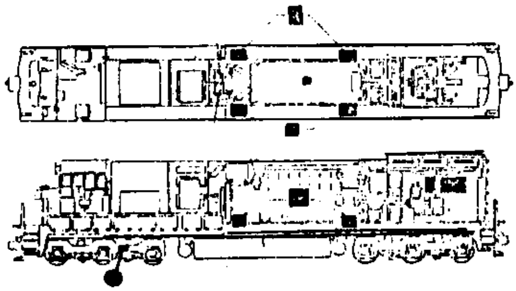
STRESS CONTOUR SECTION 3-3

BEARING LOCATION	TOTAL DESIGN CLEARANCE (MICRONS)	TOTAL RELATIVE DEFLECTION (MICRONS)	EXCESS CLEARANCE AVAILABLE (MICRONS)
LOWER JOURNAL	75.0	68.9	6.1
UPPER JOURNAL	75.0	71.1	3.7

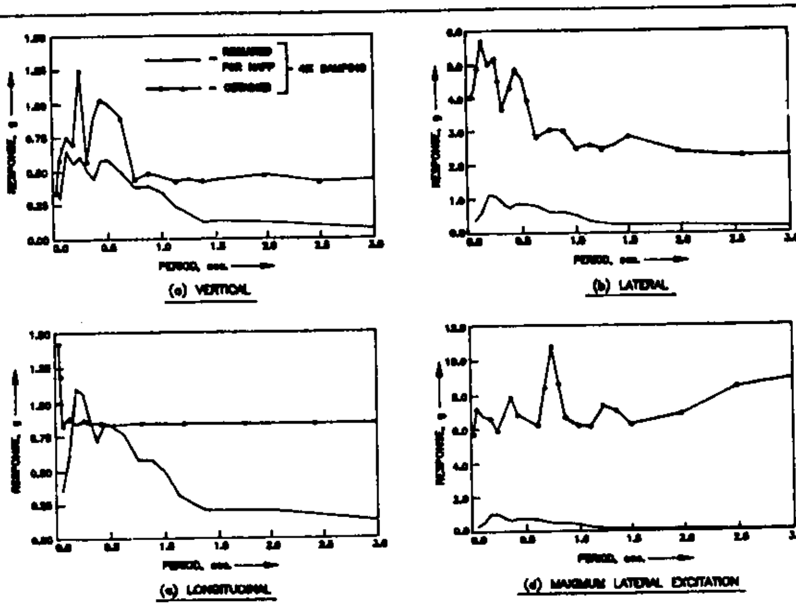
FIG. 9: SEISMIC QUALIFICATION OF MODERATOR SYSTEM PUMP-MOTOR UNIT AN ANALYTICAL APPROCH

a 220 MWe PHWR. The model adopted for the analysis consisted of modelling of static parts, rotating parts, the anti-friction bearings, the journal bearing etc. (Soni et al.,1995). A response spectrum analysis of this assembly was performed to show that the induced relative deflection between the static part and the rotating part is less than the clearance between them and hence no rubbing will take place during a seismic event, thus ensuring the functional capability of the assembly during a seismic event. The stresses induced in the pump casing were also found to be well within their limits.

While one can justify qualification of active equipment based on analysis upto a point, there can be no question about greater confidence that experimental testing provides. In our country we seriously lack these facilities both in terms of size and number. While efforts to add more facilities should be pursued to the maximum extent possible within the available financial resources, we also need to adopt innovative approaches which could be cheaper. One such example illustrated here is about the functional testing of DG sets of a nuclear power plant. Since, it was not possible to test them on the existing shake table facility in India, it was decided to instrument a diesel locomotive which uses a similar engine and record the excitation seen during its travel on the tracks in the Pune - Kolhapur sector. Accelerometers were mounted at various locations on the diesel engine and its mounting frame and the measured accelerations were then compared with the required excitation levels to which the unit is likely to get subjected to during a seismic event (Fig.10). It was observed from the recorded response that it envelopes the required acceleration spectra at most of the frequencies of interest which are present during a seismic event. This, thus, gave us a confidence that the unit can be used in a nuclear power plant without any cause for concern on this count (Moorthy et al.,1994). Similar tests can be performed on other components also for their qualification. It seems to me that one could realise a much cheaper seismic qualification test facility using some of the existing tracks and rolling stocks. I must acknowledge here that the idea of using railway track unevenness for seismic testing was given to us by Prof. R.N.Iyengar.



SCHEMATIC OF DIESEL ELECTRIC LOCOMOTIVE SHOWING MEASURMENT LOCATIONS



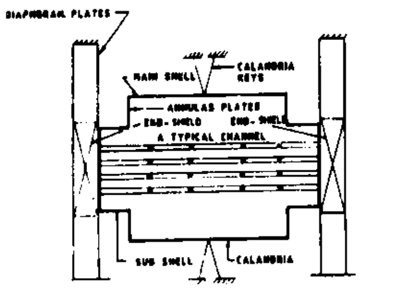
RESPONSE SPECTRA DUE TO RAILWAY TRACK UNEVENNESS INDUCED VIBRATION

FIG. 10: SEISMIC QUALIFICATION OF DG SETS AN UNCONVENTIONAL EXPERIMENTAL APPROACH

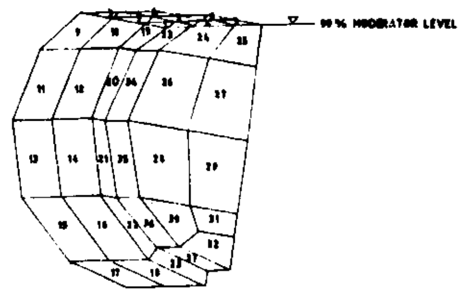
2.9 Hydrodynamic Effects

The dynamic analysis of liquid containers has been a major research and development activity in nuclear, space and petrochemical industries. Hydrodynamic effects due to the liquid give rise to two kinds of loads on the container viz. impulsive load near the bottom of the container and sloshing load near free surface. It has been observed that usually the frequency of sloshing mass is very low and the frequency of impulsive mass is very high. This behavior helps the designer to evaluate the hydrodynamic forces by decoupling these two inertial effects. However, for large size flexible tanks, coupling between the liquid impulsive modes and shell modes and also between the sloshing modes and shell modes of the container may become important because of the interaction effects amongst them. This phenomenon may have pronounced effects in cases like partially filled large petrochemical tanks, large size water tanks and process tanks in chemical industry, calandria vessel in a PHWR, main reactor vessel of a Fast Breeder Reactor etc.

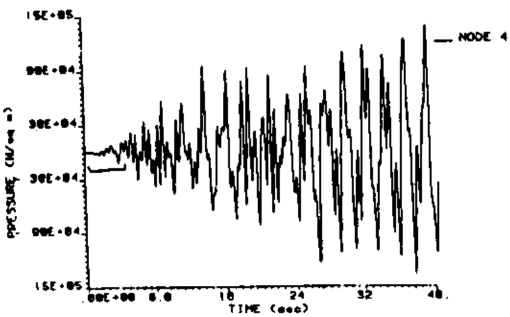
Analysis of calandria vessel of a 500 MWe PHWR for hydrodynamic effects is shown in Fig.11. The sloshing analysis of this vessel is complicated due to its horizontal configuration, flexible diaphragm supports, non-flat bottom and multiple compartment geometry with shell and subshells of different sizes and cross-sectional curvatures. This necessitates evaluation of anti-symmetric sloshing load along two principal shell axes in transverse and longitudinal directions. It is desirable to study and quantify the peak sloshing and impulsive pressures for the design of the two independent shutdown devices in addition to the design of vessel itself. A three dimensional FEM code FLUSHEL (Singh et al.,1990,1991) has been developed for studying this phenomenon. Fig.11 shows the fluid model in the calandria vessel and slosh pressure history for longitudinal direction. The figure also shows the plot for the slosh pressure in the calandria vessel at a particular time point. It was concluded from this study that since the slosh pressures within first two seconds of reactor shutdown are very small, the functional and structural integrity of both shutdown devices is ensured for the 500 MWe PHWR design (Singh et al.,1995).



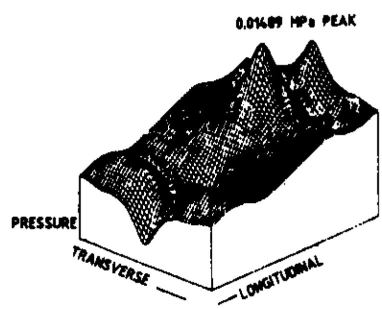
SCHEMATIC DIAGRAM OF CALANDRIA - END SHIELD ASSEMBLY



500 MWe PHWR MODERATOR FLUID MODEL FOR SLOSHING ANALYSIS



SLOSH PRESSURE HISTORY IN LONGITUDINAL MOTION OF CALANDRIA



SLOSH PRESSURE FOR LONGITUDINAL SEISMIC MOTION
(At = 32 Sec.)

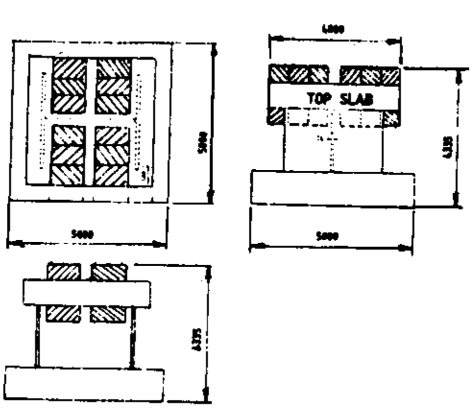
FIG. 11: SLOSHING IN PHWR CALANDRIA

2.10 Benchmarks

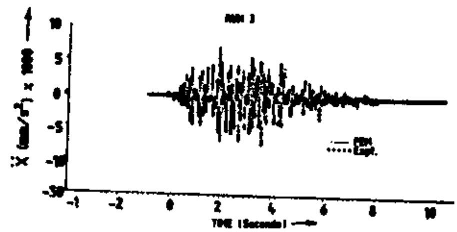
Knowledge base in the field of seismic design of structures and components is continuously growing all over the world. Developments on a variety of issues in this area are being carried out by many groups working independently or in close co-ordiantion. We had an opportunity to participate in one such joint exercise wherein test data for a wide flange I-beam type shear wall subjected to increasing levels of seismic loading was made available to various users for verification by analytical means. This shear wall was subjected to six successively increasing magnitude acceleration time histories till the collapse was observed. The deatils of this experiment are shown in Fig.12. along with summary of various response parameters. The results obtained by the software ABAQUS show fairly close matching with the experimental results. We are also in the process of developing an in-house software ARCOS3D (Madasamy et.al.,1995) to solve this category of problems.

2.11 Seismic Testing

Seismic testing is an area which encompasses many dimensions. It improves the understanding regarding the dynamic behavior of the component. It can also be used for validating the analytical modelling of any structural system. In addition,it can be used for forced vibration test of the component to see its behaviour during a seismic event. Forced vibration tests for seismic loading require large scale tri-axial shake table test facility. We have only one shake table for carrying out seismic testing at DEQ, Roorkee. This table too has limitations in terms of the largest test object which can be tested on it. While there is a case for setting up more such tables, financial constraints would always limit the size of the facility and there would be a number of equipment outside the capability of available shake table facilities. Therefore, it often becomes necessary to use other methods of testing which are quicker and cheaper to use and provide useful experimental backup. Free vibration tests to experimentally confirm the dynamic characteristics and for subsequent evaluation of the forced response analytically, snap back tests; tests using portable shakers at the support points of components etc. are some examples of such other methods.



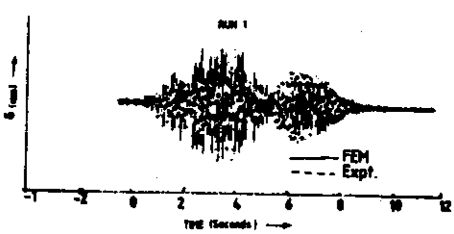
MODEL



ACCLERATION RESPONSE

SEISMIC SHEAR WALL PROBLEM INTERNATIONAL (SSWISP)

RUN No.	QUANTITY	ANALYTICAL	EXPERIMENTAL (NUPEC)
1	MAX. ACCN. (mm/s²)	1860 (3.35s)	2000 (3.30s)
	MAX. DISP (mm)	0.27 (3.35s)	0.29 (3.30s)
	FREQUENCY (Hz)	13.2	13.2
	SHEAR ANGLE (RADIANS)	0.13×10^{-3}	0.14×10^{-3}
2	MAX. ACCN. (mm/s²)	3760	3900
	TIME (SECS)	3.31	3.30
	MAX. DISP (mm)	0.60	0.58
	TIME (SECS)	3.35	3.30
	FREQUENCY (Hz)	13.2	- -
	SHEAR ANGLE (RADIANS)	0.297×10^{-3}	0.290×10^{-3}
2 D	MAX. ACCN. (mm/s²)	6430	6070
	TIME (SECS)	3.90	3.12
	MAX. DISP (mm)	1.30	1.05
	TIME (SECS)	3.90	3.03
	FREQUENCY (Hz)	11.1	11.3
	SHEAR ANGLE (RADIANS)	0.59×10^{-3}	0.52×10^{-3}
3	MAX. ACCN. (mm/s²)	7470	7060
	TIME (SECS)	2.32	2.32
	MAX. DISP (mm)	1.79	1.63
	TIME (SECS)	2.32	2.31
	FREQUENCY (Hz)	9.6	9.0
	SHEAR ANGLE (RADIANS)	0.86×10^{-3}	0.81×10^{-3}



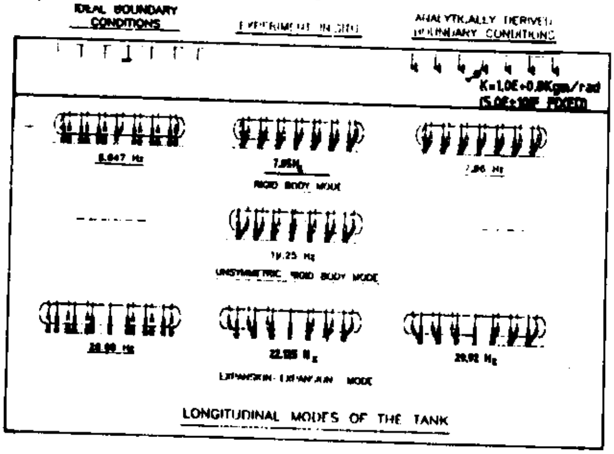
DISPLACEMENT RESPONSE

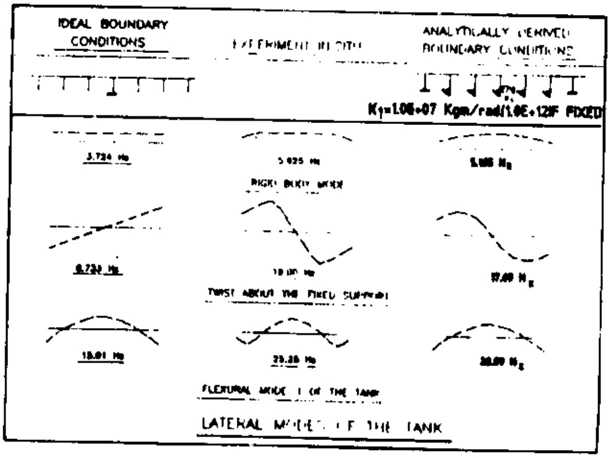
FIG. 12: AN INTERNATIONAL 3-D NON-LINEAR BENCH MARK ON SEISMIC SHEAR WALL

Fig.13 shows dynamic testing performed on a large size horizontal vessel which has seven saddle supports. The central support was fixed to the ground and the other six were roller type supports along the axis of the vessel. The impact hammer test on this vessel revealed that the roller supports do not permit free longitudinal movement of the vessel and that they do offer some resistance during the longitudinal motion of the vessel. The difference in dynamic behavior of this tank evaluated by analytical and experimental means is shown in this figure (Moorthy et al.) In is thus clear that there is no substitute for testing if one wants to know the system's dynamic behavior in a correct manner. This kind of testing also helps in arriving at basic system parameters such as the damping values of piping systems under various levels of excitation. I would like to mention here that this is an area which is still unexplored for Indian piping systems and that we have to heavily depend upon the damping values measured by other countries which may not necessarily hold true in our context.

3.0 CONCLUDING REMARKS

I have attempted to bring to you a flavor of different aspects of aseismic design that we have gone through with regard to Indian PHWR programme. Many of the issues that I have discussed are likely to be of common interest to other industries particularly in the power,chemical plant and similar other areas. It is necessary that developments in the area of seismic design and engineering form the basis of a coordinated research programmes in areas of common interest. This way we all can benefit by pooling our resources together. While significant progress has been made, there are areas which need greater attention. Development of passive snubbers,alternative means of testing,building in greater seismic resistance through cheaper and simpler alternatives,development of well tested criteria for a walk down review of aseismic design are some of the points that come to my mind. Today, perhaps we are spending much larger resources to evolve designs which satisfy ourselves and the statutory agencies, for lack of appropriate technology in some areas mentioned above. You will all agree that development of this technology is not at all difficult. What is necessary is to bring our minds together and work in a coordinated way involving other people who can manufacture and supply good quality products. It also seems





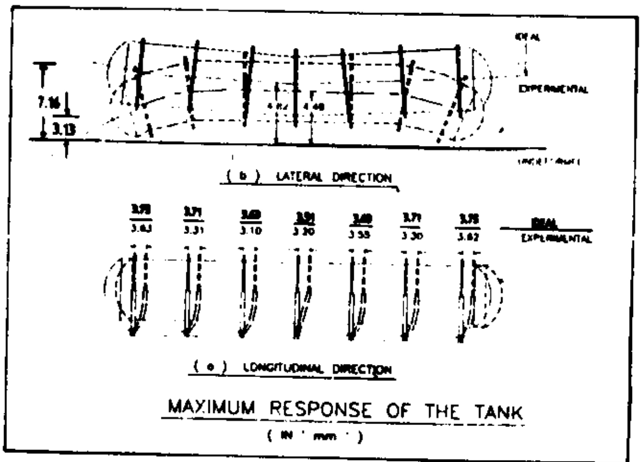


FIG. 13: ROLE OF EXPERIMENTS IN SEISMIC QUALIFICATION OF A TANK

to me that while we are computationally more or less self sufficient, there is considerable deficiency in the our ability to provide necessary experimental backup. This has arisen primarily on account of inadequate resources. Use of innovative approaches will certainly enable us to fill in this gap at least partially with whatsoever resources we can mobilise for this purpose. There is also a question of developing necessary instrumentation within the country. Today we need a variety of transducers, recording facilities with necessary on-line processing capabilities for use during experiments, commissioning of the plant and for reassessment of plant following an earthquake. Many of these items are difficult to procure in view of the technology control regime. Indigenous development of these products is the need of the hour. I hope and wish that the Indian Society of Earthquake Technology takes a lead in bringing together different groups to catalyse some of these activities.

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