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Seismic evaluation of existing nuclear facilities

Proceedings of the SMiRT-14 Post Conference Seminar No. 16 organized by the International Atomic Energy Agency and held in Vienna, 25–27 August 1997

INTERNATIONAL ATOMIC ENERGY AGENCY

March 2001

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FOREWORD

The International Atomic Energy Agency establishes and implements programmes to assist Member States with respect to safety assessment of existing nuclear facilities. Engineering Safety Review Services (ESRS) is one of the elements in these programmes. To ensure an effective and consistent approach in assessing the safety of existing nuclear facilities, relevant requirements and recommendations of IAEA codes and safety guides are implemented.

Very few nuclear power plants are currently being constructed. The recent ESRS review missions are mainly related to seismic re-evaluation of operating nuclear facilities. In the past decade, re-evaluations of seismic safety of WWER type nuclear power plants have been the primary focus of ESRS review missions.

Since 1992 the IAEA has been assisting Member States to develop plant specific guidelines used in the post-construction seismic safety re-evaluation. Working together with many experts in this field from Member States, the IAEA developed technical guidelines to establish a general framework within which a seismic re-evaluation of an operating nuclear power plant can be carried out. These technical guidelines will form the basis of an IAEA Safety Report on seismic evaluation of existing nuclear facilities.

To exchange information and share valuable experience among the Member States, the IAEA organized and hosted this post conference seminar on the subject as part of the activities of Structural Mechanics In Reactor Technology (SMiRT). This is the third time that experts involved in seismic re-evaluation and upgrading of operating nuclear facilities convened to discuss issues of mutual interest and the experience that they have gained after first meeting in Vienna, Austria, in 1993 at SMiRT-12 and then in Iguazu, Argentina, in 1995 at SMiRT-13.

The main co-ordinators of the seminar were A. Gürpinar and A. Godoy from the IAEA. Other members of the coordination committee included N. Krutzik (Germany), J. Johnson (USA), J.D. Stevenson (USA), H. Shibata (Japan), T. Katona (Hungary) and M. Zola (Italy).

The IAEA officers responsible for this publication were A. Godoy, A. Gürpinar and P. Contri of the Division of Nuclear Installations Safety.

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SUMMARY

BACKGROUND

The SMiRT 14 Post Conference Seminar No. 16 on Seismic Evaluation of Existing Nuclear Facilities was held in Vienna, 25–27 August 1997, and logically follows the previous two post-SMiRT seminars held in Vienna and Iguazu with the same title.

The scientific community, which convened in Vienna, was composed of 76 specialists from 26 countries representing regulatory bodies, electrical utilities, engineering companies and suppliers. Forty papers were presented and discussed in plenary sessions, panel sessions and panel discussions.

Most of the papers are connected to IAEA review activities in the field of the seismic re-evaluation and upgrading of existing plants carried out in recent years. Therefore, together with some general papers on criteria and methodologies, many papers deal with national experience which is the essential background for the IAEA in the development of a unified approach to the seismic re-evaluation of existing facilities, applicable to WWERs, CANDU, PWRs, etc.

The sessions reflect the variety of topics which have been considered in the seminar. In particular, three main topics attracted the most attention and discussion. These were; the draft Safety Report on the seismic re-evaluation and upgrading of nuclear facilities under development by the IAEA, the results from the IAEA Co-ordinated Research Project (CRP) entitled "Benchmark Study for the Seismic Analysis and Testing of WWER Type NPPs", and the progress of seismic re-evaluation and upgrading programmes from a number of nuclear power plants. For many of the latter this involved the implementation of the recommendations from IAEA Seismic Safety Review Services.

It is important to note, however, that the seminar was not restricted to a report on IAEA related activities; there were a significant number of contributions form the scientific community on the current state of the art in seismic re-evaluation.

As mentioned above, a number of papers in the seminar deal with the CRP on benchmark study for the seismic analysis and testing of WWER type NPPs organized by the IAEA (1993–1997). It offered the opportunity to many specialists to review and assess their methodologies. Two types of WWER reactors (WWER-1000 and WWER-440/213) were selected as prototypes for benchmarking: Units 5/6 of the Kozloduy NPP and the Paks NPP.

The main objective of the CRP was the meeting among utilities, safety authorities, engineering companies and suppliers involved in seismic re-evaluation programmes for WWER type plants. The scientific framework aimed at a harmonization of the methodologies to be used in such programmes and to their validation through dedicated exercises and in general through the experience that many Member States were accumulating in actuality.

The focal activities of the CRP were the benchmarking exercises. A similar methodology was followed both for Paks NPP and Kozloduy NPP Unit 5: The NPP (mainly the reactor building) was tested using a blast loading generated by a series of artificially generated ground explosions. The participants had to make a blind prediction of the structural response and their analytical results were then compared with the results from the test.

Twenty-four institutions from thirteen countries participated in the CRP through either a research contract or a research agreement. Two other institutions (both from Japan) contributed to the CRP informally and on a voluntary basis.

This scientific work highlighted the reliability of the available numerical tools, the need for further research, and a general judgement on the best compromise between experimental and numerical tools in the seismic re-evaluation processes.

The final results of the CRP are presented in IAEA-TECDOC-1176, "Benchmark Study for the Seismic Analysis and Testing of WWER type NPPs" (October 2000).

STRUCTURE

The meeting was a valuable opportunity to discuss the status of the seismic re-evaluation and upgrading activities for many nuclear power plants A general survey of the ongoing work is provided in Session I. Session II is dedicated to the results of the CRP, as some of the participants to the seminar also took part in the benchmark exercise, organized by the IAEA. Major outcomes from the ongoing seismic upgrading activities are presented in Session III, where the IAEA efforts to reach a consensus on a shared approach to both seismic re-evaluation and upgrading is the main focus. The "unified criteria documents" developed for some Eastern European NPPs are intended to be used in the development of an IAEA safety report on the seismic evaluation of existing nuclear facilities. The seminar served as a useful means to have international expert opinion on the development of this safety report.

Session IV is dedicated to the proposal for the development of special databases for NPP component seismic data to be used as a tool for the seismic re-evaluation of existing components and equipment by applying the similarity criterion. Such approach is foreseen in many general documents, but the limited availability of basic experience data limit its application in practice, especially for the WWER type nuclear power plants. The technical implications are discussed, the difficulties in data recovery are outlined and some elements for a financial evaluation are also provided.

Session V presents discussions on emergency preparedness measures and seismic warning systems. Both traditional (automatic scram of the reactor following an earthquake occurrence) and more sophisticated approaches for a warning system are discussed.. Safety and operational aspects are presented for a global evaluation of their effectiveness and safety relevance.

At the date of publication of these proceedings, the information collected is still valuable for the orientation of the long term objectives of the IAEA's support to the seismic re-evaluation processes in progress.

Most information provided in the TECDOC is also valid and can be used as a basis for any study and improvement in the field of simulation and qualification methods.The overview of the world experience still represents an essential background material, substantially unchanged, with many state of the art-surveys and useful "position papers".

The main message left by the meeting is associated with the usefulness of the IAEA role in the context of the harmonization of seismic re-evaluation where different approaches, traditions and requirements may often lead to different evaluations of plant safety. Seismic upgrading of a nuclear power plant can be a costly investment and justifies the refinement of the analysis design methodologies, as discussed in this publication. The identification of the safety issues, to propose focused upgrading measures and to finally provide the international community with a reliable measurement of the improved safety level of the nuclear plants have been the major outcomes of this seminar.

OPENING ADDRESS

THE IAEA'S NUCLEAR SAFETY PROGRAMME

A. Carnino
Division of Nuclear Installation Safety
International Atomic Energy Agency

The future of nuclear energy depends on three main factors, namely:

- Nuclear Safety: prevent accidents and demonstrate excellence in safety (Safety Culture)
- Economics: electricity deregulation, modernization and life extension of existing (old) plants, decommissioning
- Public Acceptance

In this context, the vision and role of the IAEA is to work towards the International Harmonization of Nuclear Safety.

The specific objectives of the IAEA activities are to:

- Strive for excellence in safety for all nuclear installations worldwide with emphasis on safety culture.
- Establish with Member States based on the Agency standards, a "reference basis" for evaluating the safety level of their installations.
- Demonstrate safety measures under harmonization of nuclear safety worldwide.

Achieving these objectives will also increase public understanding and confidence in nuclear safety.

Therefore the priorities for the next budget and programme cycle (1999-2000) for the Agency's Nuclear Safety Programme are on:

- revision of its nuclear safety standards
- strengthening regulatory bodies in Member States
- operational safety
- a strategy for nuclear safety assistance to developing countries
- safety culture enhancement
- identification and prioritization of key, safety issues including tools and analysis methods development
- response to requests of analysis of unusual events
- develop national capabilities in self-assessment
- service the Convention of Nuclear Safety

In order to best serve the interest of its Member States, much of this work has already been included in the revised Programme for 1998.

With regard to the Convention on Nuclear Safety, the role of the IAEA Secretariat is to convene, prepare and service the meetings of the Contracting Parties (CP) and transmit to the CP information received or prepared in accordance with the provisions of the Convention.

At the Preparatory Meeting held in April 1997, three documents have been agreed:

- Rules of Procedure and Financial Rules
- Guidelines Regarding National Reports
- Guidelines Regarding the Review Process

According to the Guidelines for the preparation of National Reports, the status of existing Nuclear Installations should be summarized, including, where necessary, upgrading measures to achieve a high level of nuclear safety or, if such upgrading cannot be achieved, plans to shut down the Nuclear Installations as soon as practically possible as described in Article 6 of the Convention.

Other information required includes legislation and regulation, general safety consideration, and safety of installations including the implementation of the "defence in depth concept".

STATUS OF SEISMIC RE-EVALUATION AND UPGRADING PROGRAMMES OF SELECTED NUCLEAR FACILITIES

(Session I-1)

XA0100498

SEISMIC ASSESSMENT AND UPGRADING OF PAKS NUCLEAR POWER PLANT

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Abstract

A comprehensive programme for seismic assessment and upgrading is currently in progress at Hungary's Paks NPP. The re-evaluation of the site seismic hazard had been already completed. The technology of safe shut down and heat removal is established and the systems and structures relevant for seismic safety are identified. A seismic instrumentation is installed. The pre-earthquake preparedness and post-earthquake actions are elaborated. The methods for seismic capacity assessment are selected. The seismic capacity evaluation and the design of upgrading measures are currently in progress. The easy to perform upgrading covering the most urgent measures had been already performed.

1. INTRODUCTION

In the late 1980s it was recognised that the Paks site seismic hazard may be much higher than that assumed in the design. In 1993 a preliminary study of the site seismicity gave the basis for a resolution on the seismic safety of Paks NPP issued by the Hungarian Atomic Energy Commission's Nuclear Safety Directorate. As a response the NPP launched a comprehensive programme for seismic assessment and upgrading of the plant which is due to be implemented on all of the units by the year 2002.

Here, an overview of the seismic assessment and upgrading of Paks VVER-440/V213 units is given.

2. OBJECTIVES AND SCOPE OF THE SEISMIC SAFETY PROGRAMME

The basic safety requirements are: to ensure safe shutdown, to cool down and remove any decay heat, and to limit radioactive release. In order to achieve these goals the seismic capacity reassessment and the upgrading may be performed by applying specific methods based on the possibilities and limitations of the present operating plant rather than on the requirements applicable to a new design.

The seismic safety programme includes the following tasks:

- re-evaluating of the site seismic hazard, including the geotechnical survey of the site, analysis of ground and foundation stability, liquefaction, settlement, sliding, etc.
- establishing the technology of safe shut down and heat removal, elaborating the list of structures, systems and components relevant for ensuring seismic safety,
- installing seismic instrumentation, elaborating pre-earthquake preparedness and post-earthquake actions,
- evaluating the seismic capacity of systems and structures relevant for safety,
- performing the necessary upgrading measures, prioritising the measures needed, and carrying out the urgent and easy-to-perform fixes as soon as possible even if only preliminary seismic input is available.

Re-qualification of the plant for two earthquake levels, i.e. for the safe shutdown earthquake (SSE) and operating basis earthquake (OBE) level, is not feasible. The basic issue is to re-qualify the plant for the new design base (DBE or SSE) level. The level of safe continuous operation of the plant which is not designed for an OBE would be defined on the basis of the plant capacity assessment experience.

The seismic safety programme is an important part of the overall safety enhancement programme of the Paks NPP. The implementation of the seismic safety programme is harmonised and synchronised to the implementation of other safety upgrading measures and projects which may also affect the seismic safety of the plant. The seismic requirements are taken into account in the ongoing reconstruction of the reactor protection system which reduce essentially the re-qualification and upgrading needs in C&I area. The replacement of the emergency feed water system from the longitudinal gallery building and turbine hall to a safe position under the localisation tower decreased also the seismic safety relevance of these parts of the main building. This modification allows also the cool down by bleed and feed process after an earthquake.

3. SITE SEISMIC HAZARD RE-EVALUATION

Prior to completing the site seismic hazard re-evaluation a conservative review level earthquake (RLE) had to be defined for the preliminary margin evaluation and for realising the most urgent and easy-to-perform fixes. The NUREG/CR-0098 soft site median spectrum was selected for the 0.3 g level as input for the screening, and the 0.35 g US NRC Regulatory Guide 1.60 response spectrum was used to design the easy-fixes.

The 10^{-4} annular non exceedance probability event has been defined as the safe shutdown or design base earthquake (DBE) and characterised by best estimated Uniform Hazard Response Spectrum (UHRS). In evaluating the seismic hazard the probabilistic method was applied because of the seismotectonic features of the Pannonian basin. The result was also compared with the 84% confidence level deterministically defined response spectrum. The UHRS has been calculated for the Pannonian level, 30 m below the free surface.

The free field spectra were obtained by non-linear calculation because of the soft nature of the uppermost 30 m thick soil layer. A probabilistic approach was applied to assess the uncertainties of the soil properties obtained from a state of the art geotechnical survey. The DBE ground peak acceleration (GPA) was found equal to 0.25 g, ten times more than the original design assumption.

The soil at the Paks site is soft, the shear wave velocity in the upper 30 m sandy deposit is around 300 m/s, and the groundwater table is high. The liquefaction potential of the soil has been evaluated in terms of the annual probability for liquefaction to occur. The soil below free field at depths of 10 to 20 m has a best estimate return period of liquefaction between 11,000 and 14,000 years. Soil under the NPP has a somewhat lower likelihood of liquefying with the best estimate return periods being between 15,000 and 18,000 years. Consequently, assessing the plant safety a global liquefaction of the soil should not be taken into account.

4. SAFE SHUTDOWN TECHNOLOGY

A Seismic Safety Technological Concept was developed by Paks NPP which define the method of ensuring the safety of the plant during and after an earthquake. The concept is supported by extensive safety analyses.

According to the Concept the reactor shut down and the stable subcriticality could be maintained by the reactor control and protection system together with the boron system, cooling down of the reactor could be made by secondary side bleed and feed. It would be possible to ensure a continuous decay heat removal by the low pressure emergency core cooling system heat exchanger after some modification. The Concept includes measures for mitigation of small LOCA, and also measures for containment isolation and prevention of radioactive releases. The systems mentioned above as well as the supporting systems (C&I, energy supply, cooling, lubricating, etc.) and also the necessary monitoring systems have to be re-qualified for the new DBE level together with the relevant building structures. These systems and structures form the first seismic category, where the requirement on functionality and/or integrity of each item is defined. The seismic margin of systems and structures classified should be evaluated. If necessary the systems should be re-qualified for the actual seismic level by fixing or replacement. Seismic interactions should be taken into consideration. Those systems or parts of systems not important for safety should be separated from the upgraded part by quick closing valves. System redundancy relevant for seismic safety matches the general safety philosophy of the plant, i.e. 3 times 100% redundancy should be maintained. The DBE should not be combined with Loss of Coolant Accidents.

A comparison of the Concept with internationally established requirements and practice demonstrates that the Concept significantly exceeds the minimum requirements, e.g. ensuring the decay heat removal over 72 hours and practically without limitation in time usually is not required.

According to the safety significance the systems specified by the Concept are separated into three priority groups. The safe shut down systems (i. e. systems for ensuring the control of the reactivity, primary pressure and reactor coolant inventory, and for the decay heat removal) have the highest priority.

There are methods for the cool-down and decay heat removal other than those specified in the Concept. The reactor cool-down feed and the continuous decay heat removal may be ensured after an earthquake upgrading the operational heat removal system for the required seismic level. A comparison of different methods and a cost-benefit analysis is recently in progress.

The list of seismic safety relevant structures and equipment is stored in form of a database which consists also the important for the project management information (i.e. priority, function, location, documentation, results of walkdowns, analyses, contracting information etc.).

5. SEISMIC CAPACITY EVALUATION

In 1993-1996 the capacity of the safety related systems and structures was evaluated using a conservative input. The reason for the preliminary investigations was to select the appropriate methodology, to develop adequate models, and to obtain information about as-built conditions.

The final capacity assessment of relevant systems and structures in relation to final seismic demand is currently in progress.

5.1. Methodology

Following the advises the IAEA the seismic re-qualification techniques developed for operating plants such as the Seismic Margin Assessment (SMA) method and the experience based re-qualification (SQUG) technique have been adopted at Paks. The limits and conditions of the applicability of the re-qualification methods have been defined by means of systematic analysis and comparison of the US and Soviet design codes and procedures type by type for all relevant equipment classes, distribution systems and structures.

5.2 Capacity Evaluation of Building Structures

The main building is a set of coupled structures having a separate foundation and widely varying rigidity, and the distribution of the stiffness and masses is highly complex. The problem of optimal modelling of coupled structures with very different characteristics and also the adequate modelling of twin main buildings on a common base mat had to be solved. Various calculation techniques, such as the response spectrum method and the time history method have been studied in order to determine the most cost effective yet least conservative evaluation method. In the case of the main building structure the soil–structure interaction is modelled through the introduction of the frequency dependent dynamic stiffness matrix obtained for all points of the structural model in contact with the soil, and the equations of motion are solved in the frequency domain. This approach leads to an essential reduction in conservatism compared with the routine calculation methods. The analyses of the structures response and capacity for the final input is currently going on.

From the system point of view the most critical structure is the longitudinal gallery building housing many systems and items of I&C equipment vital for safety. Relocation of the emergency feed water system from the longitudinal gallery will reduce the safety relevance of this part of the building, but will not completely eliminate the problem. The results of the calculations show that this part of the main building has to be upgraded. The reactor hall steel frame structure may need a number of fixes just to avoid falling the non-structural roof and side panels. Similarly, the turbine hall is also vulnerable but it does not house vital equipment except for a limited part where the service water lines cross the hall.

For the upgrading of the main building two different concepts were elaborated: One is based on the idea of transferring the load from the turbine hall, intermediate building (transverse gallery) and reactor hall to the very rigid reinforced concrete localisation towers. The other solution is to fix both reactor and turbine hall and due to this longitudinal gallery as well. The solutions are based on adding new structural elements, strengthening the main bearing elements of the structure, i.e. x-bracing, jacketing, improving of joints, etc.

A particularly important question is the probable change in the leakage rate of the pressure boundary of the VVER-440/V213 containment due to earthquake loads. A study of the potential leakage spots has been started. The first results were reported recently at SMIRT Conference. According to this results an essential growth of the leakage rate is not to be expected.

5.3 Equipment, Piping, I&C

For the dynamic analyses of the primary system (loops, steam generators, etc.) a coupled model was developed that comprises the reactor building reinforced concrete structure together with the components of the primary system. The purpose of this model is to provide a less conservative seismic load on the primary system on the one hand and estimates for the displacements for the evaluation of interaction effects on the other.

A concept of upgrading of the primary system by viscodampers has been elaborated. In each loop the steam generator has to be fixed by six viscodampers and one damper has to be applied on the cold leg. The detailed design work is started.

In the case of most seismic safety related systems equipment vulnerability mainly stems from its anchorage which was not designed for seismic loads. The pipelines are flexible and are subjected to low frequency resonance's because of the long runs between fixing points. In the original design a considerable number of simple and spring hangers were applied, and there are no snubbers at all.

Analysis shows that in some cases the support spacing of the existing lines is too large and additional supports or dampers have to be placed.

To assess the functionality of active equipment the experience based (SQUG) method has been applied. Although the preliminary studies demonstrate the viability of this method for most of the classes of equipment, some important items, e.g. relays, may need additional consideration because of their design features. In some cases the shaking table test may be an appropriate method for qualification. For instance, the most important relays, I&C and electrical equipment have been tested on the shaking table.

Replacement of the old equipment is being considered as an alternative to re-qualification, e.g. the ongoing reconstruction of the reactor protection system is performed taking into account the actual seismic requirements consequently the scope of I&C seismic re-qualification is quite limited.

A special topic is the qualification of the I&C and electrical equipment mounted in the already fixed, mainly top braced racks and cabinets.

5.4 Full-scale and model tests

The VVER-440/V213 building response has been studied by means of full-scale blast tests. Three series of large (up to 500 kg of TNT) explosions were carried out and the acceleration responses at characteristic points of the building structures and also the response of some large components, e.g. the Emergency Core Cooling System (ECCS) tank were recorded and analysed. For investigating soil–structure interactions the acceleration at different levels in bore holes was measured too. The full scale test results were used to check the structural model of the main building complex.

During the full scale blast test the response of the worm-shaped large low pressure ECCS tank was measured. A 1:3 scale model of this tank has been tested on the shaking table at the National Research Institute for Earth Sciences and Disaster Prevention in Japan. Comparison of the results of these two tests as well as dynamic calculation of the tank gave information concerning the behaviour of the structure and fluid–structure interaction.

6. SEISMIC UPGRADING

Making use of the international experience the items in the list of systems, structures and equipment relevant for seismic safety at NPP Paks were classified into two groups:

- the so called "easy-fix" items requiring simple seismic upgrading that can be accomplished comparatively easily and can be done during normal outage periods or even during operation; the design solution and the cost of these fixes do not depend very much on the seismic input
- all other items which may need sophisticated evaluation and input dependent and cost sensitive upgrading.

Those easy to perform upgrading covering the most urgent measures had been already realised in 1994-1995 before the completion of the site seismic hazard studies. Selection of the easy-fix items was performed on the basis of simplified capacity-demand calculations and detailed plant walkdowns. For screening the GPA of the RLE the NUREG/CR-0098 soft site median spectrum was selected for the 0.3 g level. For designing of the fixes the 0.35 g US NRC Regulatory Guide 1.60 response spectrum was used.

One of the main findings of the screening was that the I&C racks and cabinets, and also the batteries are poorly fixed. Practically all of the safety related electrical and I&C cabinets have been improved by adding new anchorage at the bottom or as a top bracing. In all cases the support spacing of the cable trays was found to be too large and additional supports had to be placed. Because of poor anchorage some of the mechanical equipment needs additional fixes.

Low seismic capacity masonry walls separating the different compartments in the gallery buildings had to be fixed to avoid any interactions with safety related equipment. The safety related batteries were replaced during the easy-fix phase, too. The easy-fix work for the four units of the NPP is listed in the Table 1.

Table 1.

The easy-fixes for the four units of the Paks NPP

Number of items checked	10184
Number of "easy fix" cases	5507
Mechanical equipment	202
Electrical equipment	465
Cable trays	2498
I & C (racks, cabinets)	2061
Masonry walls	281
Weight of steel frames built in	445 t

Design work related to the somewhat more sophisticated fixes was recently started following the final capacity calculations.

7. PRE-EARTHQUAKE PREPAREDNESS AND POST-EARTHQUAKE ACTIVITIES

In 1993 separate seismic instrumentation was installed at each unit of the NPP. This instrumentation consists in each case of seismic switches mounted on the base mat, sensitive accelerometers registering the response at the characteristic points of the structure, an appropriate data collection system, and a voting logic. Two free field stations are installed at the plant too. In 1993-1997 concept of the manual shut down was introduced.

Recently the concept of determining the OBE exceedance based on the response spectrum and cumulative absolute velocity criteria has been implemented. An emergency procedure exists which determines the post-earthquake action of the plant personnel. A comprehensive guide has been elaborated to assess the post-earthquake situation at the plant.

A new concept has to be implemented in the future together with the realisation of the system modification necessary to ensure heat removal from the reactor after an earthquake. In such a case the seismic instrumentation would trigger the isolation of the fixed systems from the non-fixed ones.

The basic question of plant response to an earthquake is how to define the level of safe continuous operation of a plant not designed and not re-qualified for any OBE. The real basis for determining the earthquake level of continuous safe operation could be the results of capacity evaluation of the safety relevant systems of our four VVER-440/V213 units and other VVER-440 plants as well as the experience behind the response spectrum and the cumulative absolute velocity limits.

8. ROLE OF INTERNATIONAL CO-OPERATION

International co-operation and technical aid are of major important for the realisation of the seismic safety programme at Paks.

The basic principles for seismic safety as well as the re-qualification philosophy and technique of the Paks NPP follow the recommendations of the IAEA. The IAEA plays an important part in transferring the best international practices in seismic hazard re-evaluation and upgrading. IAEA reviews and follow up missions both aid and check the NPPs activity. Moreover, the IAEA has an important role in co-ordinating the work of all VVER-440 plants.

By means of its PHARE programme, the Commission of the European Communities supports Hungary's seismic safety programme. The site seismic hazard re-evaluation at Paks NPP serves as an example of a successfully performed PHARE project. A new PHARE project has been launched for to assist plant re-qualification; a number of such projects are under preparation.

The help of the Japanese government in qualifying Paks personnel and in transferring knowledge is also of great significance. The shaking table experiment mentioned above is also a good example of support and co-operation.

9. CONCLUSIONS

The main results of the seismic assessment and upgrading at NPP Paks can be summarised as follows:

The safety related systems and structures of the plant have been analysed for the RLE.

The reinforced concrete part of the reactor building which forms the sealed containment of the VVER-440/V213 seems to have sufficient capacity. Those structures of the main building which are the most vulnerable are attached to the reinforced concrete reactor building, i.e. the gallery buildings, the reactor hall and the turbine hall. For these structures the design solutions are currently being elaborated and realised. Upgrading of the non-structural elements has proved to be especially important in order to prevent interactions with safety related equipment. The non-structural masonry walls in the vicinity of safety related equipment had already been upgraded.

The equipment and piping of the primary system have sufficient capacity. Visco-dampers are considered for upgrading. In many cases equipment anchorage is in need of upgrading. Anchorage for highly critical electrical and I&C equipment have been already fixed already in the framework of easy-fix projects.

The plant now has appropriate seismic instrumentation. The definition of the scram level of the units not being designed for an OBE is an essential problem to be solved.

Seismic re-evaluation and re-qualification of the units of Paks NPP units pose a complex problem which can be solved by adopting international experience, methods and requirements and by taking into account the design features of these and other such VVER units as well as the as-built and current conditions.

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STATUS OF SEISMIC RE-EVALUATION AND UPGRADING OF KOZLODUY NPP

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Abstract

The changes in the safety policy of Bulgaria in the eighties resulted in the implementation of the international safety requirements for operating NPPs and in re-evaluation of the seismic safety importance. It was estimated that the site seismic hazard may be much higher than it was assumed in the design. Complying with the international practice, broad scope studies were started for the seismic qualification of essential systems and structures. Many upgrading measures were launched.

1. INTRODUCTION

The Kozloduy NPP is situated in the north-west of Bulgaria on the bank of the river Danube. It was built and equipped in compliance with a Russian design. There are six units in operation, which were commissioned within a long time period.

No	reactor type	commercial operation	original PGA
1.	WWER 440/230	10.1974	NED*
2.	WWER 440/230	11.1975	(<5°/MSK-64)
3.	WWER 440/230	12.1980	0.1g
4.	WWER 440/230	06.1982	(<7°/MSK-64)
5.	WWER 1000/230	11.1987	0.1 (0.2g)
6.	WWER 1000/230	12.1993	(<8°/MSK-64)

During this period the assessment of the seismic hazard at the site was changed. This led to changes in the criteria and methods for seismic design as well as in the seismic re-assessment of the structures, systems and components (SSC).

The Kozloduy NPP was affected during its operational time by four strong earthquakes in 1977, 1986 and twice in 1990. The first two WWER 440 type units were designed according to the standard building practice. The seismic intensity of the site was assessed as equal to IV-V grade on MSK-64 scale, i.e. aseismic design requirements could be totally ignored. After the Vrancha earthquake the seismicity of the site was re-assessed and the SSE was set to VII intensity grade on the MSK-64 scale. The corresponding peak ground acceleration (PGA) was defined as 0.1g. It resulted into some modifications in the design of units 3 and 4 and they were put into operation taking into account the new seismic inputs. Seismic instrumentation and

*NED - not explicitly designed against earthquake automatic scram system was installed at the plant and upgrading of some structural elements and equipment was performed, e.g. hydraulic snubbers were installed on Steam Generators, main circulation pumps and primary loops for all four units.

An overview of the seismic re-evaluation and upgrading programmes being realised at Kozloduy NPP after 1990 is presented in this paper.

3. GOAL

The main goal of the seismic re-evaluation and upgrading is to provide the performing of the defined main safety functions in RLE.

Frequently, as a result of the collected data analysis, quick, easy and cheap upgrading of elements not included in the safety systems is implemented and considerable safety improving is achieved.

4. MAIN PRINCIPLES AND APPROACHES IN PLANNING AND IMPLEMENTATION OF THE ASEISMIC ACTIVITIES AT KOZLODUY NPP

The year 1990 was a milestone regarding the aseismic activities at Kozloduy NPP. The latest earthquake with epicentre Vrancha was in this year. Units 1-4 were shutdown automatically. PGA of 0.046g was recorded.

The same year, after inspection of the site, the IAEA presented to the Bulgarian Government a report, where the seismic safety of Kozloduy NPP was assessed as insufficient.

The Bulgarian authorities in close co-operation with the IAEA took a decision in principle to start immediately the implementation of a comprehensive programme including studies and activities for safety upgrading of the Kozloduy NPP site regarding the external impacts, considering the seismic impacts as a priority.

The following principles and approaches were observed in planning and implementation of the further activities:

- close co-operating with the IAEA;
- maximum applying of the experience of the international companies and western experts and technical recommendations;
- combining different methods and approaches for seismic re-evaluation and designing of upgrading depending on the actual stage at General Workplan Flowchart;
- maximum conservative approaches during the first stages;
- maximum realistic approaches in the later stages after collecting sufficiently confirmed results from the analysis, inputs, as-built/design data;
- cost-benefit-terms analysis;
- maximum usage of the annual outages for implementation of designs for upgrading;
- improving the seismic safety by replacement of the elements that have insufficient seismic capacity with seismic qualified elements;
- seismic safety upgrading simultaneously with the systems' reconstruction's and modifications (equipment and pipes replacement, changes of the configurations and connections of the systems), addition of new systems and equipment.

5. STAGES OF THE ACTIVITIES FOR SEISMIC RE-EVALUATION AND UPGRADING OF KOZLODUY NPP TILL NOW

5.1. STAGE 0

The IAEA Project BUL 9/012 was launched in 1990. The Bulgarian Geophysical Institute at BAS and "Energoproject" were engaged. The main objectives defined by this programme related to the seismic safety of Kozloduy NPP were geotechnical data collection and site seismic hazard re-evaluation.

An IAEA mission was held in April-May 1991. A preliminary seismic ruggedness evaluation of units 1 to 4 was carried out by external experts from EQE International and Westinghouse. A list of safety related equipment and structures was presented. The seismic capacity of the items in the list was determined.

5.2. STAGE 1

A comprehensive WANO programme for upgrading of the operational reliability and safety of Kozloduy NPP was launched in April 1992. This marks the beginning of Stage one of the activities.

Item HB of the WANO programme is related to the seismic safety. Its implementation was funded by the PHARE programme for the needs of the contract. The IAEA co-ordinated the development of Terms of Reference and Technical Specifications (TOR) for Seismic Upgrading Design of Kozloduy NPP for Units 1 and 2.

A wide NPP own programme was initiated simultaneously with the WANO's one. The above mentioned TOR governs these programmes and all the following seismic related activities, regarding research and design. The scope of the item HB contract is limited to units 1 and 2. The NPP programme covers the same activities for units 3 and 4 as the WANO's one and the general tasks related to units 1 to 4.

As contractors of the WANO programme were involved WESE, EA and Energoproject - Bulgaria. EQE International was the main contractor of the NPP programme at this phase. Full walkdowns were performed. As-built and design data were collected.

Seismic anchorage upgrades for the weakest equipment identified in the IAEA reports (Stage 0) were designed. This task resulted in fixes based on conservative criteria, usually called "easy fixes".

Detailed seismic upgrading designs of DG-2 and Pump House 2 buildings were prepared.

Functions, systems and components classification was carried out.

A safe shutdown equipment list (SSEL) of mechanical, electrical and I&C equipment was developed. The seismic upgrades were prioritized accordingly.

The site seismic hazard re-evaluation was finalized approximately at the same time and new seismic input (RLE, SL-1, SL-2) was defined.

The Bulgarian Building Research Institute developed specific site response spectra, approved by the IAEA mission at the end of May 1992. On this basis the seismic evaluation was to be conducted for a Safe Shutdown Earthquake anchored to 0.2g horizontal peak ground acceleration with 50% of this value for the vertical component.

All subsequent research and design activities were based on the newly determined seismic parameters.

5.3. STAGE 2

Design of seismic upgrades for "Priority 1" items was carried out. The implementation of both low capacity equipment designed in Stage 1 and "Priority 1" items was initiated in 1992.

Soil liquifaction study for the site with emphasis on Pump House 1 and the channel going to the Danube river was conducted.

The structure capacity of the following buildings was evaluated:

- within the scope of the WANO Programme:

  - Diesel Generator Building - 1
  - Pump Station Building - 1
  - Main Building - Units 1 and 2

- within the scope of the NPP own programme:

  - Spent Fuel Storage Building
  - River Bank Pump House

Detailed seismic upgrading designs of the above mentioned buildings, excluding the Main Building, were created.

In-structure response spectra were generated for all six units and for the Spent Fuel Storage Building. They were further used for the qualification of equipment and systems.

Plane models both in transversal and longitudinal direction were used for Units 1 and 2. The in-structure spectra for the remaining structures were developed on the basis of complete 3D finite element models.

The effect of seismic excitation from local earthquakes on the structures and equipment of NPP was analysed additionally according to the IAEA recommendations.

A seismic hazard analysis on the site was carried out and hazard curves were determined. The influence of local earthquakes on the already generated in-structure response spectra was estimated.

Re-assessment of previous projects for local earthquakes was prepared. By Risk Engineering LTD and EQE-Bulgaria. Some changes in few elements was recommended and implemented.

5.4. STAGE 3

In the scope of IAEA Programme BUL 9/012 in NPP KOZLODUY are well-grounded three basic seismic instrumentation systems:

- Seismic scram system (SIAZ), modernization (Fig. 1);
- Seismic monitoring system for strong motion (SASKOK) (Fig. 2);
- Local Seismological Monitoring Network (LSMN) at KOZLODUY NPP.

One system SIAZ on each unit 1-4 has been installed that is entirely equipped with components by Kinemetrics, USA. Each system has 3 strong motion station at a distance of 200 - 500 m, connected through cables to the central recording panel.

In each of the seismic stations are installed:

- one three-component accelerometer FBA-3, recording the accelerations during an earthquake;
- one three-component trigger TS-3, automatically switching on the recording system at acceleration over 0.01 g;
- one three-component seismic switch, transmitting the signal for shut down of the reactor, if the earthquake acceleration exceeds the specified level :

	old threshold	new threshold
unit 1 and 2	0.035g	0.046g
unit 3 and 4	0.035g	0.065g
unit 5 and 6	0.050g	0.083g

In accordance with recommendation of IAEA on 02.1993. at KOZLODUY NPP was put into operation a new SASKOK system.

SASKOK system includes three model KINEMETRICSs accelerographs, as follows:

- 4 accelerographs SMA-1 (optical record on 70mm photographic film on three-component accelerograms)
- 3 accelerographs SMA-2 (magnetic analogue record on audiocassette on three-component accelerograms)
- 4 accelerographs SSA-2 (recording conversioned data from the accelerometers in digital form on instruments RAM-memory)

For all of the instruments the trigger level is defined 0.01g. Only the outdoor accelerograph SSA-2, for unit 3 has defined trigger level 0.005g. This gives a possibility to register weak earthquake.

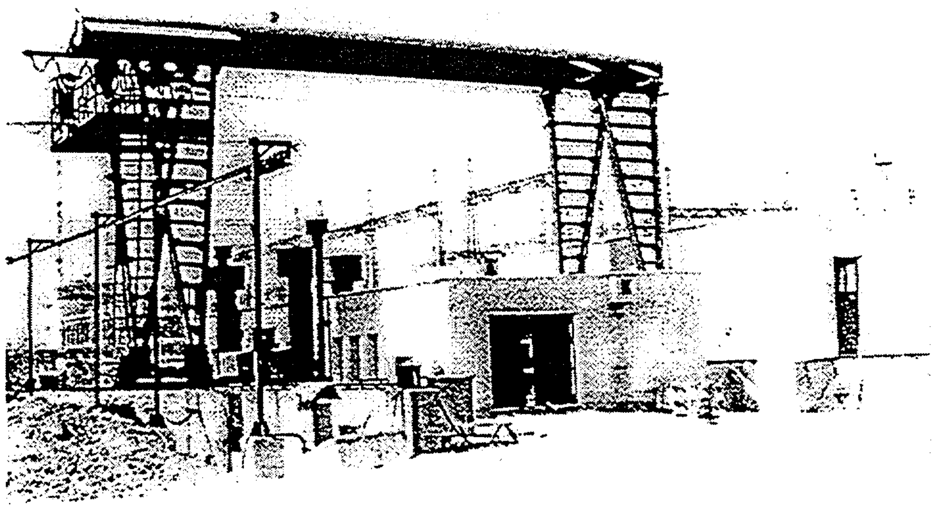


Figure 1 Fire Protection Pump Station 2/FPPS-2/ in Construction

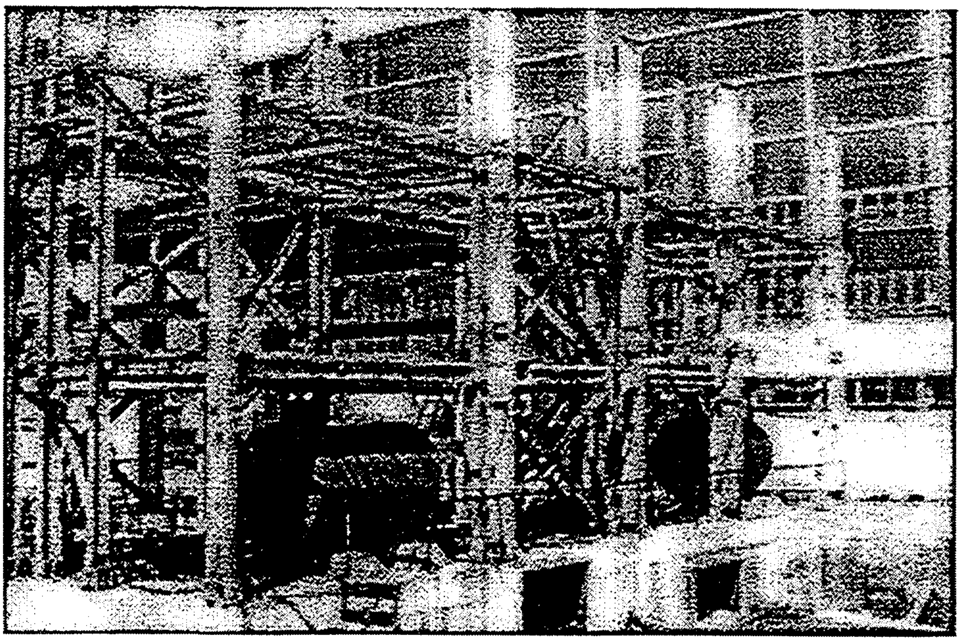


Figure 2 Building of Auxiliary Emergency Feedwater System of Unit 3 /AEFWS-3/ in Construction

At KOZLODUY NPP a new Local Seismological Monitoring Network (LSMN) system has been installed. The systems includes tree broad band seismometers with remote central registration station in Sofia. This LSMN will be common for both KOZLODUY NPP and BELENE sites with six Seismic Stations ,Radio Telemetry System for transmission seismological information to Geophysical Institute / Bulgarian Academy of Sciences.

So far, there are many records from all three seismic stations:

- Surface Seismic Station Borovan (in a cave).
- Borehole Seismic Station Vulchedrum.
- Surface Seismic Station Orjahovo.

The records are from events out of 30-th kilometers region around KOZLODUY NPP and analysis maked in GI/BAS has demonstrated good LSMN's performance.

At the third stage of the seismic re-assessment , the WANO programme continiueus with tasks 3.1 ; 3.2 and 3.6 of TOR. WESE, Emresarios Agropados , along with Energoprojest - Sofia, performed re- evaluation of Primary Circuit and auxiliary lines and equipment of units 1&2. Results projects for upgrading and supply of new supports were prepared in accordance with . These projects were completed during outage period for units 1 and 2 this year.

WESE, EA and EGP prepared a seismic upgrading project on the basis of results of Structure capacity evaluation of Main Building (units 1 and 2) which was carried out during Stage 2. It consists of detailed design of Turbine Building construction (between rows A÷B) and electrical shelves (rows B÷B) as well as conceptual design for the rest part of Reactor Building (rows B-G-D).

Within the frameworks of Stage 3, the local programme of KNPP includes two basic activities.

The first one was evaluation of seismic capacity of the three reinforced-concrete venting stacks. The final results show no need of upgrading.

The second one was design of seismic upgrading for cable routes, systems interactions and items, which were designed as second priority components. The designing started immediately after the completion of first priority items upgrading. The designing were implemented at each unit during outages. A typical example for systems interactions is upgrading of masonry brick walls that are located near by safety systems components.

Now, stage 4 is in progress. The related tasks are being performed according to local NPP Programme, EBRD Programme and Item E of WANO Programme.

5.5. Stage 4

As it was mentioned above, the activities of WANO Programme related to units 1 and 2, are expended as NPP Programme activities for units 3 and 4. Following this principle, NPP assigned to RISK Engineering structure capacity re-evaluation of Main Building complex (row А-Б-В-Г-Д) of units 3 and 4 which is in progress.

Re-evaluation of primary circuit and auxiliary lines and equipment of units 3 and 4 are already completed under the same contract.

The corresponding detailed design for unit 4 is prepared. During the coming outage of unit 3 the required walkdowns for unit 3 will be carried out and consequently a design will be done.

This evaluation, as well as structure capacity re-evaluation of Main Building Complex of units 3 and 4.

The structure's modification resulted from the evaluation of capacity of all pipelines for dependent failures as well as reconstructions and modernization's performed under EBRD Programme. The Evaluation covers the scope of activities foreseen by tasks2.3 and 4.1 of TOR.

Another topic of NPP Programme is DSA for units 3 and 4. Evaluation of capacity of underground pipelines between BPS and NPP is also included and it is in progress.

At this stage, design for safety upgrading of Spent Fuel Storage Building is done on the basis of relevant re-evaluation performed of stage 2. As mentioned above, during the first stage EQE International and EQE Bulgaria have fulfilled a mutual design for seismic upgrading of the buildings of Diesel Generator 2 and of Pump station 2. These designs were prepared following TOR conservative criteria. So fare, it turned out that it is prefarable to prepare new designs for these two buildings instead of implementing the old ones.

A design for the seismic upgrading of the building was created by EQE-Bulgaria in 1993. It was based on the preliminary WANO prescriptions which valid before the adoption of the seismic design spectrum for NPP. The conservative seismic input estimation led to complicated upgrading concepts and heavy details. cross section of the upgraded building is given of fig**. External steel braces anchored in new foundations and cast-in-situ piles were to be used. All wall panels and a considerable number of concrete beams were be dismounted and/or replaced with lighter ones. The operation of the Pump station ans DG-2 halted.

In 1997 the seismic vulnerability of Pump Station-2 and DG-2 buildings at Kozloduy NPP site was investigated for a Review Level Earthquake with 2.0g peak ground acceleration, taking into account the influence of local seismic sourse. The more precise seismic input estimation made possible the creation of an upgrading design which could be implemented considerably easier.

Currently, a procedure on PHARE Programme is underway for implementation of the designs for seismic upgrading of spend Fuel Storage Building, DG-2 Building and Pump Station-2 Building.

In accordance with the principle for providing seismic safety improvements throughout the reconstruction and modernization of the systems, the following activities are in progress:

New auxiliary emergency feedwater system for Steam Generators is under construction. It includes separate building (Fig. 3) and dubs the existing system which is located in Turbine Hall. The new system is fed with water from a new Fireprotection Pump Station-2 This pump station is under construction within the scope of the same programme and is located on the channel between Pump station - 1 and 2.

Another very important topic is the total reconstruction of elevation 14.7m (row B-B). It covers complete replacement of Main Steam Pipelines, installation of Quick Closing Isolating Valves, replacement of anchorages and supports, including local reinforcement of the wall of row B.

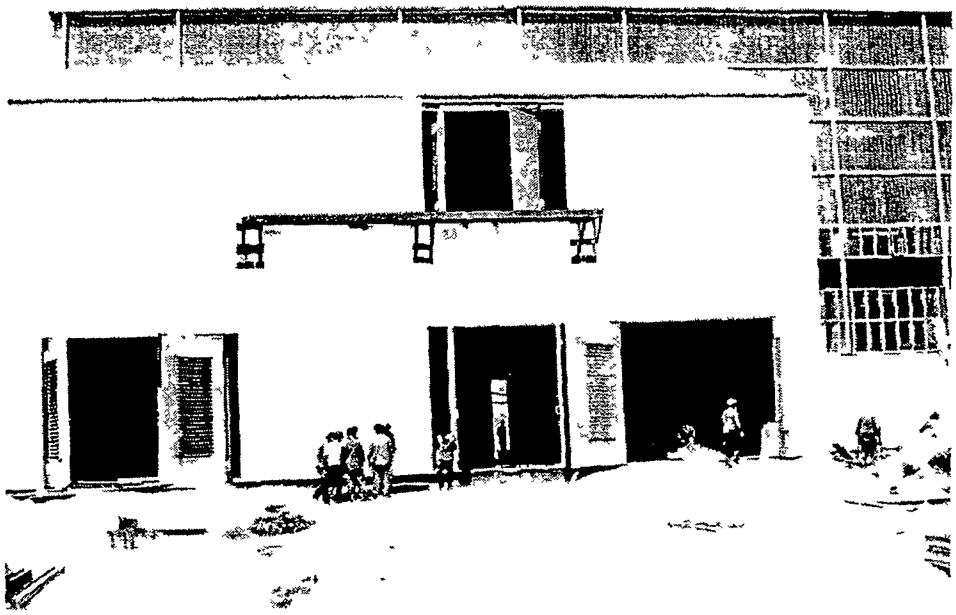


Figure 3 Building of Auxiliary Emergency Feedwater System of Unit 4 /AEFWS-4/ in Construction

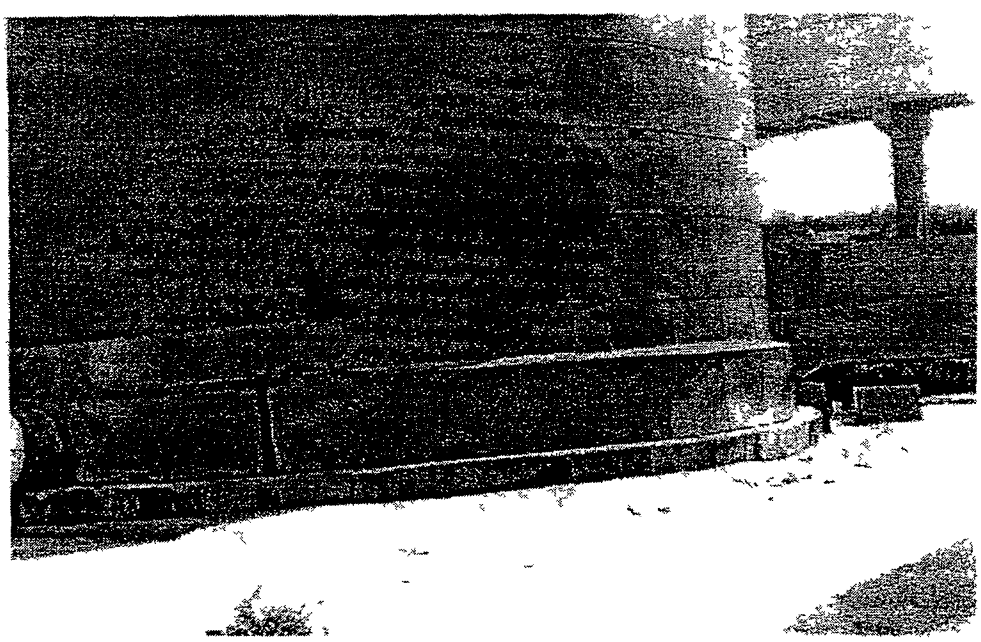


Figure 4. S.U. of Demineralized Water Tank/DWT/ at elevation +0.00m, Unit 3 Anchorage Ring

Item E of WANO Programme involves Qualification of safety related equipment under accident conditions and it is performed by Empresarios Agropados. A part of it is Seismic Qualification. So far, the list of equipment which require seismic qualification tests is ready. For some equipment such qualification tests are already done by Bulgarian organizations and IZIIS - Skopie.

6. IMPLEMENTATION PHASE

Table 1. gives a picture of allocation of selected representative provisions used in implementation phase of seismic upgrading designs and same of the upgraded units.

Table 2. represents allocations of the basic provisions per year, for Kozloduy NPP units 1 to 4. Approximate expert estimation of expenses is given in the last column of the table.

7. FUTURE ACTIVITIES

In addition to the presentation of the evolution of seismic upgrading at Kozloduy NPP site, Attachment a also indicates the status of the two basic type of activities:

- re-evaluation & designed
- implementation

Of course, all the activities identified as "in progress" status are to be finished. The expected terms for completion are as follows:

- Bank Pump Station Building Complex upgrading - 1998.
- Re-assessment of the previous project for local EQ - 1998.
  The related activities are already completed and consequent measures are implemented for units 3 and 4, the final completion of the re-assessment is foreseen for outage'98.
- Implementation of the S.U. design for T.B. and El. shelves (A-Б-B) on unit 1 and unit 2 is not planed up to now. It is foreseen to be performed after completion of Main Building Complex.
- Construction of new auxiliary emergency feedwater system for SGs - end of 97.
- Reconstruction of elevation 14.70m; is completed for three of the units; for the last one will be finished during current outage.
- Implementation of S.U. design for Spent Fuel Storage Building, for DG Building-2 and for Pump station Building - 2 will be performed according to the terms of the contract being prepared.

As noted in Attachment A (A-5), within the frameworks of WANO programme item E, a list of equipment that require seismic qualification tests is prepared. Extension of the contract for item E is underway and the corresponding tests will be performed accordingly.

Evaluation of capacity of pipelines between BPS and NPP - January'98.

Competition of structure capacity re-evaluation of Main Building Complex (units 3 & 4); re-evaluation of primary circuit and auxiliary lines and equipment (units 3 & 4) and consequent S.U. design 98.

TABLE 1: ALLOCATION OF THE BASIC MATERIALS USED IN THE SEISMIC UPGRADING (S.U.) OF NPP KOZLODUY UNITS 1-4 TILL AUGUST 1997.

Unit 1:

Outages [year]	Steel [kg]	Number of anchors [pcs]	Upgraded units		
			Mechanical equipment [pcs]	El. and I&C equipment [pcs]	Masonry walls [m^2]
1993	6 287	274	42	58	-
1995	11 804	1 418	12	111	432
1996	11 000	1 555	15	22	705
1997	16 500	1 155	-	-	924
Total:	45 591	4 402	69	191	2 061

Unit 2:

Outages [year]	Steel [kg]	Number of anchors [pcs]	Upgraded units		
			Mechanical equipment [pcs]	El. and I&C equipment [pcs]	Masonry walls [m^2]
1991/1992	3 324	108	22	23	-
1994	8 333	635	12	106	-
1995	27 403	1 685	7	1	1 542
1997	10 854	1 456	36	13	525
Total:	49 914	3 884	77	143	2 067

Unit 3:

Outages [year]	Steel [kg]	Number of anchors [pcs]	Upgraded units		
			Mechanical equipment [pcs]	El. and I&C equipment [pcs]	Masonry walls [m^2]
1993	28 381	3 161	31	294	765
1994	22 627	2 483	4	19	1 183
1996	6 458	927	13	65	540
Total	57 466	6 571	48	378	2 488

Unit 4:

Outages [year]	Steel [kg]	Number of anchors [pcs]	Upgraded units		
			Mechanical equipment [pcs]	El and I&C equipment [pcs]	Masonry walls [m^2]
1992/1993	49 558	4 713	43	129	1 560
1995	3 086	328	14	17	105
1997	3 800	470	-	-	240
Total	56 444	5 511	57	146	1 905

TABLE 1 (cont.)

Bank Pump Station					
Steel [kg]	Number of anchors [pcs]	Concrete [m^3]	Upgraded units		
			Mechanical equipment [pcs]	El. and I&C equipment [pcs]	Masonry walls [m^2]
22 337	430	65	6	3	48

Total					
Steel [kg]	Number of anchors [pcs]	Concrete [m^3]	Upgraded units		
			Mechanical equipment [pcs]	El. and I&C equipment [pcs]	Masonry walls [m^2]
231 752	20 798	65	257	861	8 569

Provisions for items that contribute to improvement of seismic safety.				
No	Object	Steel [kg]	Number of anchors [pcs]	Concrete [m^3]
1	Replacement of Accumulator Batteries /AB/.	19 047	1 079	-
2	Building of Auxiliary Emergency Feedwater System of Unit 3 /AEFWS-3/	135 000	-	605
3	Building of Auxiliary Emergency Feedwater System of Unit 4 /AEFWS-4/	267 000	-	775
4	Fireprotection Pump Station - 2 /FPPS-2/	186 000	-	1 643
5	Reconstruction at elevation +14,70m	26 000	2 340	-
Total:		633 047	3 419	3 023

TABLE 2.

BASIC PROVISIONS ALLOCATION PER YEAR				
Year	Steel [kg]	Number of anchors [pcs]	Concrete [m^3]	Cost [thousand $]
1992	31 246	2 601	-	130
1993	60 771	5 655	-	260
1994	35 427	3 118	-	150
1995	46 760	3 646	-	200
1996	171 458	3 892	30	650
1997	519 137	5 305	3 058	2100
Total	864 799	24 217	3 088	3 490

Some major activities have not been started yet. The ongoing reconstruction and modernization of technological process systems resulted in changes in some safety functions, some functions have been added, another - excluded, some components have been dubled or supplemented and so on. On that basis NPP considers necessary to perform studies for re-assessment and justification in order to structures and components.

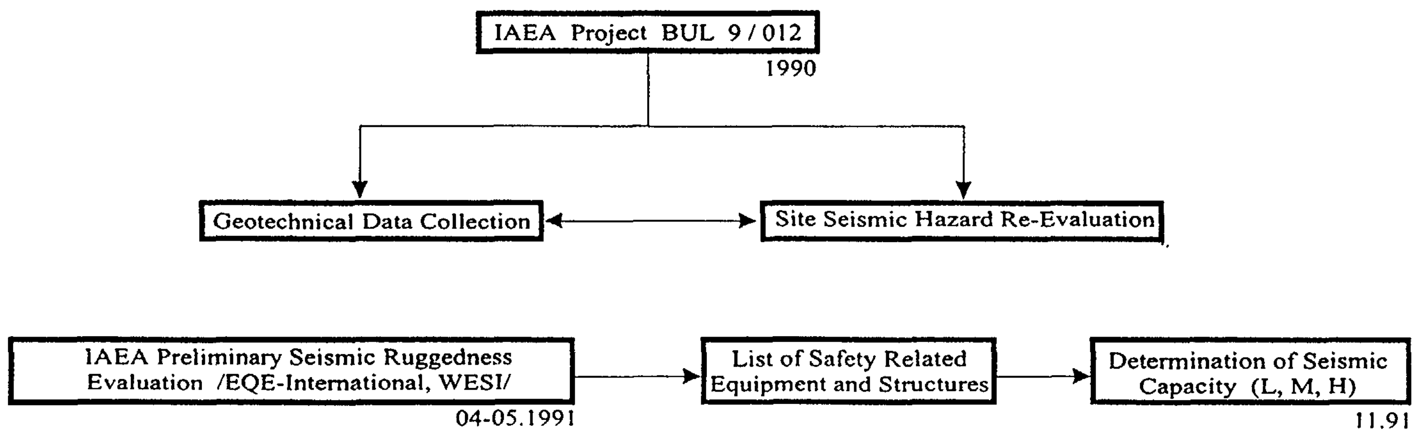
For example, part of the safety functions were transferred from Turbine Hall this requiring re-assessment of the Turbine Building seismic stability for RLE lower that the accepted one of 0.2g maximum free field acceleration.

On the basis of these new studies the SSEList will be revised and updated. The remaining designs for second priority items and qualification tests (item E) will be performed in compliance with the updated SSEL.

A complete analyses and detailed S.U. designs for Reactor Building Complex (rows В-Г-Д) on units 1 and 2; for service water lines underground lines inlet in Turbine Building and for nozzles and supports of the Primary circuit piping to the Pressurizer, high pressure injection system and spray system, as well as main steam header and feedeater piping are to be performed.

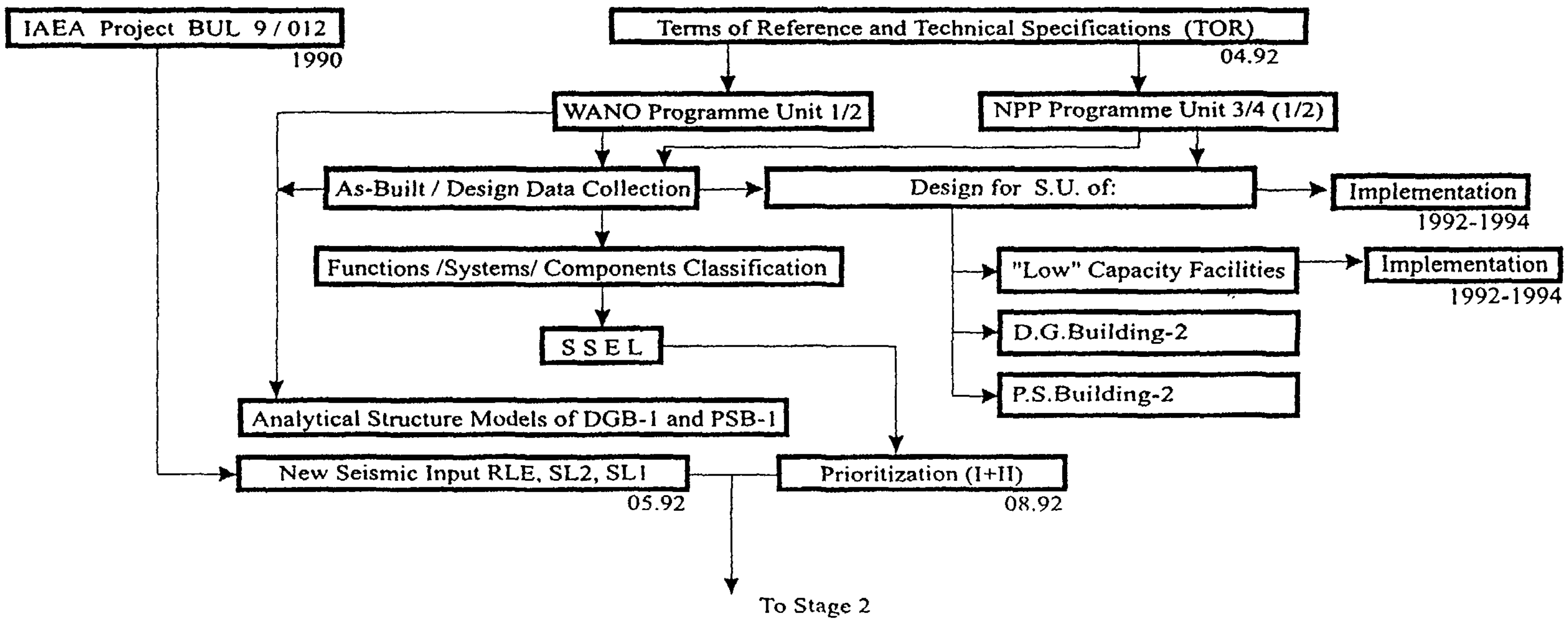
SEISMIC RE-EVALUATION FLOWCHART

Stage 0



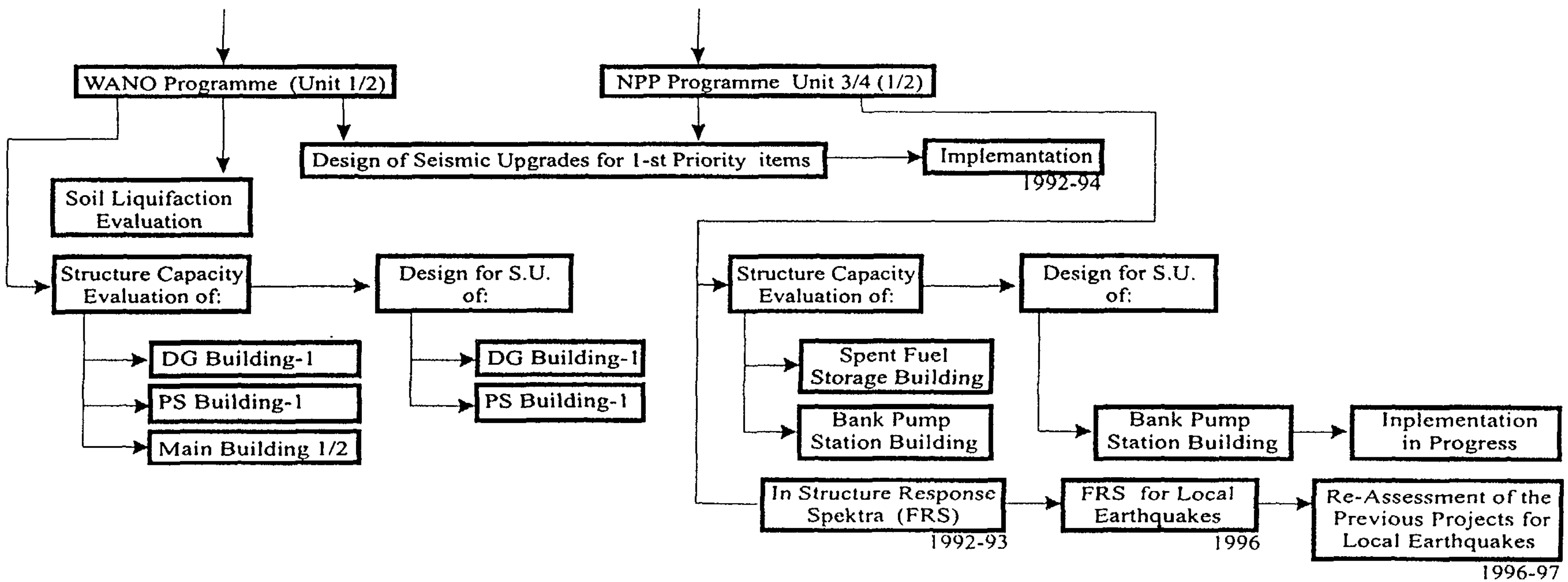
SEISMIC RE-EVALUATION FLOWCHART

Stage 1



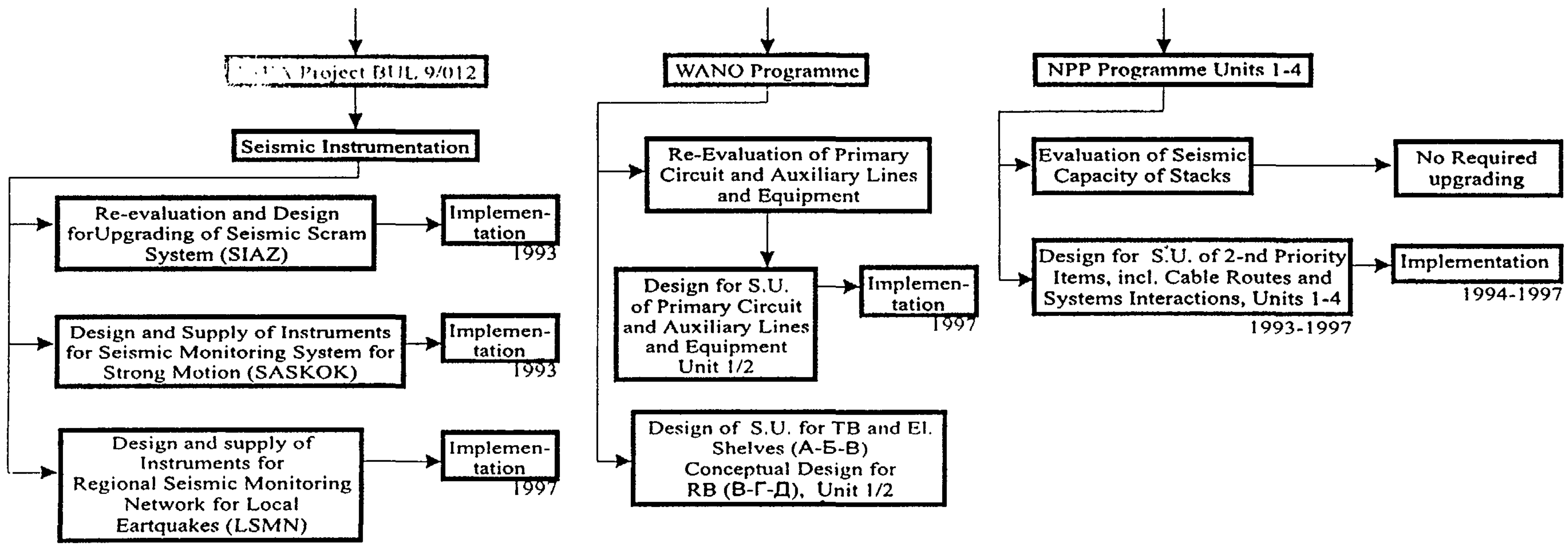
SEISMIC RE-EVALUATION FLOWCHART

Stage 2



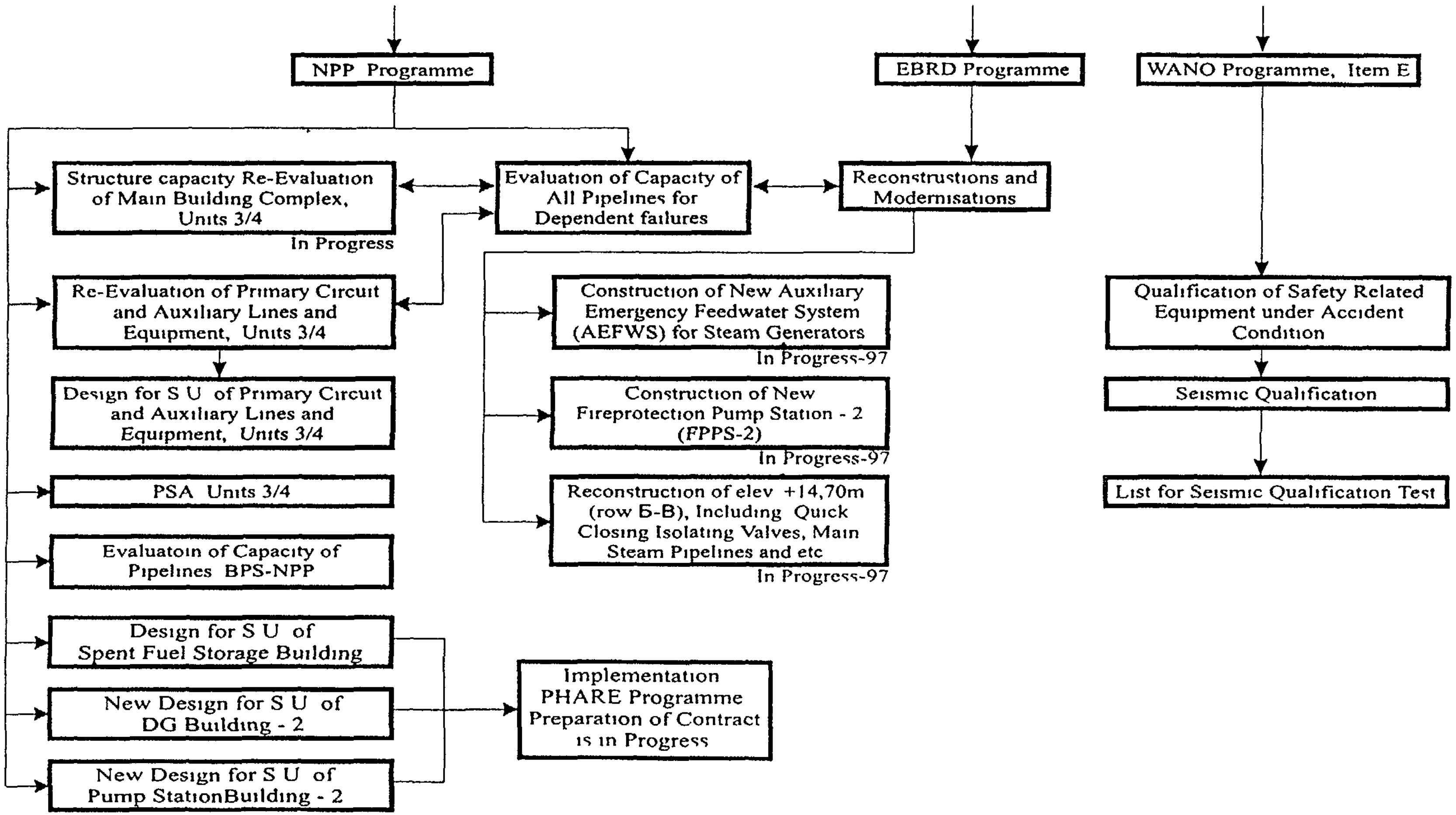
SEISMIC RE-EVALUATION FLOWCHART

Stage 3



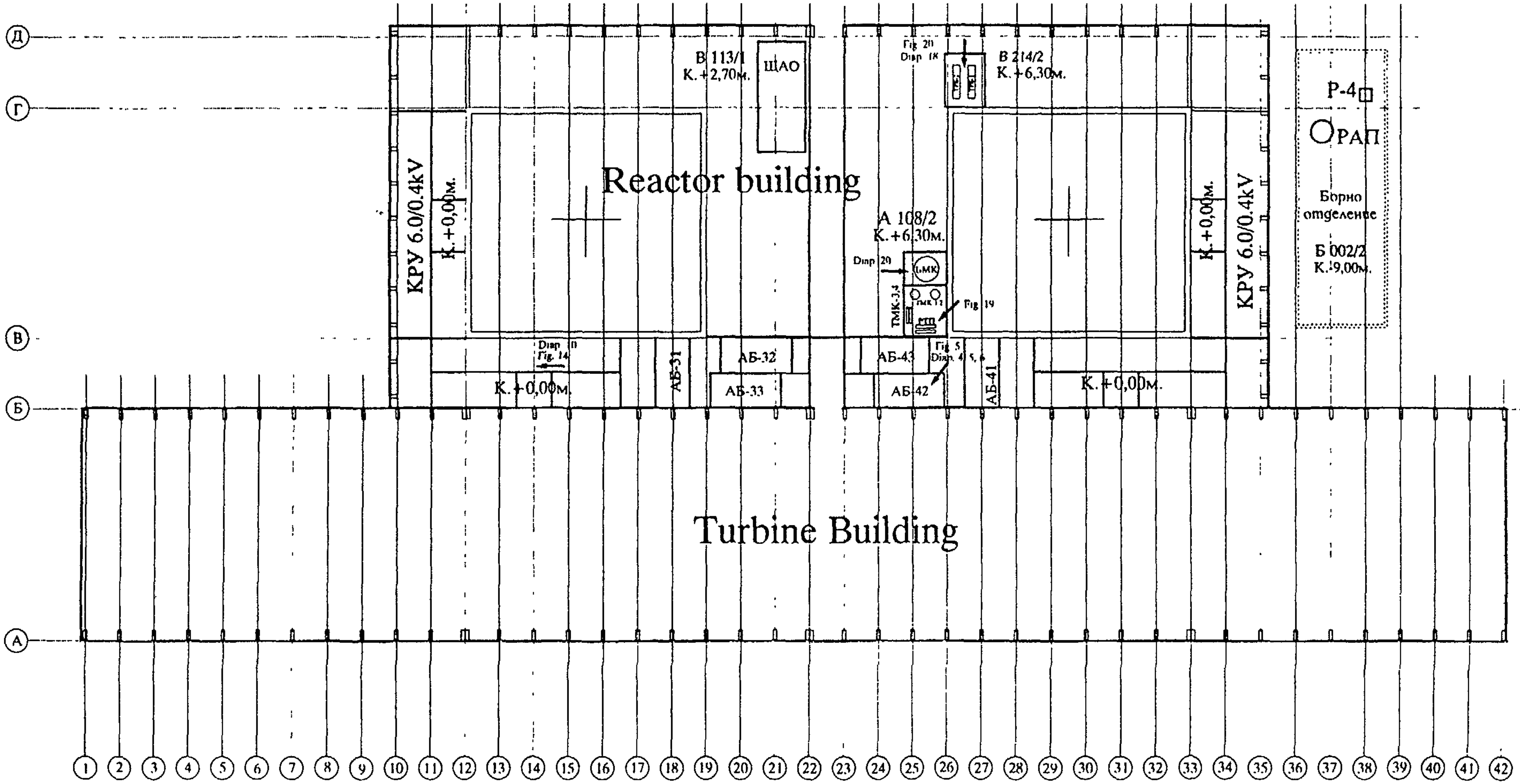
SEISMIC RE-EVALUATION FLOWCHART

Stage 4



MAIN BUILDING

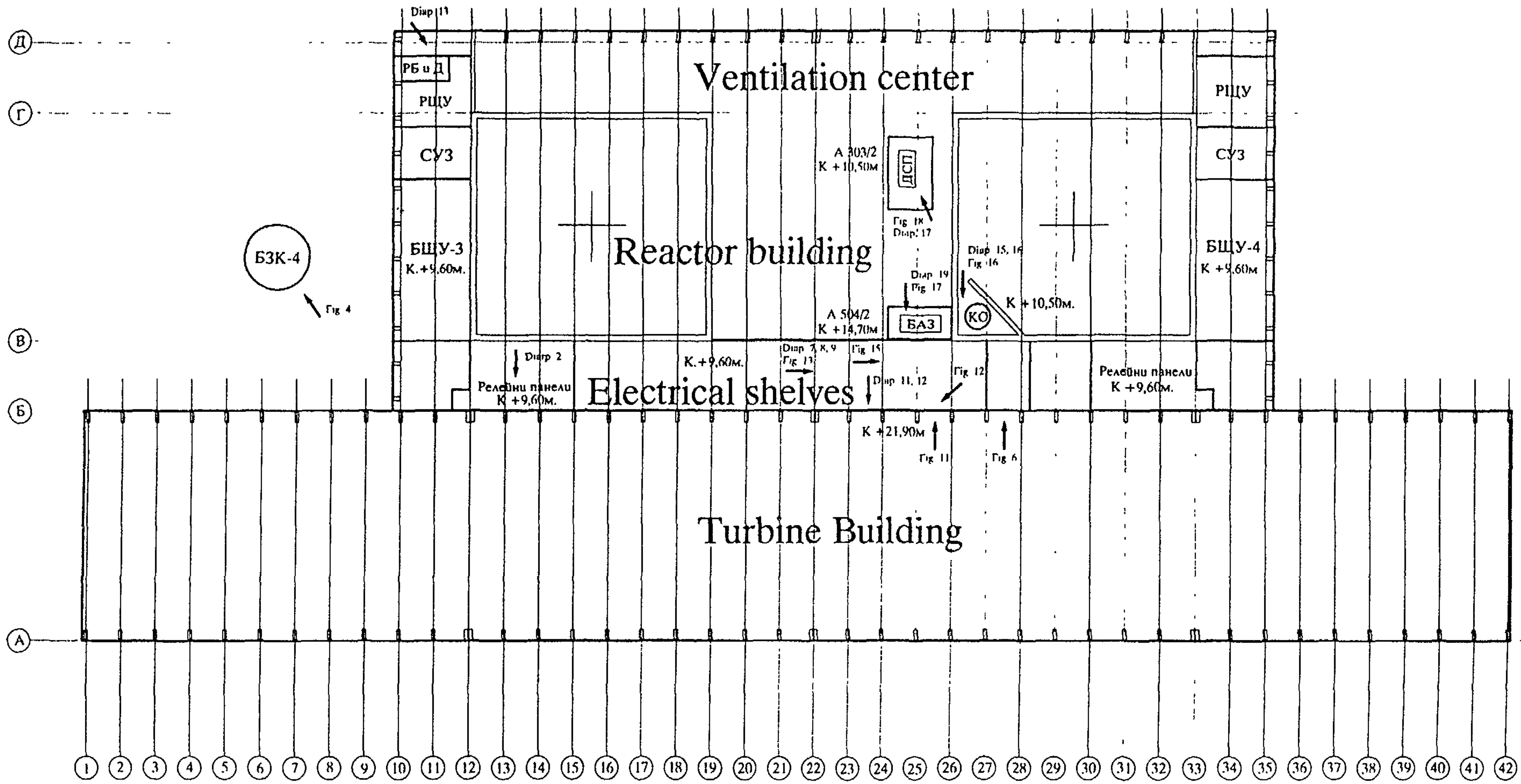
Elevations -9,00м., +0,00м., +2,70м. and +6,30м..



C-1

MAIN BUILDING

Elevations +9,60m., +10,50m., 14,70m. and 21,90m.



C-2

STATUS OF THE SEISMIC UPGRADING PROGRAMME AT MOCHOVCE NPP

T. ZAJÍČEK, R. DOLNÍK, M. ŠTEVKO

Abstract

The paper provides an overview of the seismic characterisation of the Mochovce site in Slovakia. Particularly, emphasis is given to differences between the original siting and design procedures and the re-evaluation approach, much more based on the data from the micro-earthquake monitoring system installed at the site.

Details are also provided for the seismic monitoring of the buildings, as confirmation of the design assumptions.

Basic Information

The Mochovce NPP (EMO) is owned by the Slovak Power Plants sc. (SE a.s.). The power generation in Slovakia is shared among the different production areas as in the following: 50% nuclear, thermal 30%, hydro 8% and 12 % import (for year 1996). Estimated import for year 1997 is about 20% (Fig.1).

NPP Mochovce is located in the Southwest region of the Slovak Republic. The site is about 20 km from the town Levice, 35 km from the district town Nitra and 135 km from Bratislava, the capital of Slovakia .

Seismic input data

As result of all geological and seismological investigation, Mochovce NPP was designed according to seismic criteria. Construction of the NPP has been carried out, as the first in formal CSSR , in accordance with the CSSR standard CSN 73 0036 - seismic loads for buildings and Soviet standard VSN 15-78-construction of the seismic resistant of NPP.

For the seismic design of seismic resistant buildings the following values have been assumed:

Maximal Design Earthquake (MDE) = 5° of MSK - 64

-this value as OBE (SL1-IAEA 50-SG-S1 code)

-with the horizontal ZPGA for MDE = 0,025g

Maximal Calculation Earthquake (MCE) = 6° MSK-64

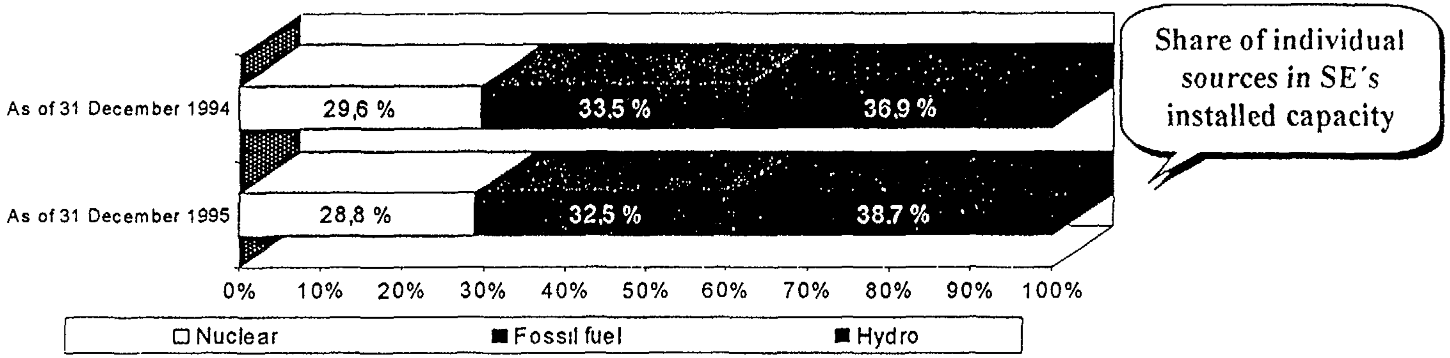
- this value as SSE (SL2-IAEA)

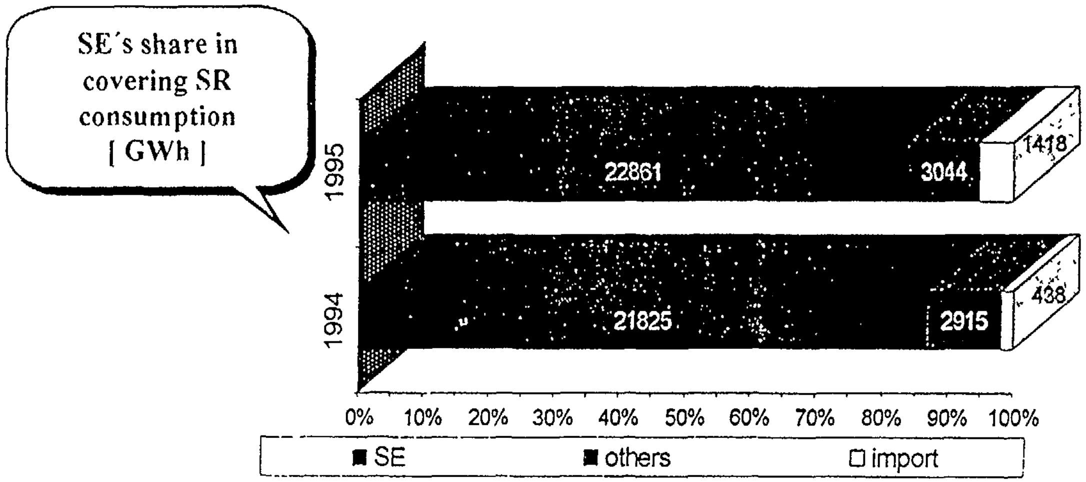
- with the ZPGA for MCE = 0,06g

From the point of view of seismic classification, buildings and equipment's for safe shutdown, aftercooling and residual heat removal for 72 hours were selected. They have been grouped into seismic category 1. All other equipment's and buildings are 2-category, that means non- seismic resistant.

From the point of view of core melting probability, we followed the international practice and basic design criteria for seismic events. As acceleration diagram (Time history) the record from „Nis“ (Serbia) was accepted, and from earthquake 4 march 1977 in Vrancea area (Romania). This accelerogram was selected on the basis of administrative considerations. As MCE=6° of MSK-64 could be used only with the condition that there is rock under the NPP, the former site at about 3 km on the east direction had to be replaced with a different site, involving extensive mining of about 6 million m^3 of rock.

SE´s share in installed capacity, generation and coverage of the Slovak Republic consumption





Seismic characteristic of this region is a very low activity. From the historical point of view, we collected data from time period 1022 up to 1994. For the Mochovce site, the Komarno area is the most dangerous, with maximal historical earthquake is 8,5°;of MSK - 64 scale: it is about 55 km from EMO site, south - west direction (Komarno area) (Fig.2). The attenuation law decreases the intensity from 6 MSK-64 to about 2,5. Another seismic area is „Middle Slovakian area" (Kremnica, Banska Bystrica (7.5) and Dobra Voda (9) (Fig.3). But from this direction the attenuation is also about 2 of MSK-64 and 3,5 of MSK-64.

EMO site is located on the rock soil with volcanic layer (andesit). Characteristic shear wave velocity is between 2,000-3,000 m/s.

Seismic re-evaluation

The original design of Mochovce NPP did not follow the IAEA recommendation about the minimal seismic hazard. The requirements from today authorities are higher for NPP safety to external hazard and therefore some upgrading is required to a level generally accepted by the international community. Therefore, the seismic input SSE has been upgraded to ZPGA = 0.1 g for the estimation of the seismic resistance of buildings and equipment. In accordance with IAEA documents, the RLE has been defined as:

PGA_{RLE} is 0.1g (in horizontal direction),
PGA_{RLE} is 0.067g (in vertical direction),
GRS_{RLE} is NUREG-0098 ground spectrum of absolute acceleration (median +1sigma) for rock site, resp. for shape site of next buildings.

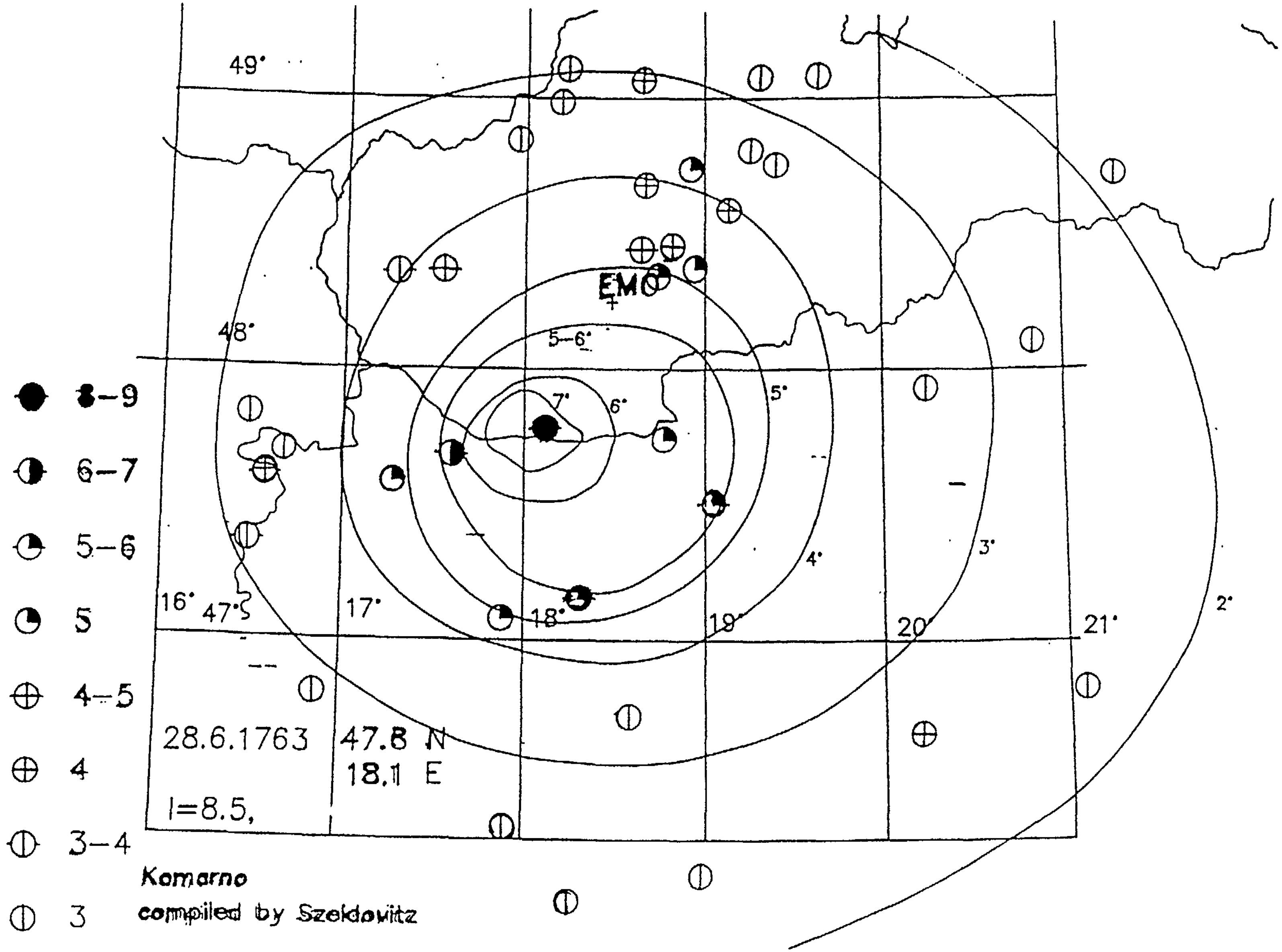
This PGA is corresponding with an intensity 7° in MSK - 64 scale. For re-evaluation of reactor hall the Newmark's ground rock spectrum is applicable because the velocity of shear waves is higher than 1100 m/s limit (Fig.4).

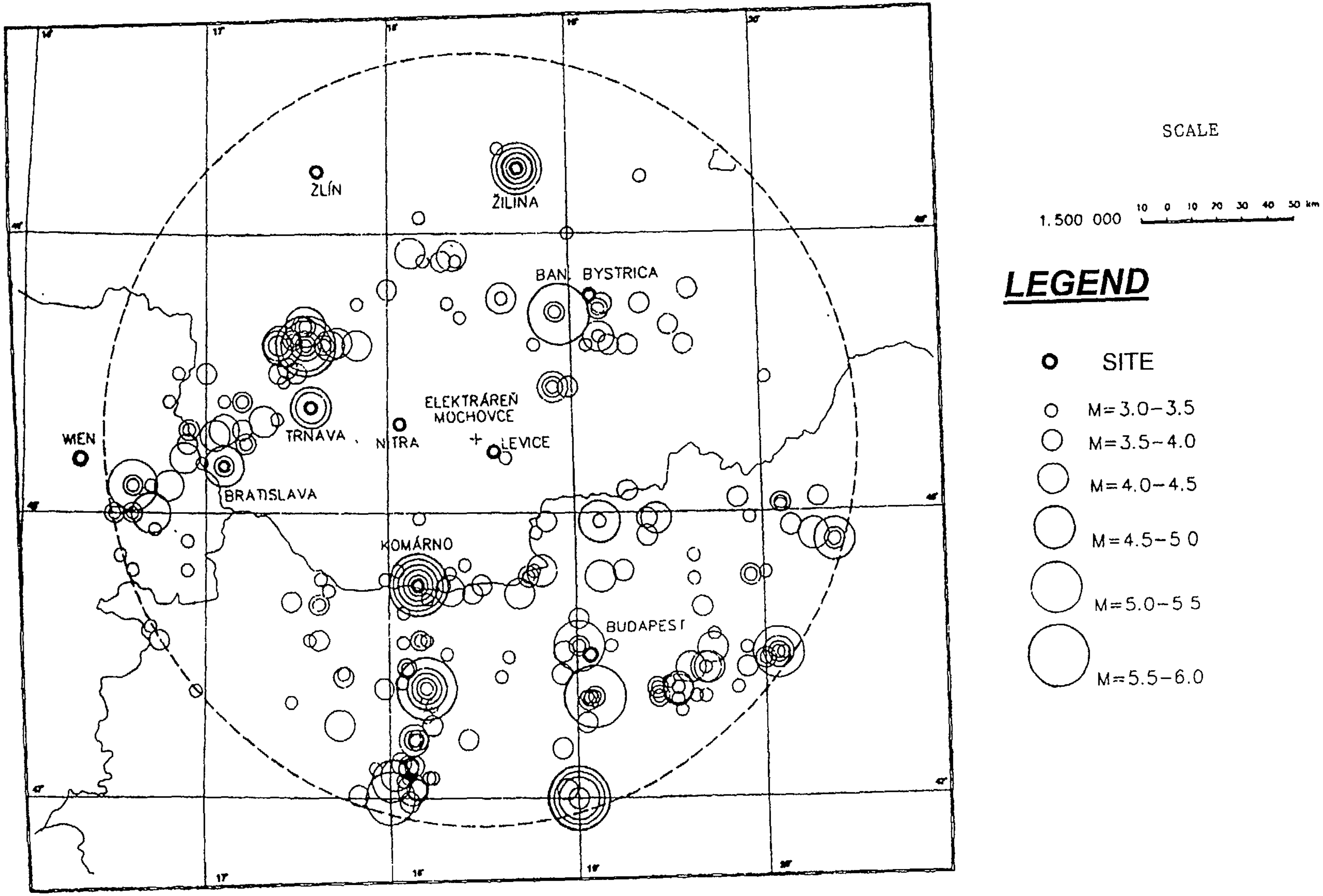
We used a less conservative method than used in formal design process. This fact was confirmed in the revised edition of POSAR - part Seismic Hazard.

The methodology „Seismic re-evaluation guide of Mochovce NPP structures and equipment Units 1 and 2" was prepared by Skoda Praha and Stevenson & Associates. The guideline of NPP Mochovce -Unit 1 and 2 - seismic re-evaluation is based on IAEA document „Technical guidelines for the re-evaluation programme of Mochovce NPP" from august 1995. This document recommended for seismic re-evaluation of NPP Mochovce a SMA methodology and a special GIP procedure for the qualification of active safety related equipment. The SMA methodology defines the boundary seismic capacity of NPP as the whole. This methods studies the question whether the capacity of the already built plant exceeds the target earthquake input which was selected for review. Following the guidelines, we recalculated the floor response spectra to be used for the qualification of those structures, systems and components needed to bring the plant to a safe shutdown condition after an earthquake, and maintain it in a safe shutdown condition for certain defined period.

The main criteria for PWR reactor units are integrity of primary system. The plant must be capable to be brought and maintained in a cold safe shutdown condition during the first 72 hours following the occurrence of the RLE and seismic interactions prediction. The first step was based on the original design and 8 000 mechanical components (pumps, tanks, pipes) and about 15 000 electrical and I&C components (cables, cable traces, cabinets, etc.) were qualified.

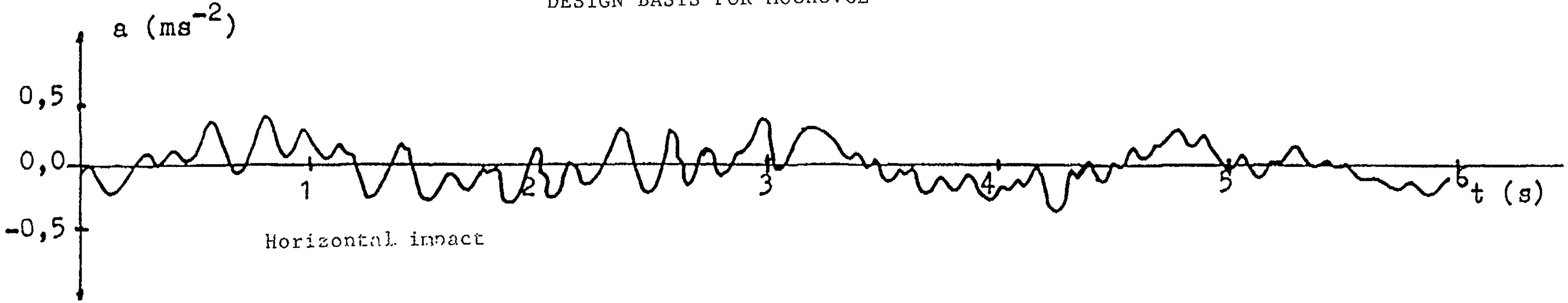
The rest of structure and components, which are out of the safe shutdown equipment list, are not seismically resistant. At the present time equipment list is divided into two groups:

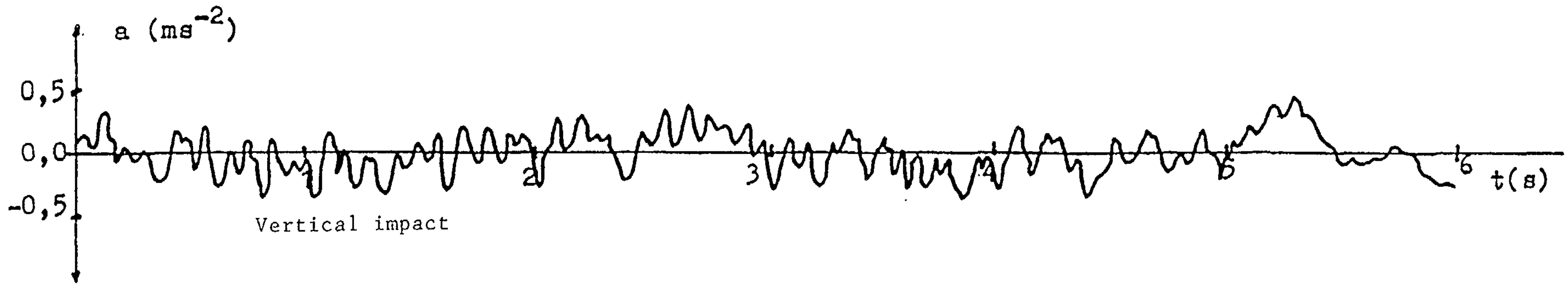

49°
48°
16°
47°
17°
18°
19°
20°
21°
EMO
5–6°
7°
6°
5°
4°
3°
2°
8–9
6–7
5–6
5
4–5
4
3–4
3
28.6.1763
47.8 N
18.1 E
I=8.5,
Komarno
compiled by Szekdovitz



EARTHQUAKES MAPS INTO A 150KM RADIUS FROM NPP

DESIGN BASIS FOR MOCHOVCE





- Priority H (high) and L (low)
(H priority means - equipment's are needed to bee reinforced up to start-up)
(L priority means - equipment's can bee reinforced later or in the first outage)
At the and we collected the list of equipment's with approval protocols of seismic resistance based SQUG-GIP (HCLPF):

-recalculation
-reinforcement
-replacing

as appendix of POSAR.

Seismic instrumentation

- **Internal seismic instrumentation**

In the basic design for NPP Mochovce, the plant was designed with a seismic shutdown system called SIAZ of Soviet production (System of Industrial Antisesmic Protection). His function was initiating a signal (as automatic reactor scram) for:

- Reactor Protection System - to initiate shutdown
- Safety system - to switch on the equipment's for aftercooling though the initiating schedule
- Alarm to Main Control Room
- Switching off crane and refuelling machine
- Recording of the absolute acceleration versus time

SIAZ had nine triaxial accelerometers in three independent systems with independent electric power supply and two sets of them (totally 18 sensors). The output from seismic monitoring system is an active input for reactor protection system and for many other Safety systems. In the basic design the reference value of acceleration was assumed to be that measured by triaxial sensors located at the NPP foundation base. The triggering level is 0.01g and the initiating level is 0.05g.

- **External seismic monitoring network**

Monitoring of seismic activity at the site of NPP and near region is a standard activity in the world. It gives useful information of seismic sources and micro-earthquake capability.

The minimum monitoring period required to obtain meaningful data for seismo-tectonic interpretations is several years.

The system comprises of a network of 7 seismometer stations located within a radius of about 25 km from the Nuclear Power Plant at Mochovce. One of them is located inside the area of NPP about 600 m from Reactor Hall. Seismic instrumentation has been purchased from Lennartz Electronic (Germany) and GEMI (Czech Republic). The mounting of the seismograph stations were preceded by an extensive field survey and careful investigation of noise background.
The Seismic Monitoring Network System is currently operated and its data transferred, processed and analysed by Progseis Trnava and the Geophysical institute of Slovak academy of sciences, on a weekly basis.

XA0100501

STATUS OF SEISMIC RE-EVALUATION AND UPGRADING OF KANUPP

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Abstract

The Seismic upgradation activities at Karachi Nuclear Power Plant (KANUPP) begin in 1992 after a preliminary plan was chalked out with IAEA to conduct the seismic walkdown of the plant and initiate site studies to reconfirm geotechnical parameters & determine new seismic input. Consequently, the seismic walkdown was arranged in May 1993. The site geotechnical parameters were re-evaluated by performing the geotechnical investigations and cross hole seismic survey. This was followed by collection of data for seismic studies, geological surveys, surface fault studies and development of a seismotectonic model for determining the new seismic parameters as per IAEA safety guide no. 50-SG-SI. In parallel with the seismic studies, the short term fixes work was also initiated with iRLE value but gained momentum during the last six months. Dynamic analyses of some structures / equipments identified by the IAEA mission with a fabricated spectra and cross checked with simplified techniques have been completed and retrofitting / anchoring details provided to the implementation division while analyses / fixes design of other structures are in progress.

1.0 INTRODUCTION

KANUPP founded on rock is located in the south of Pakistan on the Arabian sea coast, at long. 66° 47' 22'' & lat. N 24 49 10", near Karachi. It is a 125 MWe (net), heavy water moderated & cooled, natural uranium fuelled horizontal pressure tube reactor with once through on power bidirectional fuelling plant. The construction of the plant begin in Sept., 1966 and was made operational in 1972. Fig. 1 shows the layout of the plant.

The safety related equipment is housed in the containment, service, Turbine and DG buildings. All these buildings except the containment are concrete frame structures with reinforced block masonary infill walls. The containment building is a prestressed structure with Freyssinet prestressing system designed against an internal pressure of 27 psi. All these buildings were designed in accordance with Canadian codes for a 'g' value of 0.1 and wind speed of 100 mph.

Factors considered in the estimation of 'g' value for NPPs in thé sixties were not adequate and as such a value of 0.1g adopted for KANUPP was based on the seismic data only. With the technological development during the last three decades and the know how available today, the seismic risk estimated for Karachi area is higher due to the tectonic setting in which KANUPP is placed. Consequently it was decided in 1992 to reavaluate the NPP under the technical guidance of IAEA.

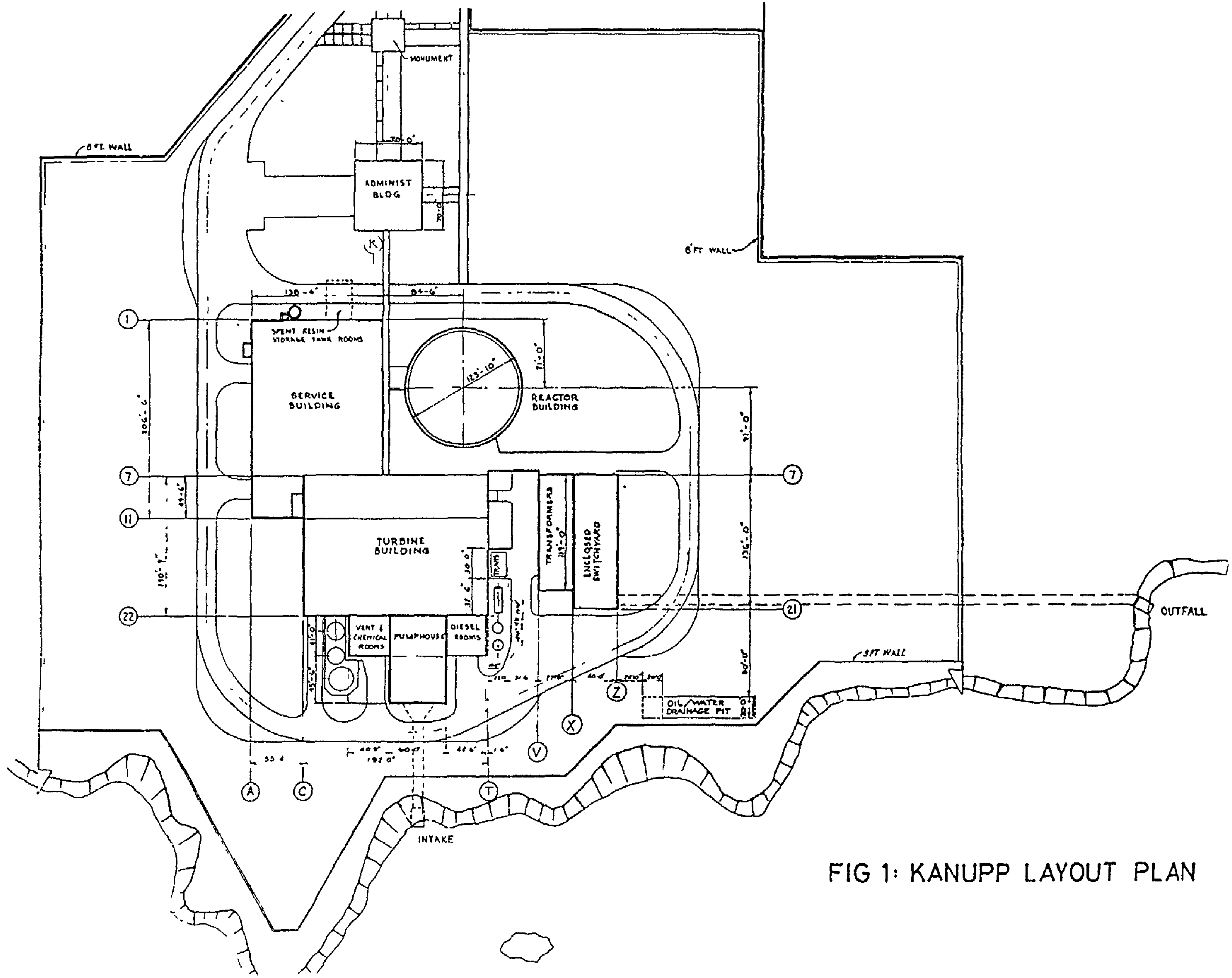

FIG 1: KANUPP LAYOUT PLAN

2.0 SEISMIC WALKDOWN

The seismic walkdown of the plant was conducted in May 1993 by an IAEA mission in a similar pattern performed for the NPPs in the eastern European countries. Prior to the seismic walkdown, all background details relating to layout, design criteria, description of plant structures & equipments were provided to the experts.

The walkdown was conducted after functions, systems, components and structures required during and after an earthquake were identified.

As a result of the seismic walkdown, recommendations were provided to improve the seismic safety in a systematic way based on a phased approach of short term and long term actions. The walkdown brought out a very clear picture of KANUPP as the seismic capacity of every important equipment / structure was precisely assessed and described. The general conclusions were very encouraging as most of the plant was assessed to withstand much stronger goundmotion than the original design while some critical areas were pointed out which could be even vulnerable to an earthquake with PGA of 0.1g. This conclusion is based on the results of seismic requalification programmes in western countries for plants of the same vintage as KANUPP.

For the execution of the tasks corresponding to the phase 1-short term actions, it was recommended to anchor / fix equipment with no / in adequate anchors with a conservative value of Review level earthquake (RLE). The RLE should be selected on conservative basis, in accordance with the current information and knowledge for the verification / design of anchor / fixes as a high value of RLE has no significant influence on the cost of this work. The list of equipment for phase 1 included emergency batteries, cranes, refuelling machine, control room panels, equipment in distribution room, emergency diesels, ventilation fans, large tanks, heat exchangers, steam generator pads, inadequately supported pipe spans & masonary walls. A further walkdown was also recommended to identify interaction effects & unanchored safety equipment.

For the long term, it was recommended to perform detailed dynamic analysis of the structure foundation system and evaluate seismic capacity demand of all structures, allowing acceptable levels of non linear response to the RLE if the spectral intensities are equal / larger than twice the original design.

In case of RLE less than twice the original value, simplified methods were recommended.

Although, the seismic studies have been completed and a value of 0.2g has been obtained, a decision on the long term tasks shall be taken after the seismotectonic studies are reviewed by IAEA.

3.0 SEISMOTECTONIC STUDIES

The seismotectonic studies to reassess the seismic hazard were completed in April 1997 by following the guidelines of IAEA safety guide no. 50-SG-S1 (1991):

3.1 Seismotectonic Setting

The seismic behaviour of various structures present in the environs of KANUPP site basically owe to the north ward push of the Indian plate due to which its northern extremities have subducted below the Eurasian plate after consumption of the Tethys sea (Fig. 2). The area is located very close to the triple junction of Eurasian, Indian and Arabian plates, lying towards west of the site in the Arabian sea. The northward movement of the Indian plate, although did not register direct effects of the subduction in Sind area yet it did register the drag effects produced by the transform movement of the plates along Ornach - Nal Fault which is the southward continuity of the Chaman fault towards north and Murray ridge towards south. This movement imparted

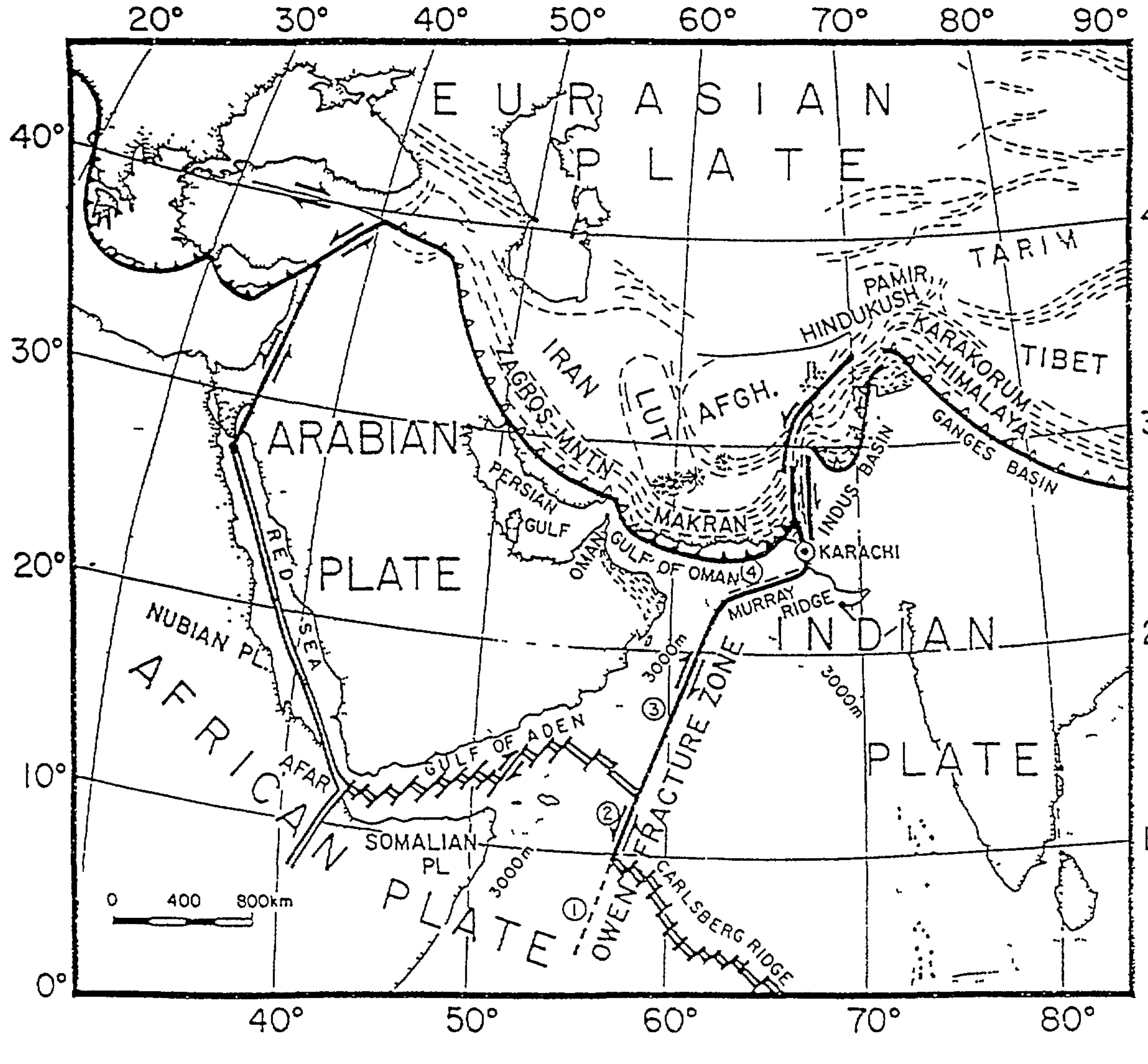

FIG.2: REGIONAL PLATE TECTONICS

a typical structural scenario to the area around KANUPP with roughly N-S oriented wrench faults such as Pab, Surjan, Ornach Nal and Kirthar alongwith development of various thrust faults produced during the isostatic balance to cope with the crustal shortening.

3.2 Construction of Regional Seismotectonic Model

In the first step, a seismotectonic model of the region, covering an area with a radius of 150 kms around the site was developed. The area was thoroughly studied and the investigations were performed in four scales. Regional, near regional, site vicinity and site area. Maps were drawn on different scales and greater is the contained information as the site is approached. Maps were drawn on regional (r = 150 km), near regional (r = 25 km) and near site (mapping, neotectonic studies) area. By super imposing the data on geology, seismology, remote sensing and geophysical studies, this region was subdivided and boundaries for different seismogenic structures & seismotectonic provinces were defined. Fig. 3 shows the seismotectonic model of the Karachi region.

3.3 Determination of the Maximum Potential Capability

On the basis of available data each seismogenic structure and seismotectonic province were assigned the maximum earthquake generating capability. For the floating earthquake, it was found appropriate to add a conventional one to the maximum recorded / everfelt earthquake while physical characterisation of the important structures were taken into account in assessing the maximum earthquake generating capability. Similary, half the length of known faults were taken in determining the maximum potential capability.

The maximum postulated earthquakes in each structure / province were moved in their respective structures / provinces to the point closest to the site for determining the epicentral distances.

3.4 Assessment of Peak Ground Acceleration (PGA)

After determining the maximum postulated earthquakes and epicentral distances for each structure / province in the developed seismotectonic model of the region, Peak ground accelerations were estimated at the site from the different earthquake sources.

The accelerations at site were estimated by using the attenuation relations developed for similar conditions in the world as the site specific relations are not available for Pakistani sites due to insufficient data on strongmotion. For each estimated value of 'g' a standard deviation was added to the mean value to obtain 84.1 percentile values.

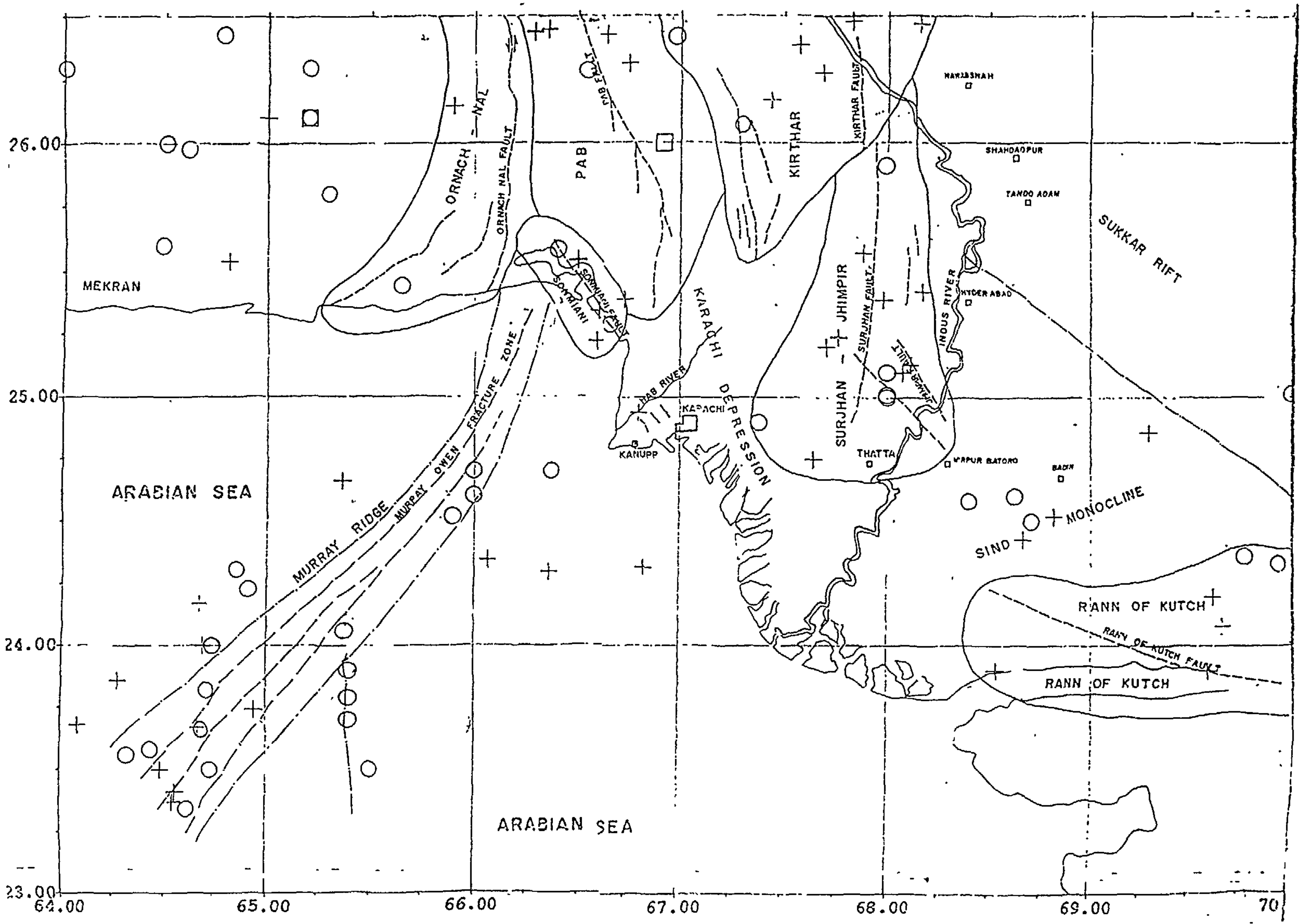

FIG.3. SEISMOTECTONIC OUTLINE OF COASTAL AREAS OF PAKISTAN

3.5 RLE or SL2 Level Earthquake

Values of site accelerations were estimated using the attenuation relationships developed by Sadigh (1987) and Idriss (1987) for each of the maximum earthquakes determined in the seismotectonic analysis.
The estimated 'g' values (84.1 percentile) for each of the seismogenic structures / seismotectonic provinces are given in Table 1. The estimated values varied from .12 g to .20 g from the important structures. As such, 0.20 g has been proposed as the RLE or SL2 level earthquake for the KANUPP site.
Site specific spectra generated using Sadigh's attenuation relation is shown in Fig. 4.

TABLE 1

PEAK GROUND ACCELERATIONS AT KANUPP DUE TO DIFFERENT EARTHQUAKE SOURCES

SEISMOGENIC STRUCTURES/ SEISMOTECTONIC PROVINCES	MAX. RECORDED/ HISTORICAL EQ.	MAX. CREDIBLE EARTHQUAKE	EPICENTRAL DISTANCE (km)	ACCELERATION (g) 1	2
The Rann of Kutch	6.1	7.8	155	0.07	0.04
The Surjan-Jhimpir	5.6	7.7	50	0.20	0.18
The Sonmiani	5.1	7.1	40	0.18	0.16
The Ornach-Nal	5.9	7.6	80	0.12	0.09
The Pab	6.4	7.3	50	0.17	0.14
The Kirthar	6.8	7.0	85	0.07	0.05
The Murray Ridge	5.9	7.7	50	0.20	0.18
The Makran Thrust	8.3	8.3	225	0.07	0.03
The Sukkar Rift	6.0	7.0	160	0.03	0.02
The Sind Monocline	5.2	6.2	100	0.03	0.01
The Karachi Depression	5.1	6.1	0	0.14	0.12

1 Idriss (1987)
2 Sadigh et al (1987)
All g. value are + 1 sd

4.0 GEOTECHNICAL INVESTIGATIONS

In order to reconfirm the geotech parameters, 3 boreholes were drilled in the vicinity of the reactor building till a depth of 50 m, samples taken and tested. A cross hole survey was also performed in these holes after making necessary arrangements.

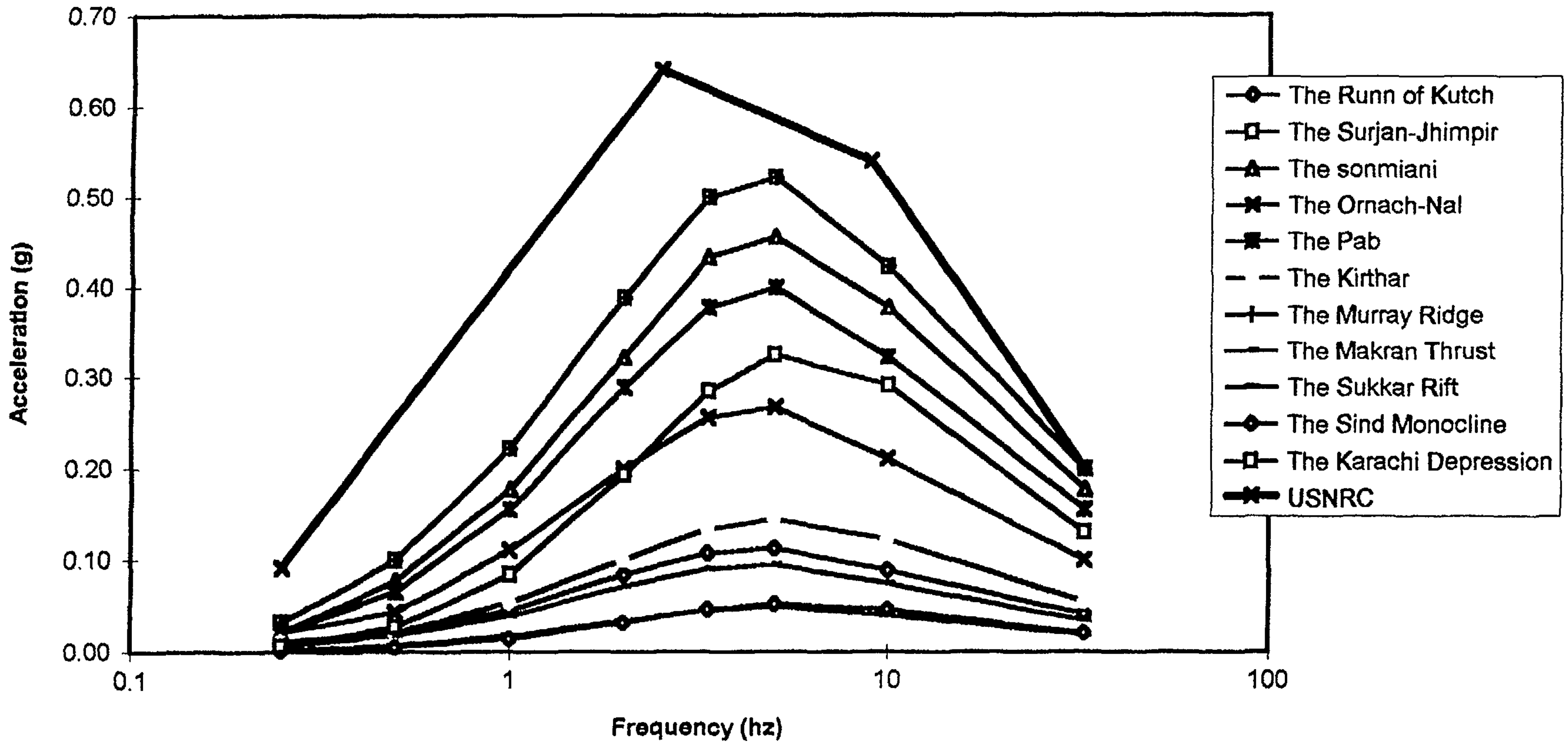

FIG. 4 Peak Ground Acceleration at KANUPP due to maximum credible earthquakes at different zones

The drilling was performed with a rotary drilling machine using NX carbide steel bit and double tube core barrel for countinous sampling. Drilling mud was used during the entire drilling work to ensure good core recovery.

As a result of drilling & sampling, the underlying strata was reconfirmed. The overburden in the plant area consists of unconsolidated silts, coarse sands and gravels, with a thickness of 3-6 meters. The predominant geological formation is laminated sandstone, interbedded with clay, claystone & limestone. Tests relating to natural moisture content, dry density, direct shear and uniaxial compression were performed in the laboratory.

The crosshole seismic survey was performed by grouting the PVC casing in the drilled boreholes, spaced at 8 meters. The equipment used comprised of seismograph, shear wave hammer, triaxial geophones and a sino slope indicator. Readings were taken by inflating / deflating the geophones at 1 m interval. The shear wave velocity varies from 885 m/s at 7 m to 1000 m at 45 m depth while the compressional wave velocity varies from 1560 m / s at 7 m to 2075 at 45 m.

5.0 IMPLEMENTATION OF SHORT TERM ACTIONS

In order to enhance the seismic capacity of the plant in a short time at nominal cost, short term actions were recommended by the IAEA mission for the following equipment:

- Batteries room
- Control room panels
- Distribution room
- Emergency diesels
- Ventilation fans and coolers
- Unit air cooler
- Bridges / cranes
- Large tanks, heat exchanger & pressure vessels
- Main feedwater / steam lines
- Fuelling machine vault
- Miscellaneous equipments

All this equipment is either unanchored or inadequately anchored. The short term fixes task requires simplified analysis with a preliminary value of Review level earthquake and anchoring design. A value of 0.25 g based on conservative assumptions was therefore selected for this task.

The tasks relating to short term actions could not be immediately implemented due to various reasons but are currently in progress and the portion relating to analyses / fixes design will be completed by November 1997.

The structural analysis of battery racks and control room panels have already been completed and retrofitting / anchoring details have been provided to the implementation division.

The structural analysis could have been performed using simplified techniques only but dynamic analyses were also performed to study the behaviour in more detail. Consequently, 3-D models were developed and analysed with spectra fabricated by consulting relevant references. A computer program, SAPV was used for this purpose and the groundmotion was applied simultaneously in the three directions in the ratio IH:IH:0.67V. The results cross-checked with simplified techniques by performing 2-D frame analysis with an amplified static value, added confidence in the final stresses used for anchor design. 3-D models are shown in figures 5 & 6.

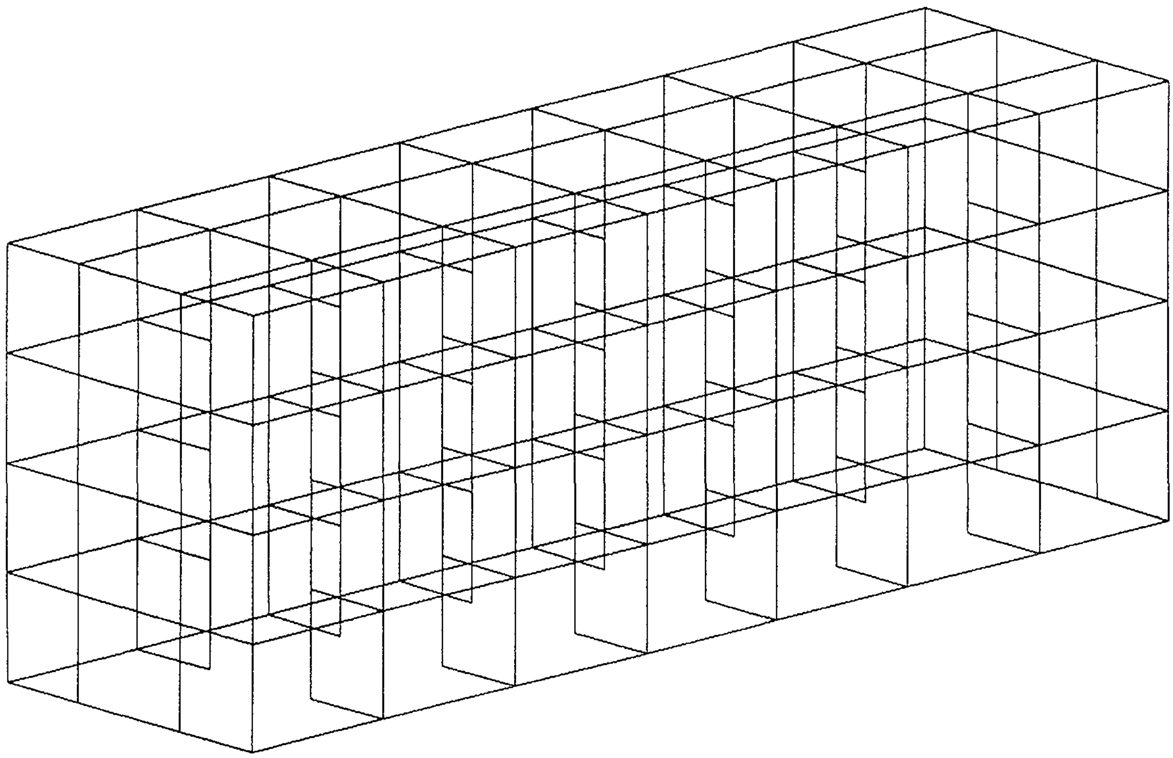

Fig. 5 3D FINITE ELEMENT MODEL OF CONTROL PANELS FOR KANUPP

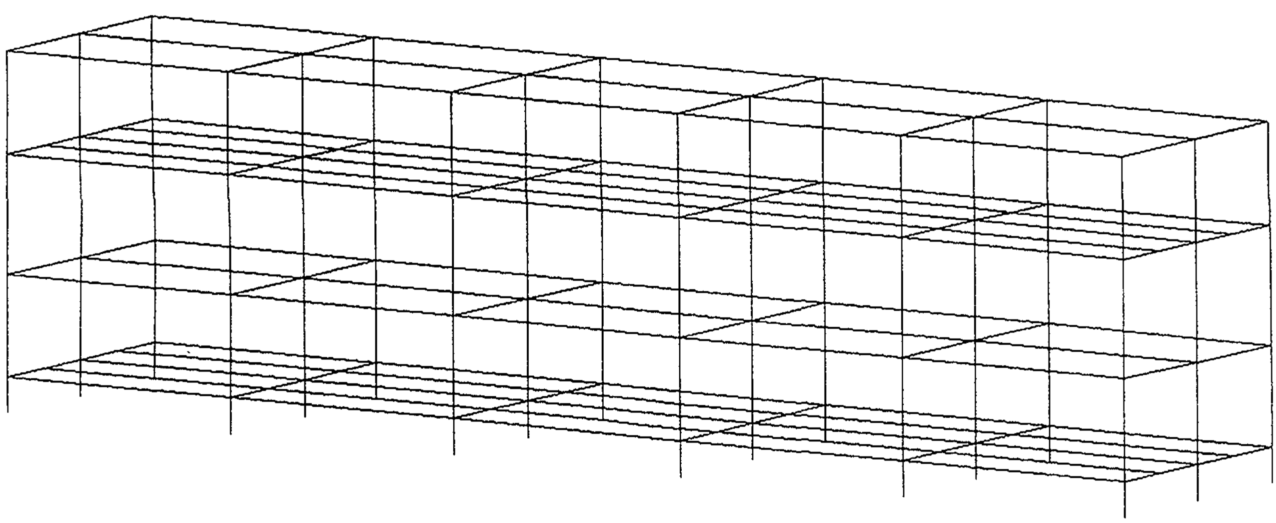

Fig. 6 3D FINITE ELEMENT MODEL OF BATTERY RACKS FOR KANUPP

The anchoring arrangements were designed by combining stresses from dead and earthquake loads. There was no yielding in any member of the control room panels but the main legs of the battery racks yielded under the anticipated seismic load and were replaced with required sizes.

For computing the stresses at the equipment anchoring locations, simplified structural analyses techniques have been used while the larger tanks have been analysed through dynamic analyses also.

6.0 FUTURE ACTIONS

Analyses / fixes design relating to short term actions shall be completed by November 1997 while the site studies have already been completed. Since the upgradation activities are being performed under the guidance of IAEA, future actions shall be decided after the completed studies / tasks are reviewed and discussed with them.

7.0 CONCLUSIONS

Two major upgradation activities relating to seismic walkdown and site studies have been completed. With the completion of these activities, the retrofittings required to upgrade KANUPP have been identified and only easy fixes are likely to achieve the desired upgradation level.

SEISMIC RE-EVALUATION AND UPGRADING OF BOHUNICE V1 NPP

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J. KUZMA
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Germany

XA0100502

Abstract

Bohunice V1 in Slovakia is a two unit WWER 440/230 who's units went into commercial operation in 1979 and 1981 respectively. The plant was not initially designed for seismic loading. Later geotechnical studies concluded that the site seismic hazard should be defined as an earthquake of MSK 8 intensity. This relates to approximately 0.25g peak ground acceleration in the free field at the site. Some early reconstruction to strengthen the plant against earthquakes was done in the early 1990s but did not include all safety significant structures and equipment. In 1996, EBO, the plant operator, entered into a contract with consortium REKON, a Siemens and VUJE joint venture, for a major reconstruction program to update all safety systems required for a safe shutdown, to improve integrity of confinement and assure spent fuel cooling. This reconstruction project includes verification of seismic adequacy of all safety related structures and equipment in the REKON scope which is not being replaced by new construction. Siemens and EQE International are jointly conducting the seismic verification and required upgrading for the existing structures and equipment.

Criteria for the verification and upgrading were developed for the project utilizing Technical Guidelines provided by IAEA, Reference 1, and linking them with international and local codes and standards and specific methodologies developed for similar projects in the US and Western Europe. The criteria are briefly discussed herein and are summarized in a companion paper, Reference 4.

Because of the major improvements being implemented in safety systems, much of the essential safety related equipment is being directly replaced or complete new systems are being constructed that supersede existing ones. Consequently, a significant amount of the equipment that would normally require seismic adequacy verification is deleted from the verification scope (see Table 4). The reconstruction project will continue through 1999. This paper summarizes the progress to date in seismic adequacy verification of existing structures and equipment which will remain as essential elements of the plant safety systems.

1. INTRODUCTION

The Bohunice site consists of two units of the first generation WWER 440/V230 (V1), and 2 units of the second generation WWER 440/V213 (V2).

The V1 units went in operation in 1979 and 1981 respectively and have made a high contribution to the Slovakian power supply. Up to 1990, the Slovakian operator realized more than 1000 modifications to the original Russian design of the V1, which increased the safety aspects of the plant.

From 1991 to 1993 the so called "81 + 14 task program" was realized which further improved the safety of the plants. The most important tasks of this "small reconstruction" were:

- Annealing of the reactor pressure vessel
- Improvement in emergency operating procedures
- Improvement of the probabilistic safety analysis
- Substantial system modifications

As a result of the small reconstruction, a decrease in the probability of core damage by a factor of 2 as well as the increase of the confinement tightness by a factor of 40 was achieved.

At the beginning of 1994 the Slovakian licensing authority, UJD, required additional safety improvement of the V1. In preparation for significant safety upgrades, a "Basic Engineering Program" was started in spring of 1994 by Siemens/KWU with the goals of:

- Improvement of the mitigation of loss-of-coolant
- Improvement of the reliability of emergency cooling system
- Decrease of radiation release by improvement in the tightness of confinement
- Improvement of seismic safety

After establishing requirements to achieve these improvements, a consortium of Siemens/KWU and the Slovakian institute VUJE contracted with the Bohunice owner in April, 1996, to do a step-by-step reconstruction of the V1 units.

The program is scheduled from 1996 to 1999 and is budgeted to approximately 275M DM, whereas Siemens will deliver 40% and VUJE 60% of the service scope. The REKON projects covers 18 tasks, Table 1, which significantly improve:

- Electrical and I+C Systems
- Emergency Core Cooling System
- Emergency Feedwater System
- Cooling Water System
- Relief Valves in Primary and Secondary System
- Fire Safety
- Seismic Safety

Part of the modifications have already been installed, whereas in this years outage six main steam control and stop valves and part of the Emergency Feedwater System have been realized. Upgrading will continue through 1999.

2. *SEISMIC REEVALUATION CRITERIA*

It is internationally accepted that for upgrading existing NPPs it is not necessary to qualify all safety related structures, systems and components (SSCs) according to current standards for new design. Adequate safety can be achieved by applying alternate approaches like seismic experience, testing experience and analytical technique with allowable stresses beyond the elastic limits.

Table 1

18 TASKS OF REKON PROJECT FOR BOHUNICE V1

Task	Description	Schedule
1	Reconstruction of Pressurizer Relief Station	1997
2	Reconstruction of Super Emergency Feedwater System	1997
3	Reconstruction of Main Steam Relief Station	1996/7
4	Upgrading of Emergency Electrical Power	1996/7
5	Reconstruction of ECCS and Leakage Return System	1998/9
6	Improvement in Fire Safety	1997-1999
7	Reconstruction of Plant Normal Power Electrical System	1996-1999
8	Reconstruction of I&C	1996-1999
9	Reconstruction of Spray System	1997/8
10	Upgrading of Purification System	1997/8
11	Increasing Confinement Tightness	1997
12	Upgrading of Confinement Strength	1997/8
13	Upgrading of Venting System	1999
14	Reconstruction of Technical Water System	1997-1999
15	Reconstruction of HVAC and Chilled Water System	1999
16	Seismic Upgrades of Structures and Equipment	1997-1999
17	Development of iRLE	1996
18	Development of Response Spectra	1996

The fundamental guidelines for Seismic Assessment in the REKON Project are contained in "Draft, IAEA Technical Guideline for Reevaluation Program of Bohunice NPP Units V1-V2" (Reference 1). These guidelines, though incomplete and in draft form, consolidate reevaluation criteria from several reevaluation programs in the U.S. and focus on using the Seismic Qualification Utility Group (SQUG) Generic Implementation Procedure (GIP), Reference 2, which was developed to resolve the seismic reevaluation issue of older U.S. NPP's having little or no seismic design. These criteria have been accepted by the NRC for resolution of USI A-46. The GIP combines seismic and testing experience and well-defined analytical procedures. This procedure has become internationally recognized and is now a basis for several seismic reevaluation programs in various countries.

The GIP is focused on U.S. plants and their equipment, which is characterized by twenty generic classes, and is supported by a comprehensive database. This equipment in the database is primarily U.S. manufactured with a representative amount of equipment manufactured in Europe and Japan. Therefore some additional work was required for use as a general assessment basis for seismic reevaluation in Bohunice plant.

The Seismic Upgrading Program of the REKON project is slightly different than the intent of USI A-46 and also covers items which are not part of the GIP such as piping and HVAC, consequently additional criteria had to be established. Siemens has joined SQUG in order to utilize the methodology of the GIP. EQE, their partner in the seismic evaluation of Bohunice V1, was one of the major contractors that developed the GIP. Together, Siemens and EQE have developed specific criteria for the reconstruction of Bohunice V1.

For piping and HVAC, procedures similar to the seismic margin approach are utilized using analytical procedures and empirical verification guidelines for safety related systems and their supports. For non-safety related systems, which may be a spatial interaction source, some experience based methods which have been accepted by European regulators in many cases are being used as evaluation criteria.

An overview about the results of this approach and a description of additional assessment criteria, is given in a paper for 14th SMiRT Conference, "KAMM" (Reference 3) and a further Paper in Post SMiRT Conference Seminar 16, "Seismic Reevaluation Criteria for Bohunice V1 Reconstruction" (Reference 4). Table 2 summarizes the methods to be applied.

Assessment criteria for seismic reevaluation focuses on two major aspects. First is the design of the equipments and their support structures and second is the equipments quality

Table 2

SEISMIC ASSESSMENT CRITERIA

Item	Criteria
Structures	Analysis
Mechanical Components	GIP
Electrical Components	GIP, Testing
I&C	GIP, Testing
Cable Trays	Screening Tables
Piping	Analysis, Screening Tables
Piping Supports, Welding	Analysis, Screening Tables
HVAC	Screening Tables
Anchorage	Analysis, Screening Tables
Seismic Interaction	GIP, Screening Tables

and their installation. It is recognized, that many of the design elements are similar to Western plants although some features are partly missing in regards to seismic resistance. The quality of equipment installations is, however, an important issue and sometimes is very poor. In many cases the welding of substructures is inconsistent or nonexistent (welds burn through the material, lack of fusion, excess splatter, etc.). To handle this specific question some supplemental criteria for assessment of welds with respect to load path, loading and welding type have been established.

The GIP rules and variety of other assessment criteria, and the specific situation due to the installation quality requires experienced engineers having a broad knowledge in the experience-based rules and their background and being properly trained in seismic verification.

3. SCOPE OF ESSENTIAL EQUIPMENT (SSEL)

According to IAEA Technical Guideline, Reference 1, and the German KTA 2201.4 Criteria, Reference 5, a minimum set of systems and their components must be verified to be seismically adequate to achieve a safe shutdown, maintain the plant in a safe condition and confine radioactive materials. The safety functions are defined as:

- Reactivity trip
- Maintaining the reactor subcriticality
- Residual heat removal
- Pressure and inventory control
- Limitation of the release of radioactive substances

With this shutdown and confinement scenario a certain amount of systems and subsystems - so called Class 1 - are defined, Table 3, where due to different demands all equipment parts are classified in three safety functions, namely: Stability (S), Integrity of pressure retaining boundary (I) and Functional capability (F).

All other plant equipment are classified as Class 2 components, whereas systems and components which can affect Class 1 components due to falling, sliding or flooding are classified as Class 2A and are verified by seismic interaction criteria.

Because of many deficiencies in the WWER design relative to Western Standards considerable reconfiguration of plant mechanical, electrical, I+C systems and building structures is required.

The 18 REKON tasks dealing with reconstruction overcame these deficiencies by inserting either additional systems like super emergency feedwater and new service water or by replacement and separation of individual plant components.

For Bohunice V1 systems, a significant amount of the safety related items are planned to be new or modified, Table 4. REKON still is an ongoing project over the next three years. There are still numerous iterations concerning the amount and type of equipment to be replaced. Most of the ongoing iterations are focused on the electrical power and I+C systems.

Table 3

SCOPE OF SAFETY RELATED SYSTEMS AND BUILDINGS

Systems and Components		Class	
Primary Circuit and Pressurizer	Complete System	1	F, I
Coolant Purification System	Suction and Injection	1	I
Filling and Cooling of Fuel Pool	Complete System	1	F, I
Confinement Sprinkler System	Complete System	1	F
Emergency Core Cooling System	Complete (Modified)	1	F
Leakage Return System	Complete (New)	1	I
Main Steam and Feedwater	Up to Gate Valves	1	I
Blowdown System	Up to Isolation Valves	1	I
Emergency Feedwater System	Complete (Modified)	1	F
Technical Water System	Complete (New)	1	I
Air Cooling and Ventilation System	Complete System	1	I
Exhaust Air Systems	Up to Isolation Valves	1	I
Building Structures			
Reactor Building		1	S
Turbine Building		2A	S
Diesel Generator and Storage Bldg.		1	S
Emergency Feedwater Pump Bldg.		1	S
Auxiliary Building		2A	S
Service Water Pump Building (New)		1	S
Stack		2A	S
Piping Ducts		1	I

Table 4

SCOPE OF EQUIPMENT TO BE REEVALUATED

Item	Amount	Percent	
		New	Remaining
Pumps	80	15	85
Valves	1,100	20	80
Tanks and Heat Exchangers	60	25	75
Large Bore Piping (>DN 100)	700m	10	90
Small Bore Piping	5,000m	25	75
Piping Supports	1,700	25	75
Electrical Equipment	100	20	80
I+C Equipment	350	70	30
Cable Trays	25,000m	5	95
HVAC - Supports	400	70	30
Others (HVAC, etc.)	50	20	80

4. *REEVALUATION PROCEDURE*

The reevaluation work is primarily focused to either demonstrate compliance with the specified reevaluation criteria or to modify and rebuild systems and components by use of simple construction and design changes that, as a minimum, will meet the reevaluation criteria.

Before the detailed REKON assessment work, a Basic Engineering Project started in 1994 to obtain some preliminary results concerning the seismic adequacy of the plant equipment. A selective amount of about 300 safety related items of the main equipment categories located in about 40 rooms of the main building were visual reviewed during a three week walkdown in the spring of 1995. Due to previous activities of seismic reconstruction, the equipment was categorized into three different categories.

Category 1: Equipment corresponded to items already improved and modified for the previous seismic demand.

Category 2: Systems and components were currently in an upgrading process.

Category 3: Items had not been considered as priority items or correspond to interaction concerns.

No building structures were included in the Basic Engineering scope nor was the emergency generator station. For all assessment steps, the criteria of the GIP were applied.

Figure 1 shows the general process for verification of seismic adequacy of existing structures and equipment which are not to be replaced. Table 5 summarizes the implementation of the verification and upgrade process applied to Bohunice V2 in the REKON Project.

5. *UPGRADE DESIGN PROCEDURE*

As previously discussed, in most cases, the upgrade designs were conducted at the site in conjunction with the walkdowns. For equipment and distribution systems (piping, HVAC and electrical raceways) simple concepts were worked out by the walkdown team and the task of developing construction and installation drawings was given to local design firms who also were at the site in adjacent offices. Loads for detailed design were usually developed by the walkdown team and support staff using simple hand calculations or simple computer models. Final sizing was often done by the local design firms. In a project such as this, it is absolutely necessary to utilize local design firms who are familiar with local construction standards and who are authorized by their governments to make fabrication and construction drawings.

This process worked out to be very efficient. The Western contractors with experience in seismic evaluation and upgrade design performed the vital walkdowns, made the screening decisions and developed realistic upgrade concepts, while the local designers conducted the more time consuming detailed design under the supervision of the Western contractors.

In developing the conceptual designs, it was emphasized that standardization as much as possible was important. Ease of construction was also important. For instance, the use of expansion anchors was discouraged if attachments to nearby locations could be made by welding Most upgrades of supports incorporated welding to existing structural steel members. Support members were fabricated from standard rolled shapes and complex cutting and forming was avoided if possible. Since many upgrades were generic, the designs accommodated field adjustments for generic components to compensate for differences in final required lengths, etc.

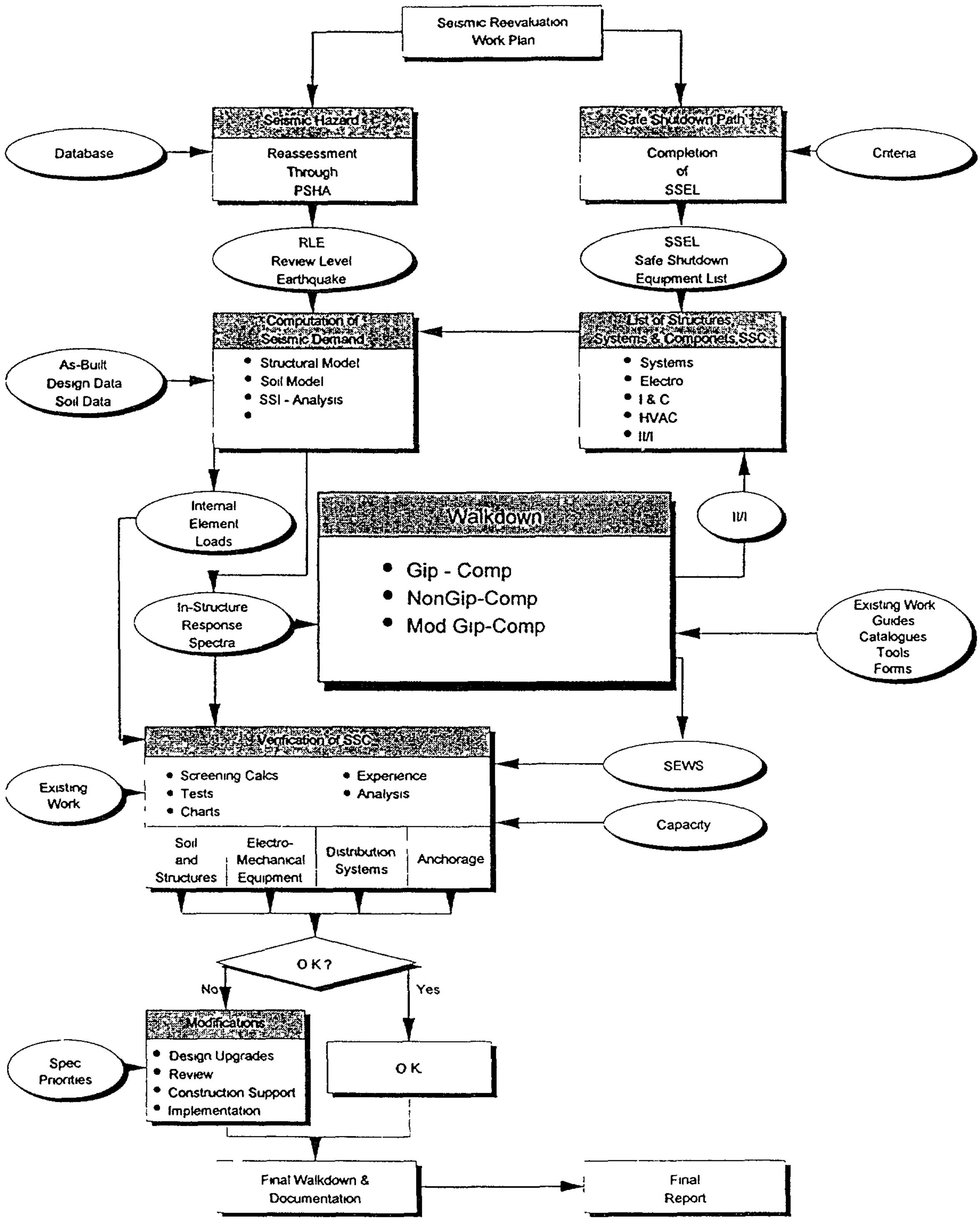

Figure 1: Flowchart for Seismic Verification of SSCs

Table 5

STEPS IN SEISMIC ADEQUACY VERIFICATION OF EXISTING BOHUNICE VI COMPONENTS AND STRUCTURES

1.	Review existing documentation from the small reconstruction program.
2.	Prepare reevaluation criteria for: GIP components Modified GIP components Non-GIP components
3.	Develop schedule for evaluation and reconstruction corresponding with scheduled outages.
4.	Establish a site office and full-time core staff.
5.	Perform walkdowns and screening.
6.	Define analytical tasks if required.
7.	Develop upgrade concepts if screening criteria are not met.
8.	Finalize upgrade designs.
9.	Prepare final report documenting the screening results for items screened out and the upgrading designs.

For most cases, the designs were deliberately conservative in order to minimize expensive Western engineering time to fine tune loads and member sizes. Also, in the interest of standardization, upgrades were designed for the highest demand in the plant, whereas many of the components of similar construction were located in regions of lower demand.

6. *RESULTS OF ASSESSMENTS AND UPGRADE DESIGN SOLUTIONS*

As of this publication, most of the walkdowns have been completed and upgrade concepts have been developed. During the next outages, installation of upgrades will take place and some final iterations on seismic adequacy verification and upgrade designs will be performed. Following are some of the major findings of the walkdowns and screening and highlights of the upgrading.

Mechanical and electrical components were initially evaluated using the GIP criteria for screening. The fundamental screening requirements of the GIP are that the equipment is included in the earthquake experience equipment class, the capacity is greater than the demand and that the equipment is free of systems interaction effects. There are also some detailed screening caveats which are specific to each of the 20 equipment classes. The first requirement that the equipment is included in the earthquake experience equipment class is more that a generic categorization of each equipment into one of the 20 seismic experience based equipment classes. The design and construction of the equipment being evaluated must be determined to be similar to the equipment in the earthquake experience database. The database that supports the GIP screening criteria consists primarily of US equipment but does contain some Western Europe and Japanese equipment. In many cases the equipment of

Czech, Russian, Polish and Slovakian origin is fundamentally very similar in design and construction to the official database and the representation in the database is satisfied without controversy as long as all associated screening caveats are met. In other cases, the equipment is not so similar and the comparison to the database is more challenging.

One of the capacity to demand criteria for screening of equipment is based on comparison of the ground motion spectrum to the seismic experience based SQUG bounding spectrum. This capacity to demand screen requires that the equipment be located less than about 13m above effective grade level and that the fundamental frequency is greater than 8 Hz. Alternatively, the in-structure response spectrum must be enveloped by 1.5 times the SQUG bounding spectrum. In this case, there is no limit on elevation or on the fundamental frequency. Because of high amplification of the in-structure spectra, a lot of the equipment at less than 13m elevation have spectral input motion that exceeds 1.5 times the SQUG bounding spectrum, thus in order to be screened, the fundamental frequency must be demonstrated to be above 8 Hz, or the equipment must be braced or stiffened.

In many cases, upgrades were done to stiffen the equipment and assure that the screening criteria were met. Upgrades were usually not done to satisfy a stress limit based upon a detailed stress analysis. In most cases if there was a stress issue, there was also a stiffness issue for screening and the upgrade design alleviated both issues.

Some initial upgrading of equipment had been conducted by local organizations in the early 1990s. These upgrades consisted of anchoring some of the essential electrical power and control cabinets, anchoring of some unanchored mechanical equipment, internal bracing of low voltage switchgear and DC distribution panels and stiffening of panels in the 6kv breakers to which relays are mounted. In many instances these previous upgrades proved to be inadequate for various reasons. Primarily though the actual upgrades were not in accordance with the analytical designs.

Mechanical Components: In most cases, mechanical equipment outside of the primary system had not been previously upgraded and was either unanchored or the existing anchorage was inadequate for the iRLE. Also, many cases occurred where steel frame supports of mechanical systems were too flexible for screening and when analytically evaluated were found to be significantly overstressed. In general, the upgrades for equipment and support frames were accomplished by bracing the component to a steel structural element or to a concrete wall to resist the overturning moment resulting from horizontal seismic inertia forces and to minimize the labor in installation. Reinforcement of load paths and addition of expansion anchors to anchor components from the base only was avoided except where necessary. In most cases attachments of the component or support sub-structure to a structural member by welding was the most efficient. In some instances where the component was passive and served only as an anchor point for piping beyond a second isolation value, the installation of stops to prevent sliding was all that was necessary.

For some pumps, the anchorage and nozzles were marginal when piping reaction loads were considered. In many of these cases, the best solution was to support the piping to minimize loading on the pump nozzles and anchorage.

Some upgrades had previously been done to motor operated valves with extended operators which would not pass the GIP screening criteria. In some instances, the upgrades violated one of the GIP fundamental screening criteria that if the operator is braced, the valve body or adjacent piping must also be braced to a common structural member. Some additional bracing and modifications to existing bracing was necessary to satisfy the GIP screening. In a few instances, calculations were performed to verify the adequacy of valves which could not be screened.

Electrical Components: The construction of most electrical cabinets would not meet the GIP screening criteria, primarily due to the fact that the sides were open and provided no shear stiffness in the front to back direction. Also, in many cases, the cabinets were very flexible in the side to side direction even when bolted together. Many of the electrical cabinets had previously been upgraded but most upgrades proved to be inadequate. Anchorage of electrical and control cabinets had been accomplished by installing angles on the concrete floor below the concrete topping and welding the cabinets to the angles or by welding or rewelding cabinets to steel embeds. Some cabinets were stiffened by addition of internal bracing. Not all essential cabinets were upgraded and for those that were, in almost all cases, the upgrades were inadequate to provide the required stiffness for screening or to justify reasonable amplification factors for which existing component tests could be used for seismic adequacy verification. The inadequacy resulted primarily from inconsistent installation of internal bracing, poor or unknown quality of welding and expansion anchor installation, prying action on expansion anchors and incomplete evaluation of load path. For electrical cabinets, the most effective way to alleviate all potential problems was to top brace them to adjacent structural members to increase stiffness and to resist overturning. The fix at the base then only required resistance to base shear.

In several rooms that contained circuit breaker and control cabinets, the cabinets were welded to raised steel floors constructed of a gridwork of channel. All of these steel floors needed to be upgraded to increase the stiffness. The lack of stiffness in many cases resulted from the steel floors not being built in accordance with the drawings. Also, in many cases, the steel floor gridwork was only supported for dead weight whereas the drawings showed positive attachments of the vertical supports to the floor. Upgrading of the steel flooring required addition of horizontal cross bracing and some diagonal bracing to the concrete floor.

In the 6kv switchgear, some stiffening of the panels to which relays mount had been done in earlier modifications. In this case the modifications were found adequate to justify lower amplification factors than suggested in the GIP. In this case. relays are not mounted on the front door panel as seen in many U.S. manufactured switchgear and the stiffened internal panels resulted in significantly lower amplification. Relays in electrical power equipment and instrumentation and control equipment were evaluated separately as described in a following subsection.

The 6kv transformers required bracing of the internal coil assembly. The top of the coils were connected to the metal enclosure which first appeared to result in top bracing of the coils but upon a close examination it was determined that the coils were actually supporting the enclosure. An internal A-frame was designed to stabilize the upper portion of the coil assembly and enclosure and to reanchor the transformer to the concrete floor.

Piping: During the small reconstruction program, some of safety related piping was upgraded to resist seismic forces. GERBS dampers were added to the primary coolant system piping and components to stabilize the six primary loops. Other piping connecting to the primary loop system was also reinforced by the addition of GERBS dampers. In the REKON project, all safety piping within the scope of reconstruction is being reassessed and upgraded if necessary.

Some of the analyses from the small reconstruction project were initially reviewed. These analyses were conducted for a different earthquake than is currently defined so the objective was to see if current loads exceed the loads used in the small reconstruction and also to verify the adequacy of the small reconstruction modeling. In some cases the reviewers disagreed with the existing modeling and that work will be revisited using the most current definition of the Review Level Earthquake. This may result in modification to the supports for the primary system. For most of the piping, whether it was included in the small reconstruction or not, the simplified walkdown and screening criteria were applied and, where warranted upgrades were recommended.

The walkdowns and screening revealed many issues of poor construction of pipe supports, improper supporting of heavy motor operated valves and a general lack of supports for seismic loading. In some instances, it was observed that GERBS dampers had been placed in illogical locations and had been used when rigid struts would have been acceptable. Several instances were noted where pipe guides, which were supposed to allow thermal growth in one direction, were binding and not allowing thermal movement.

The solution to most of the issues identified by walkdown and screening, using chart methods, was to add simple supports or fix existing supports. In most cases, the upgrade designs were accomplished in the field without computer modeling of the piping system or without conducting detailed analysis of supports. New supports were generally selected from standard configurations contained in the pipe routing guidelines. Screening by experienced Western contractors and preparation of detailed fabrication and erection drawings by local contractors proved to be very efficient to resolve piping seismic issues.

Cable Raceways: Because of fire separation issues and the addition of new I&C, many new cable raceway systems are being added. These new systems were designed for seismic loading by classical stress analysis methods. There were still a large amount of cable raceways that were to remain in service, which for the most part, required upgrading. The GIP with some modification was used for initial screening and design of upgrades. Typical discrepancies found during the walkdowns and screening were:

- Trays not attached to raceway supports
- Non-ductile connection of raceway supports to structures
- Floor to ceiling columns in the cable spreading room lacked sufficient flexibility to accommodate vertical differential movement of floors
- Overloading of raceways
- Unacceptable welding quality

The upgrade designs were performed primarily in the field. In general, the upgrades focused on altering the details to meet GIP requirements rather than to meet classic structural strength criteria. In this manner, the upgrade designs could usually be accomplished without detailed mathematical modeling or detailed calculation of strength for supports. New supports were attached to the existing structure by welding where possible. Expansion anchors were used to attach supports to concrete only if welding to steel structures was not practical.

HVAC Ducting: The initial screening was done by walkdown and comparison to allowable span charts. The span charts were based upon ducting capacities derived from test data. In almost all cases, existing HVAC ducting required resupporting. The typical detail for existing support of the ducting was by rod hangers where the rods were attached to the edge of structural I beams by poor quality welding. The only lateral support provided to ducting was at wall penetrations. but most of these interior walls were of unreinforced masonry and required stabilization measures as well.

Most of the new supports for ducting were attached to existing structural steel by welding. Use of expansion anchors into concrete was avoided if possible. Just as for piping and cable raceways, the support designs were primarily done in the field using standard configurations contained in the routing guidelines. Very little detailed analysis of supports was required.

Relay Evaluation of Electrical Distribution and Control Systems: The distribution systems of Bohunice and the relay I&C-cubicles consisted in the past exclusively of conventional switchgear cabinet types; no motor control centers were used. The mechanical design varied slightly depending on the voltage level, function, date of manufacturing and manufacturer. Variation is partly as a result of the available equipment at the time of construction of the plant and/or of the responsibility of the different supplier and partly as a result of the historical development. The design is dominantly of Czech origin. Some parts, like the trip breakers, are from the former Soviet Union. But, fortunately, for physical reasons the basic construction of the feeders and relay I&C constituting the systems turned out to be quite comparable. Generally, the same design is used for safety and non-safety systems.

As mentioned in Section 3, during the course of the reconstruction of the Bohunice plant, major parts of the safety systems have to be replaced. The new equipment to be installed will be qualified by conventional shake table testing. Because of almost total replacement of I&C, the group of remaining equipment containing relays could be reduced exclusively to the distribution systems for the emergency power supply. For reasons, which will be explained in the following, within the emergency power supply we did not distinguish, whether a piece of equipment is necessary for the mitigation of the seismic event or not (including all supporting functions and consequential functions). Regardless of function and level of safety, with few exceptions, switchgears and I&C-cabinets are generally equipped with devices which may vary in size and power dependent on the special task, but types, manufacturer, and functional arrangements of the constituting parts necessary for the active functions are in most cases very similar.

Medium Voltage Distributions: The 6kV medium voltage emergency power distribution systems are exclusively made up of bus bars and feeders; they contain:

- Circuit breakers
- Interfaces (relays) to I&C (automation and control)
- Protective relays (overload, short circuit, electric arc, etc.)
- Time relays
- Mini circuit breakers for the power supply of the protection and control circuitry
- Relays for electrical interlocks (if any)

Low Voltage Distributions (AC and DC): These 400v AC and 220v DC systems are exclusively made up of bus bars and:

- Circuit breakers
- Load-break switches
- Break switches
- Contactors
- Manually operated switches
- Time relays
- Protective relays (overload, short circuit, etc.)
- Relays for electrical interlocks (if any)

I&C Cubicles Including Diesel Generator Control: These cubicles contain:

- Relays for automation and control
- Time relays
- Memory relays
- Mini circuit breakers for circuitry power supply
- Contactors

The total number of remaining cubicles and local distribution boxes of the original design is small. As a consequence we decided to deviate from the general procedure and to perform the screening based solely on equipment types necessary for relevant and typical functions rather than on the specific functions required and the associated equipment.

Using a notebook computer allowed a highly effective data collection during the walkdown, avoiding repeatedly recording the same devices in different locations (see Table 6). This way, it took in total less than three days to collect the data looking into each of the 81 cabinets and boxes. As a side effect we made sure to get the latest information. That is, the potential of the influence of unrecorded changes of design and type of equipment was eliminated.

As a result of the so-called small reconstruction, performed from 1991 to 1993, the existing documentation in the plant included a collection of reports which demonstrate by test the seismic resistance of nearly all relays and breakers to be evaluated. The tests were single frequency, single direction. The adequacy of the test results was checked by response spectra comparison (see Figure 2) where the single frequency envelope of response was compared to the required response spectra. In performing the spectral comparison for single device tests, it is necessary to consider the amplification of the support structure. As already mentioned above, the amplification was quantified to be generally in the range of a value of three. This generic value is acceptable for all cubicles and boxes, provided improvements suggested by REKON like top bracing and local reinforcements are properly introduced.

The relay evaluation work, including some minor re-qualification testing, will be completed by the end of 1997.

Systems Interactions: Miscellaneous systems interactions had to be stabilized to prevent falling, swaying, overturning or sliding into safety relevant equipment. A common source of systems interaction was unreinforced masonry walls. Some initial stabilization of some of the walls had previously been done. In these cases, simple steel angle braces were

Table 6

6KV - SWITCHGEAR
DESIGNATION OF DEVICES

6 kV - Switchgear Desgnation of devices			500 kW - consumer	500 kW - consumer	500 kW - consumer	Spare-Cabinet	Spare-Cabinet	Incom'g Feeder	Measuring	Feeder.to 8r6-08.11b	Feeder Transformer	250 kW - Consumer	160 kW - Consumer	Feeder. to 1rd6-08-12	250 kW - Consumer	Feeder. to DG1R	Feeder to Second Part of Bus Bar	Verb. der B1-Schiene
Distribution: rd 6 8.11 Documentation complete Level: 0,0m	**qualified**	**Protokoll No. / Cabinate No.**	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Overload Protection	yes	EP001/01	X	X	X							X	X	X	X			
Overload Protection	yes	EP002/01													X			
Overload Protection	yes	EP004/01									X						X	
Grounding Protecton Relay	yes	EP006/01							X									
Auxiliary Relay	yes	EP010/04													X			
Circuit Breaker	yes	EP011/01	X	X	X	X	X	X			X	X	X	X	X	X	X	
Load Switch	yes	EP012/01	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Electric Arc Protection	yes	EP013/01	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Time Relay	yes	EP015/02	X	X	X				X			X	X	X	X	X		
Short Circuit Protection	yes	EP016/02												X		X	X	
Memory Relay	yes	EP017/02	X	X	X						X	X	X	X	X			
Contactor	yes	EP018/02	X	X	X							X	X	X	X	X	X	
Programable Protective Relay	yes	EP011/03							X									

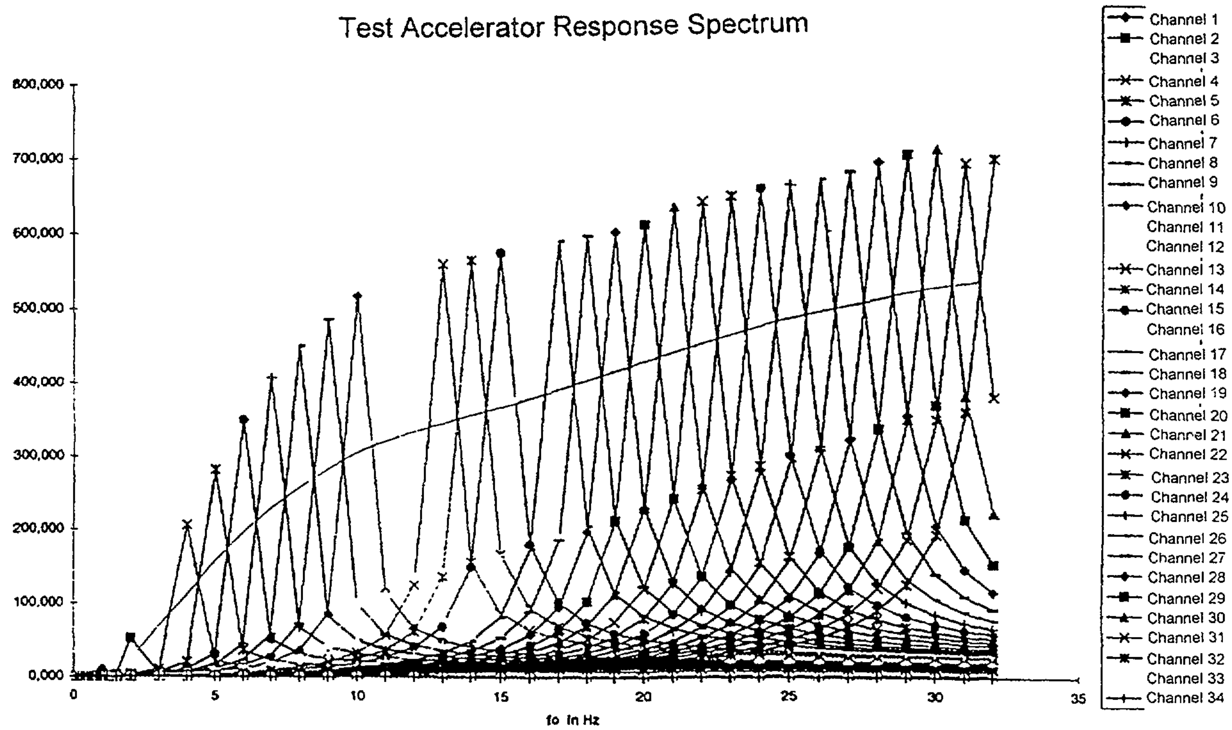

Figure 2: Enveloping of Single Frequency TRS

welded to existing steel columns to prevent out of plane collapse of the walls. Similar concepts were applied to walls that had not been previously upgraded. Another common systems interaction was the potential impact of adjacent cabinets. This is primarily a relay performance issue. Most electrical and control cabinets with similar function were adequately bolted together to prevent impact. The most common impact issues were cases where unlike cabinets were placed too close together and they had to be upgraded by adding a connection at the top of the cabinets. In one instance, a building joint ran under an essential row of 6kv switchgear. Some of these switchgear had to be relocated.

While systems interactions were quite common, their fixes were usually very simple.

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SEISMIC QUALIFICATION OF EXISTING NUCLEAR INSTALLATIONS IN INDIA — A PROPOSAL

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Abstract

In India, the work toward seismic qualification of existing nuclear facilities has been started. Preliminary work is being undertaken with respect to identifying the facilities which would be taken up for seismic qualification, approach and methodology for re-evaluation for seismic safety, acceptance criteria, etc. Work has also been started for framing up the criteria and methodology of the seismic qualification of these facilities. Present paper contains the proposal in this respect. This proposal is on similar lines of the present practice of seismic qualification of NPP , as summarized in the Appendix, but has been modified to suit the special requirements of Indian nuclear installations.

1. INTRODUCTION

Earthquakes have the potential to induce common cause failure. The frequency and severity of seismic hazard is site related. Measures for protection against seismic hazard are incorporated into the plant design. Plants built using earlier standards may have deficiencies both in the requirements relating to the derivation of design basis ground motion (DBGM) as well as in criteria and measures (i.e. design features) for protection against the effects of seismic hazard. In view of this, it is necessary to re-evaluate the capability of the structures, systems and component (SSC) of older facilities to withstand the effect of earthquake in line with the current criteria.

The fundamental safety principles of nuclear power plants (NPP) and the basis for judging the safety of NPPs built to earlier standards are given in references - 1 and 2. The approach and methodology for the evaluation of seismic safety of existing plants built to earlier standards can be formulated using this basis. Reference-3 outlines the criteria for re-evaluation of safety of WWER type NPPs of Eastern Europe. Methodology had also *been developed for seismic re-evaluation of PWR based NPP [4,5,6,7] . A brief overview of the present practice of seismic qualification of existing NPP is given in the Appendix.

Indian nuclear facilities includes all facilities under nuclear fuel cycle and associated activities covering from the front end to the back end of the nuclear fuel cycle processes and also associated industrial plants (see Fig. 1). Example of such facilities are Nuclear Power Plants (NPPs), Research Reactors, Heavy Water Plants, Spent Fuel Reprocessing Plants, Fuel Fabrication Plants, etc. The present approach of aseismic design of the Indian nuclear facilities, specially the nuclear power plants,

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* Note: Numerical number inside the square bracket indicates reference number.

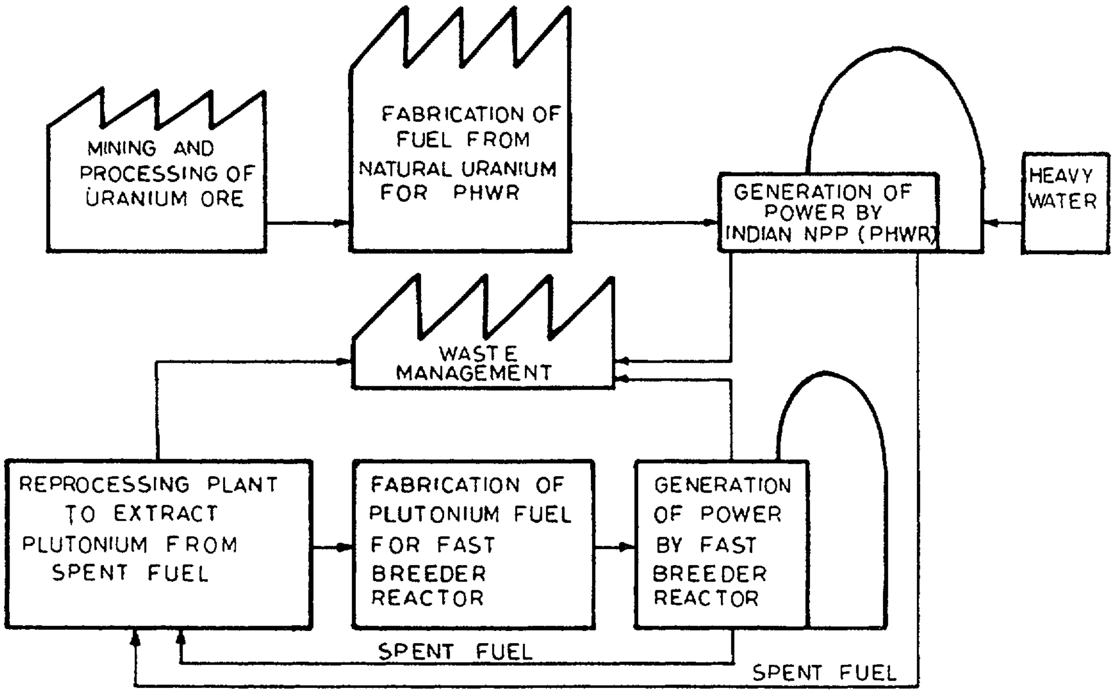

FIG. 1. Indian nuclear fuel cycle.

has been evolved over a period of time. Some of Indian nuclear facilities were constructed several years back. For example Tarapur Atomic Power Station, a light water reactor based NPP, was commissioned in 1969. Two CANDU reactors of Rajasthan Atomic Power Station were commissioned in 1972 and 1981 respectively. The first indigenised pressurized heavy water reactor based NPP was commissioned at Kalpakkam in 1984. In addition to these, a number of facilities of Indian nuclear fuel cycle were constructed about 30 years back. If these installations have to be re-evaluated in terms of current practices for seismic safety, the proposal presented in this paper is designed to address such requirements.

2. PROPOSED SEISMIC QUALIFICATION METHODOLOGY FOR EXISTING INDIAN NUCLEAR FACILITIES

2.1 Objective

The objective of the proposed seismic qualification methodology is to carry out safety re-evaluation of existing Indian nuclear installations against the perceived seismic hazard (site specific as far as practicable) using current postulation and aseismic design approach [7,9,10], principal objective of nuclear safety [8] and also present status of the plant.

The seismic qualification programme based on the proposed method would have three major components:

1) Deriving an earthquake level for the seismic qualification. (Experience gained elsewhere indicates that this earthquake level is expected to be larger than the one for which the installation was originally designed).

2) Assessment of seismic margin of the structures, systems and components, with respect to the above earthquake level.

3) Upgradation of structures, systems and components, if found necessary, using the information obtained from the seismic margin assessment.

All activities of the proposed methodology need to be carried out following planned programme. Flow diagram (see Fig. 2) for the proposed method is similar to the one given in ref-11.

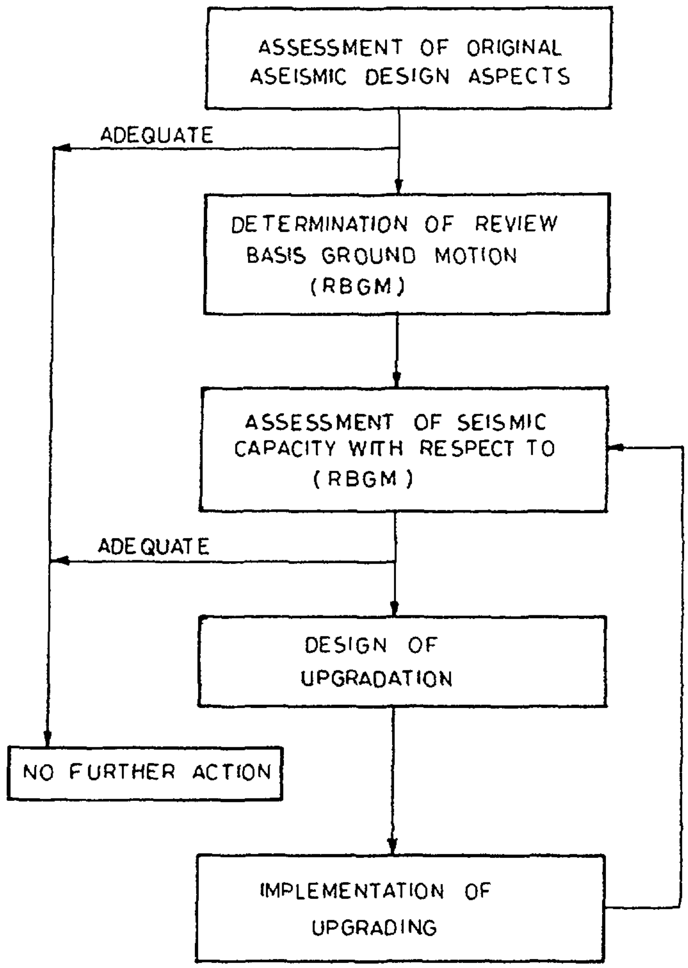

FIG. 2. Flow diagram for seismic qualification of existing nuclear installations (following proposed method).

2.2 Review Basis Ground Motion

One of the main activities of the proposed method is to determine the ground motion parameters (PGA, Spectra, etc.) which will be used in the assessment of seismic margin. The terminology review level earthquake (RLE) is used in this context [3,11]. This terminology may create confusion with regard to the other terminology related to the aseismic design of NPP used in India [9]. In India, the level of earthquake [9] refers to the severity of earthquake and not the ground motion parameters. For example, S1 level (OBE) or S2 level (SSE) earthquake. It may be noted that the terminology, design basis ground motion (DBGM) is used in defining the parameters (i.e. PGA, spectra and time history) of different level of earthquake (i.e. S1 level, S2 level) which are considered in the design of plants. The seismic qualification of existing installations basically aims at reviewing the adequacy of a nuclear installation to withstand the seismic hazard with respect to current approach and methodology of aseismic design of nuclear installations. In view of this, it is proposed to term the parameters of ground motion which would be used in seismic qualifications as review basis ground motion (RBGM).

RBGM for the seismic qualification of a NPP is the ground motion parameters corresponding the S2 level of earthquake [9]. A median plus one sigma PGA value with mean spectral shape is proposed to define the ground motion parameters of RBGM for NPP [3,17]. If deconvolution approach [18] is used in response analysis the ground motion parameters of RBGM may be taken as same as those, of design basis ground motion (DBGM) which is generally used for the design of new plant. However, This aspect needs a very detailed deliberation.

In general, conservative approach is adopted in developing the DGBM [9]. The areas, where conservatism are typically found, are in the specification of design basis earthquake, deterministic derivation of PGA, spectral shape, etc. These conservatisms are desirable for designing new facilities but all of them may not be required for seismic re-evaluation of an existing installation. Less conservative approach with minimum level of uncertainties is proposed to be adopted for deriving the RBGM [17].

All nuclear installations are not required to be re-evaluated with respect to same level of earthquake. The severity of earthquake level, to be considered in the seismic re-evaluation of an installation, should be linked with the overall safety requirement of the installation. In view of this, structures, systems and components of existing nuclear installation are proposed to be categorized in the following three groups for defining the corresponding parameters of RBGM;

Category-1

- o Systems of a NPP or a research reactor associated with the safe shutdown of reactor, decay heat removal from reactor, containment, spent fuel storage pool or others whose failure would cause radioactivity release beyond acceptable limits.

- o SSC of any other hazardous plants situated nearby NPP whose failure could jeopardise the safe shutdown of reactor and decay heat removal.

RBGM parameters of Category-1 SSCs correspond to S2 level earthquake as mentioned above.

Category-2

- o Radiochemical plants like waste management facilities, fuel reprocessing plants, etc.

RBGM parameters for Category-2 SSCs correspond to S1 level earthquake. In the absence of detailed analysis, PGA value for category-2 SSCs may be taken as 50% of the PGA value considered for Category-1 SSCs.

Category-3

- o Hazardous chemical plants whose failure would not jeopardise the safe shut down of nearby reactor and its delay heat removal or would not release radioactivity beyond acceptable limits.

For Category-3 SSCs, RBGM parameters should be as per IS· 1893 [19].

2.3 Identification of Plants and Associated Structures, Systems and Components for Seismic Qualifications.

All structures, systems and components (SSC) of an installation need not be re-evaluated for seismic adequacy. Only those structures, systems and components of an installation failure of which could lead to radiological risk beyond acceptable range would be re-evaluated. Based on this principle, following SSCs would primarily be considered for the seismic qualification;

1. Nuclear Power Plant (NPP) and Research Reactors.

   - o SSCs associated with safe shut down of reactor

   - o SSCs associated with decay heat removal from reactor

   - o Containment

   - o Spent-fuel storage pool

   - o Any other SSC whose failure would cause undue radiological releases beyond acceptable limit.

2. Chemical plants

   - o Failure of which would cause undue radiological release beyond acceptable limit.

   - o Plants and installations, failure of which would jeopardise the safe shutdown of nearby NPP, if any.

The criteria / assumptions given in para A-3.0 of the Appendix, may be followed to identify the SSCs for seismic qualifications.

In assigning the priority of the identified SSC, consideration should be given to the healthiness (ageing effect) of the SSC, whether any undesirable events occurred during the operating period of the plant, etc.

2.4 Re-evaluation of Seismic Safety

The seismic safety of a plant will be quantified in terms of seismic margin. A definition of seismic margin, similar to that given in ref. 5, is adopted in the proposal. The seismic margin is expressed in terms of the earthquake motion level that compromises the plant safety sufficiently leading to melting of the reactor core. In this context, margin is defined for the whole plant. The margin concept can also be extended to any particular structure, function, system, equipment, item, or component for which compromising safety means "sufficient loss of safety function to constitute to core melting if combined with other failure".

When this concept is extended to SSC level, it is termed as seismic capacity. The seismic capacity of SSC is the ground motion acceleration upto which, if a component is subjected, would have the ability to sustain its effect and continue to perform the intended function. Therefore, in the proposal, seimsic margin refers to the ground motion parameter (PGA) with reference to the overall plant safety and seismic capacity will refer to the same parameter level with respect to SSC.

Like the accepted practice [5], the concept of high confidence low probability failure (HCLPF) will be applied in calculating seismic capacity of the SSC. The HCLPF capacity values are approximately equal to a 95% confidence (probability) of not exceeding of about 5% probability of failure.

Both fragility analysis (FA) [4,12,13,14] and conservative deterministic failure margin (CDFM) [3,6,7,15] methods are proposed to determine the seismic capacity of SSC. Determination of HCLPF capacity from fragility analysis is significantly dependent on the judgment and accuracy in calculating median capacity, randomness variability and the uncertainty factor. Moreover, there is no consensual methodology available to develop randomness and uncertainties factor in consistent manner. As a result of these, there may exist inconsistency in the HCLPF capacity of plant when different groups of experts carry out the work [3]. On the other hand, CDFM method is a code based method. It basically assumes that if the capacity of SSC determined using the codal value of material strength and other parameters and codal criteria of strength, the HCLPF criteria in determining seismic capacity would be satisfied. As this method is principally code based, chances of existing inconsistency in HCLPF capacity of different component carried out by different groups of experts is minimum.

India has adequate experience of code based design analysis of nuclear installations. Though preliminary work in the field of fragility analysis has already been started in India, considering the difficulties still remaining in the state of the art of fragility analysis, it appears to be prudent to put more emphasis on the CDFM method in the initial period of seismic qualification work.

Assessment of the strength of structures systems and components of an existing plant is to be carried out with respect to the review basis ground motion (RBGM). The response analysis may be carried out using higher damping values than those used in the design work. The damping values suggested in the reference-3 is proposed to be used in the response analysis work. The linear response analysis would be carried out using linear spectra. Almost all the structures and components exhibit certain level of ductility, by virtue of which they may withstand a higher level of loading than those corresponding to elastic level prior to failure ("failure" is to be defined appropriately). The effect of ductility would be considered in evaluation of seismic capacity. There are two approaches available in this respect. In the first, the spectra is scaled down by an appropriate ductility factor and the response analysis is carried out using this modified spectra [15]. In the second approach, the response of structural elements determined from a linear analysis is reduced by appropriate ductility factor [11]. Both the approaches are acceptable to the proposal. However, it may be noted that the second approach seems to be more rational. The values of ductility factors, as suggested in ref.15 and those of ref. 11, may be used for the first and second approach respectively.

Seismic qualification based on experience and test results may need data from international sources.

2.5 Plant walkdown

The plant walkdown is proposed to be carried out using the similar procedure as described in references 3 and 16. Information on construction of the plant and that on current status of various SSCs of the plant would be collected and documented in this step. The activities of this step also include screening the system and components which needs to re-evaluated. The important systems, components which would be examined during the walkdown are anchorages of equipment, cable trays, and other components which may suffer excessive movement during earthquake. Possibility of spatial interaction between near by structures, components, equipments would also be examined during walkdown. Screening of component would be carried out following the similar criteria as described in reference-3.

3. SUMMARY

Preliminary work for the seismic qualification of these existing installations have been started in India. Present paper contains a proposal to outline the approach and methodology for the work. The proposal is similar to the present practice adopted elsewhere for the seimsic qualification of existing NPP with certain modifications to suit special requirements of Indian conditions. The salient features of the proposal are:

1) The SSCs related to safe shutdown of reactors, decay heat removal from reactor, containment and spent fuel storage pool of existing NPP or research reactors will be considered for seismic qualification. In addition, any other SSC of NPP and research reactors whose failure may cause radiological release beyond acceptable limit and SSC of chemical plants whose failure may jeopardise the safe shut down of near by NPP, if any, are also to be included in the list of seismic qualification.

2) The SSCs of nuclear installations are proposed to be categorised into three groups for seismic qualification work depending on the overall safety demand of the installations.

3) The RBGM parameters for Category-1 SSCs should corresponds to S2 level earthquake and are defined by median plus one sigma PGA value with mean spectral shape. If deconvolution approach is adopted, the ground motion parameters of DBGM are proposed to be considered; this aspect needs a detailed deliberations. For category-2 SSCs, RBGM would correspond to S1 level earthquake or PGA may be taken as 50% of that of Category-1. For category-III, RBGM parameter should be determined from IS1893.

4) Both fragility analysis and conservative deterministic failure margin methods based on HCLPF concept would be used to determine seismic capability of SSC. However, more emphasis is proposed for CDFM method at the initial period.

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APPENDIX

BRIEF OVERVIEW OF THE PRESENT PRACTICE OF SEISMIC QUALIFICATION OF EXISTING NPP

A.1.0 INTRODUCTION

The nuclear power plants (NPPs), constructed before early seventies, are being subjected to severe scrutiny through out the world for safety against earthquake with respect to present standard of aseismic design of NPP. Seismic qualification of existing NPP has been started in the late 1970's. A number of NPPs had already been scrutinised in different countries and several other NPPs are being examined.

A-1.1 Objective of Seismic Qualification of Existing NPP

Primary objective of seismic qualification of existing NPPs is to assess and enhance, if required, the seismic capacity of safety related SSCs of these NPPs required for safe shutdown of the reactors and to reduce the potential for release of radioactivity beyond acceptable limits during a seismic event.

A-1.2 Stages of Seismic Qualification Activities [3]

Seismic qualification of an existing NPP is generally carried out in four stages;

Stage-1: Determination of earthquake level and corresponding ground motion parameters which would be used for the qualification of SSC. This earthquake level which defines the seismic demand is known as review level earthquake (RLE).

Stage-2: In this stage, building structures of NPP are evaluated against RLE, i.e. their seismic capacity is determined with respect to RLE. The floor response spectra required to define the seismic demand for SSC housed in the building structures are also determined.

Stage-3: The seismic capacity of critical systems, such as reactor coolant system, reactor protection systems, etc. are determined in this stage. Engineering of rectification measures required for seismic upgradation of SSC, if found necessary, for which detailed analysis and test are required is also undertaken in this stage. This stage also includes the design of rectification measures of the building structures and also the large tanks, systems and components for which inadequate seismic resistance data are available.

Stage-4: In the fourth stage, seismic evaluation of auxiliary system is carried out using the experience and judgment based on the performance data of the similar component during actual earthquake or test; or during walk down by qualified personnel. The modifications, if required for these systems and which could be easily engineered in place during operation or scheduled outage, are carried out in this stage.

A-2.0 REVIEW LEVEL EARTHQUAKE: GROUND MOTION FOR SEISMIC QUALIFICATION [11,17,20]

The geological stability and the ground motion parameters are assessed according to specific site conditions and in compliance with criteria and methods valid for new facilities. The review level earthquake should correspond to the S2 level which is directly related to ultimate safety requirements (safe shut down of reactor) and is the level of extreme ground motion having a very low probability of being exceeded during the plant lifetime and represents the maximum level to be used for design and re-evaluation purposes.

In defining the ground motion parameters of RLE, a median plus one sigma peak ground acceleration (PGA) is considered along with mean response spectra ordinates.

A-3.0 IDENTIFICATION OF SSC FOR SEISMIC QUALIFICATION [11]

Each and every structure, system and component of an existing NPP need not be re-evaluated for seismic safety. The SSCs are identified for seismic qualification based on the following criteria and assumptions,

1) The plant must be capable to be brought to and maintained in a safe shutdown condition during the first 72 hours following the occurrence of the RLE;

2) Safe shutdown means hot or cold shutdown

3) Simultaneous off site power loss occurs for up to 72 hours

4) The required safe shutdown systems should fulfill single active failure criterion

5) Loss of make-up water capacity from off-site sources occurs for upto 72 hours

6) Other external events such as fires, flooding, tornadoes, sabotage, etc. are not postulated to occur simultaneously;

7) Loss of Coolant Accident (LOCA) and High Energy Line Breaks (HELB) are not postulated to occur simultaneously.

It is seen from the above that those SSCs which are associated with the safe shut down of the reactor (at least for 72 hours) in the event of S2 level earthquakes (25) and also those associated with the decay heat removal need to be assessed. Other systems, like containment system which perform the mitigatory role in connection with radiological release in the event of design basis accidents like LOCA and MSLB, are also included in the scope of seismic re-evaluation programme. The seismic safety essential list (SSEL) is the list of minimum SSC, selected for seismic safety qualification. This is an important outcome of this step of activities.

A-4.0 DETERMINATION OF SEISMIC CAPACITY OF SSC [4,5,6,7,16]

The terminology seismic margin refers to different type parameters compared to the ones which standard codes generally refer to in using the word margin. Seismic margin refers to the earthquake motion level that compromises plant safety, expressed in terms of earthquake ground motion which is generally defined by means of peak ground acceleration (PGA). Therefore, broadly, seismic margin is quantified by a PGA value; which if exceeded during an earthquake would lead to accident scenario jeopardizing the overall plant safety. The concept of seismic margin is extended to define the seismic capacity of component which is the ground acceleration value upto which if a component is subjected, will not loose its performance on intended function.

The concept of High Confidence Low Probability Failure (HCLPF) capacity is used in the assessment to quantify the seismic capacity. HCLPF corresponds to the earthquake level at which, with high confidence (≥ 95%), it is unlikely (≤ 5%) that failure of structures, systems and components required for safe shutdown of the plant will occur.

Available methodology to determine the seimsic capacity may broadly be categorized into two groups.

i) Methods based on analytical approach.

ii) Methods based on experience.

In both the methods, seismic capacities of the identified SSCs are assessed with respect to RLE. In general, RLE is greater than that was considered in the original design of the plant. In the first approach, the seismic capacity of a SSC is assessed using primarily analytical methods while in the second method the seismic margin of a given safety related SSC is assessed considering the experience on the behavior of similar type of SSC at other plants under earthquake or from the test results.

A-4.1 Determination of seismic capacity by Analytical Approach

Estimation of HCLPF seismic capacity includes response analysis, conditional on occurrence of RLE and estimation of the capacity of the structures, systems and components. Two methods are generally used for determination of HCLPF seismic capacity;

1) Fragility Analysis (FA)

2) Conservative Deterministic Failure Margin (CDFM)

Building structures, major equipment and pipelines associated with the reactor coolant system and protection system, etc. are covered by the analytical approach.

A-4.1.1 Fragility Analysis Method [3,4,5,12,13,14]

The general definition of fragility of a component is the conditional probability of its failure given a value of the response parameter, such as stress, moment, spectral acceleration, etc. For seismic re-evaluation, the component fragility is calculated by developing the frequency distribution of the seismic capacity of a component and finding the frequency for this capacity being less than the response parameter value. The capacity of a component for a particular failure mode is expressed in terms of the ground acceleration capacity. The fragility is the frequency at which the random variable, the ground acceleration capacity, is less than or equal to the specified value.

The HCLPF seismic capacity using fragility model, is given by [5];

$$a_c = a_m \exp[-1.65(b_r+b_u)].$$

Where,

a_c = HCLPF Seismic capacity determined by fragility analysis.

a_m = Median ground acceleration capacity

b_r = Logarithmic standard deviation representing randomness in capacity.

b_u = Logarithmic standard deviation representing uncertainties in median value a_m.

The median capacity a_m can be estimated as a product of an overall median safety factor times the PGA value of RLE.

A-4.1.2 Conservative Deterministic Failure Margin (CDFM) Method [3,6,7,15]

The CDFM method to determine HCLPF seismic capacity is the code based standard design analysis method. In CDFM, deterministic value of ground motion parameters of RLE(PGA and response spectra ordinates) and material properties (strength, damping value, ductility, etc.) are considered. However, The excessive conservatism, in determination of the design value of these parameters, is avoided in the case for CDFM. The guidelines of CDFM approach are [3];

1) Ground motion parameters, as outlined in A-2.0 above, is to be considered.

2) Response analysis of SSC is carried out using mean values of material properties like, damping, etc.

3) Material strength as specified in code or 95% excedence actual strength, if adequate test data is available, is used in capacity estimation.

4) The capacity or strength of a component is determined using the equations and criteria given in codes (for example, limit state methods, etc.). However, if adequate test data is available on the strength of component, 84% excedence of test data for capacity may be used. In estimation of capacity conservative values of ductility is considered.

Other important considerations for the evaluation of seismic margins capacity are;

i) The term "failure" for each of the systems, structures and components being evaluated is to be clearly defined. It is possible that there may exists several failure mode of a component. However most dominant failure mode, to be caused by the seismic event, is identified by reviewing the SSC design. This mode only is considered in the capacity calculation. Sometimes more than one mode of failure are also considered.

ii) The response analysis for RLE is conducted with appropriate damping values, which may be used if the stresses in the majority of the resisting building elements for the applicable loading combination are greater than 50% of ultimate strength for concrete or yield capacity for steel. However, higher damping values may be used for the seismic re-evaluation work if properly justified considering the stress level.

iii) Nearly all structures and components exhibit at least some ductility (i.e. ability to strain beyond the elastic limit) before failure or even significant damage. The additional seismic margin due to ductility are considered in capacity calculation.

iv) Seismic response of building structures is evaluated on the basis of dynamic analysis of models of the soil-structure system. In order to develop appropriate analysis models, special attention is given to the following;

(a) structural configuration and construction details (joints, gaps, restraints and supports).

(b) non structural elements, such as masonry or precast reinforced concrete panels that may modify the structure response. Stiffness and strength of such panels, and those of their attachments to the structure, should be accounted for in the formulation of the models.

(c) as-built material properties and dimensions of structural members.

(d) geotechnical data of foundation materials and their potential implications on the necessity to perform soil-structure interaction analysis, for which direct methods are usually being applied. For soil-structure interaction analysis radiation damping value is not limited but resultant composite modal damping would not exceed 20.0%.

iv) Combinations of seismic and non-seismic loads as per acceptable design codes.

The HCLPF seismic capacity determined using CDFM method is given by [13];

a_c = $C\ a_r$

a_c = HCLPF seismic capacity determined by CDFM method

C = $k[(S-P_N)/(P_T-P_N)]$

a_r = PGA value corresponding to RLE

k = ductility factor

S = Seismic capacity of the component against a given failure mode

P_N = Non seismic concurrent loads on the component.

P_T = Total load on the component.

A-4.2 Evaluation of Seismic capacity Based on Experience and Test Data [3,6].

Seismic qualification based on experience is basically an earthquake experience and test based judgmental procedure. The procedure is principally based on the performance of installed equipment which have been subjected to actual strong motion earthquakes as well as the behaviour of the equipment during simulated test condition. Primary sources of experience data are the non-nuclear facilities which have been subjected to strong motion earthquakes. Seismic qualification using experience and test data is carried out in following steps [3,16].

1) Establishment of various alternative methods or paths related to safe shutdown functions.

2) Identification of SSC associated with safe shut down functions.

3) Identification of SSCs which satisfy the seismic demand for qualification. This is carried out using following screening criteria.

   i) The seismic capacity of the equipment, based on earthquake experience data, seimsic testing data or equipment qualification data should be greater than the seimsic demand imposed on equipment by RLE.

   ii) In order to use the seismic capacity determined using a standardized spectrum, the equipment under consideration, should be similar to the one for which existing data bases are available and also gets the specific caveats for that class of equipment.

   iii) The equipment anchorage installations and rigidity should be adequate to withstand the seismic demand at the equipment location as per in-structure response spectrum determined from RLE.

   iv) The effect of possible seismic spatial interactions with near equipment or structures should not cause the equipment failure in performing its intended safe shutdown functions.

The evaluation of equipment with respect to above screening criteria is carried out through walk down, analyses and using supporting data. The effective and successful appreciation of the above method greatly depends on the engineering judgment of the engineers associated with the work. Active mechanical and electrical component such as motor control center, switch gears, transformers, distribution panels, cabinets and racks,etc. can be effectively evaluated by this method.

A-5.0 PLANT WALKDOWN [3,11,16]

During plant walkdown, emphasis is given to the collection and compilation of original design basis data and documentation in order to minimize the efforts required for the re-evaluation programme. Plant walkdown is principally performed to collect information on as-built conditions and to assess the seismic capacity of equipment. The important aspects of the walkdown is to examine the status of anchorages of the equipment; load path from the anchorage up through the equipment; the equipment structure; and spatial systems interactions.

In general, there could be three alternative disposition categories for each structure, system and component being evaluated during the walkdown.

1) Disposition 1: a fix is required

2) Disposition 2: the seismic capacity is uncertain and an evaluation is needed to determine if a fix is required.

3) Disposition 3: the seismic capacity is adequate for the specified RLE and the items appear to be seismically rugged.

Judgement of walkdown teams plays significant influence in working out the above disposition. When the dispositions are worked out using earthquake experience and test data, screening guidelines mention in A-4.2 above are used.

Screening guidelines are used to determine if the components are represented by the experience database and applies to the component in question. In case of the components and distribution systems for which seismic and testing experience has not been gathered and reviewed, seismic response analysis should be carried out.

Seismic walkdown may be conducted in two phases. In the first phase, which is also known as preliminary screening walkdown, disposition category 3 is identified. The disposition categories 1 & 2 require detailed walkdown and are covered in the second phase. The walk down are completed by filling up standardized screening walkdown sheet for preliminary phase and seismic evaluation work sheet for the second phase.

A-6.0 ASSESSMENT OF ANCHORAGES [11,16]

The presence of adequate anchorage is important for the satisfactory seismic performance of distribution systems and components against slide, overturn, excessive movement etc. Strengths of system and component anchorage are determined by one of the many commonly accepted methods. The load or demand on the anchorage system are obtained from the in-structure response spectra acceleration for the prescribed damping value and at the estimated fundamental or dominant frequency of the system or component. A conservative estimate of the spectral acceleration may be taken as the peak of the applicable spectra. This acceleration is then applied to the mass of component or system at its center of gravity. There are four main steps for evaluating the seismic adequacy of equipment anchorage;

1) Anchorage installation inspection;

2) Anchorage capacity determination;

3) Seismic demand determination;

4) Comparison of capacity to demand.

In addition to the inertia effects, there may also be significant secondary stresses induced in systems and components by differential or relative anchor motion if the system or component is supported or restrained at two or more points. For supports, it is common practice to evaluate such seismic induced anchor motion, where the relative or differential motion of the building structure at the different points of attachment should be input to a model of the multiple supported component or system. Resultant forces, moments and stresses in the support system determined from the seismic anchor motion effects acting along with normal loading shall meet the same limits for normal operation plus RLE induced inertia stresses.

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SPECIAL TOPICS AND CASE STUDIES

(Session I-2)

OVERVIEW OF RECENT DEVELOPMENTS IN ATTENUATION MODELS

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Abstract

Attenuation equations predict features of the seismic motion, such as the horizontal and vertical peak ground accelerations (PGA), the peak ground velocities (PGV) and the 5% damped spectral acceleration response (SA), in terms of the earthquake magnitude and distance from source to site. Occasionally other factors, like the type of faulting, are considered in the attenuation expressions. An overview of recent developments in this field is presented in the paper, including a discussion of the applicability of various models for short source to site distances. In such case, i.e. in the neighbourhood of the epicentral region, which is of utmost importance in Nuclear Power Plant applications, the use of two parameters to define the earthquake size is suggested, instead of the single parameter, a magnitude scale. Recent evidence of the importance in such situations of so-called directivity effects, which require a more complete description of the focal mechanism, completes the paper.

1. INTRODUCTION

Seismic hazard assessments can be performed both deterministically, by specifying earthquake scenarios without defining their probability of occurrence, and probabilistically, in which case all seimic events are associated with given probabilities of occurrence. Both approaches require ground motion attenuation models. These are usually based on statistical analyses of recorded ground motions which are necessary to estimate future seismic motions at a given distance from the source of an earthquake of a given magnitude. Thus, these estimates are usually given in the form of equations, called attenuation equations, that predict features of the ground motion in terms of magnitude and distance, and occasionally other variables such as type of faulting. The most commonly mapped parameters of the ground motion are horizontal and vertical peak ground acceleration (PGA), also designated zero period ground acceleration (ZPGA), because it constitutes the ordinate at the origin of the acceleration response spactrum, peak ground velocity (PGV) and 5% damped spectral acceleration response (SA).

It is widely acknowledged that to estimate ground motion it is necessary to define the earthquake magnitude, distance and site conditions, i.e. soil profile at the receiving station. The type of faulting has been recently included in the list of important factors (Abrahamson & Shedlock, 1997) for attenuation relations not restricted to a small specific region. In those approaches, the *size* of the earthquake is defined by its magnitude. Moment magnitude is the preferred magnitude measure, because it is directly related to the seismic moment of the earthquake. However, the use of a single parameter to describe the earthquake *size* or *strength, for engineering purposes* has been questioned (Riera & Doz, 1991). It is noted that the effect of distant earthquakes on Nuclear Power Plants (NPP) is normally irrelevant in the final PSA, while the large contributions to the total risk are due to seismic events associated with sources located at small distances to the NPP site, say less than 20 or 30 Km. In fact, the closer the site is to the epicenter, the less adequate is the magnitude as a *single* earthquake strength parameter. For instance, more than two decades ago, Trifunac (1973) pointed out that the peak acceleration associated with high frequency components of the excitation is very poorly correlated with the magnitude, noting at the same time that, in the neighborhood of the fault, the size of the fracture area looses significance. Since the fracture area A *is* strongly correlated with the magnitude, Riera, Scherer and Nanni (1986) explored the possibility of using A in conjuction with the mean stress-drop $\Delta\sigma$ as measure of the earthquake strength Riera and Doz (1991, 1996) further explore the idea of adopting a two-parameter strength scale

It seems appropriate at this point to call attention to Atkinson & Beresnev's (1997) objections to the use of stress-drops obtained indirectly from certain theoretical models, which may bear no relation to the actual stresses along the fault. It is herein understood that *the stress drop is the difference between the shear stress along the fault surface before and after one given seismic event*, as illustrated in the stick and slip model analized by Doz & Riera (1985). It is also relevant to note that Atkinson & Beresnev (1997), in proposing the use of the difference between the high-frequency and moment magnitudes, which they designate ΔM, in conjunction with the magnitude, implicitly recognize the need for a *two-parameters strength scale.*

Another important factor in the assessment of ground motion at a site are the potential *directivity effects*. These effects have been largely ignored in engineering applications in the past, whether for purposes of design or of reliability analysis, which can be easily explained by the extensive representation of earthquakes as caused by a *point source*, associated to a given magnitude. Of course, there is no orientation of a point (the source) in relation with another point (the site). In addition, directivity effects tend to fade away as the distance to the fault increases. On the other hand, directivity effects *naturally occur* when models such as the sick and slip model are employed, because in such case the fault must be represented by a contact surface. Important results on this issue (Somerville ét al', 1997) are now available and will be briefly described in this paper.

2. ON RECENT ATTENUATION RELATIONSHIPS

Basic data used to derive attenuation relationships as well as models and assumptions employed are widely scattered and frequently unavailable to the engineering community. A recent issue of *Seimological Research Letters (Vol 68, Number 1, Jan/Feb 1997)* was designed to rectify this problem. On account of its global quality and actuality, much of the following material is based on this volume.

It must first be noticed that different source-to-site distance measures are used in the various attenuation relationships available in the literature. A brief summary, adopted from Abrahamson & Shedlock (1997), is given in Fig.1. Moreover, different site classification schemes for local soil conditions are employed in the selection of the data base for the determination of attenuation relations. In this context, the author believes that local geology may be expected to significantly increase the variability of the prediction equations and that therefore the appropriate procedure should be to always derive attenuation equations for hard or sound rock foundation and to obtain the ground motion at the surface of soil deposits by analytical means, using the former as basic input. Consequently, all relations quoted in this paper refer to sound rock outcrops. One restriction to this approach is of course the fact that fewer records on rock may be available, for statistical analysis, than for another soil type of interest. A second restriction is related to applications to sites in which bedrock is found at considerable depths, say more than a few hundred meters. In such case, questions may be raised concerning the determination of surface motions on the basis of rock motion in the free-field.

As an example of attenuation expressions for response spectra in terms of earthquake magnitude, results obtained by the author will first be mentioned. Riera, Scherer & Nanni (1986) presented equations of the form:

$$S_v = S_{vo}(f, M)\ \Phi(f, M, r) \qquad (1)$$
$$S_a = S_{ao}(f, M)\ \Phi(f, M, r) \qquad (2)$$

in which S_v and S_a denote the pseudo-velocity and pseudo-acceleration response spectra, respectively, the same symbols with an added o subscript the corresponding *source* spectra and Φ an attenuation coefficient that describes the decrease in amplitude of the spectra with epicentral distance. The coefficients in empirical equations for the source spectra were determined by nonlinear regression on a data base consisting of 186 accelerograms corresponding to 57 earthquakes,

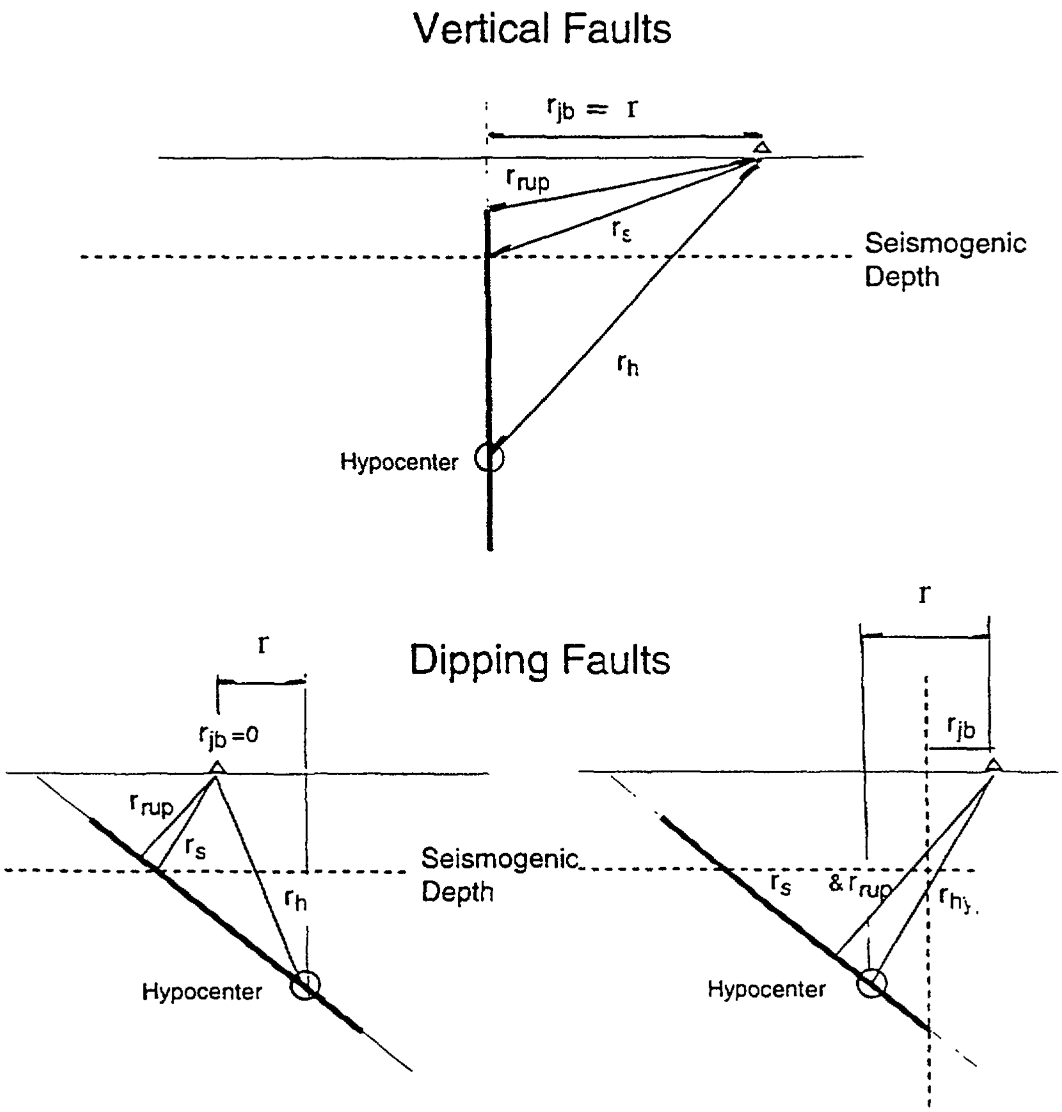


Fig 1 Definitions of distance from earthquake source to site used in different attenuation equations (adapted from Abrahamson and Shedock. 1997)

classified according to soil conditions at the recording station. For rock records, the following expressions resulted:

$$S_{vo} = 0.0253 \exp \{4.5 - 20\, M^{-1} - (0.58 + 6.2\, M^{-1})(1 - \varepsilon^{-0.1\,Mf})\ln 0.125\, f M^{-1}\} \qquad (3)$$

$$S_{ao} = 0.00396\ M^{4.8}\ \{\exp[-(0.52 + 4.9\, M^{-1})\, T] + 494\ T^{2.02} \exp(-10.6\, T)\} \qquad (4)$$

in which the frequency f must be given in Hz, and the period T in seconds. S_{vo} results in m/s and S_{ao} in m/s^2. It may be shown that assuming nonlinear, amplitude proportional damping, the attenuation factor takes the form:

$$\Phi = r^{-1} / (1 + 0.00238\, e^{0.69\,M} \ln r) \qquad (5)$$

where $r > 1$ is given in Km. The preceding equations allow the determination of expected response spectra on rock outcroppings. Note that the equations represent mean values of the spectral velocities or accelerations and should in principle be applied only for epicentral distances larger than the square root of the rupture area.

More recently, Atkinson & Boore (1995, 1997) presented similar assessments of the acceleration spectrum An earthquake source spectrum E (M_o , f) is defined as the Fourier spectrum at a distance of 1 km, from which the desired result can be obtained by multiplying E by an attenuation factor D (r_h , f) and frequency dependant filters, used for instance, to assess response spectra These results are based on a large number of records from eastern North-America Sample values of the acceleration response spectra expected for three moment magnitudes M_o and a wide range of hypocentral distances are given in Table 1

TABLE 1

ENA Median Horizontal Component: Hard Rock Sites

Natural logs of values, in g, are given Abridged version of Appendix of Atkinson and Boore, 1995

		SA (5% damped) for frequency (Hz) =					
Moment M_o	**r_h (Km)**	**1.0**	**2.0**	**3.0**	**5.0**	**10.0**	**PGA**
5 00	10 0	-4 22	-3 01	-2 20	-1 50	-0 77	-0 97
5 00	15 0	-4 68	-3 45	-2 78	-2 12	-1 35	-1 71
5 00	20 0	-5 12	-3 85	-3 16	-2 40	-1 79	-2 17
5 00	30 0	-5 57	-4 33	-3 71	-3 02	-2 42	-2 88
5 00	40 0	-5 96	-4 64	-4 03	-3 49	-2 88	-3 40
5 00	50 0	-6 24	5 04	-4 47	-3 80	-3 23	-3 80
5 00	60 0	6 52	-5 33	-4 70	-4 06	-3 55	-4 18
5 00	80 0	-6 69	-5 53	-4 90	-4 33	-3 85	-4 57
5 00	100 0	-6 69	-5 53	-4 96	-4 33	-3 95	-4 70
5 00	150 0-	-6 86	-5 73	-5 16	-4 63	-4 25	-5 12
5 00	200 0	-7 12	-6 01	-5 51	-4 98	-4 74	-5 65
5 00	300 0	-7 56	-6 57	-6 08	-5 68	-5 65	-6 62
6 00	10 0	-2 73	-1 54	-1 02	0 42	12	-0 33
6 00	15 0	-3 23	-2 04	-1 54	-0 94	-0 44	-0 88
6 00	20 0	-3 56	-2 44	-1 90	-1 30	-0 79	-1 30
6 00	30 0	-4 04	-2 94	-2 40	-1 86	-1 37	-1 91
6 00	40 0	-4 38	-3 28	-2 75	-2 29	-1 79	-2 38
6 00	50 0	-4 64	-3 59	-3 04	-2 60	-2 15	-2 76
6 00	60 0	-4 93	-3 86	-3 35	-2 84	-2 44	-3 09
6 00	80 0	-5 13	-4 05	-3 61	-3 11	-2 75	-3 44
6 00	100 0	-5 10	-4 05	-3 61	-3 16	-2 81	-3 59
6 00	150 0	-5 28	-4 21	-3 81	-3 40	-3 16	-4 00
6 00	200 0	-5 55	-4 58	-4 12	-3 81	-3 60	-4 52
6 00	300 0	-5 93	-5 03	-4 69	-4 41	-4 49	-5 37
7 00	10 0	-1 52	-0 58	-0 09	36	80	32
7 00	15 0	-1 95	-0 96	-0 54	-0 13	31	-0 21
7 00	20 0	-2 28	-1 34	-0 91	-0 47	-0 03	-0 58
7 00	30 0	-2 75	-1 76	-1 36	-0 95	- 52	-1 10
7 00	40 0	-3 02	-2 10	-1 70	-1 30	-0 93	-1 53
7 00	50 0	-3 36	-2 41	-1 93	-1 59	-1 22	-1 89
7 00	60 0	-3 54	-2 62	-2 18	-1 86	-1 49	-2 21
7 00	80 0	-3 76	-2 79	-2 46	-2 10	-1 81	-2 54
7 00	100 0	-3 79	-2 83	-2 53	-2 19	-1 88	-2 67
7 00	150 0	-3 96	-3 06	-2 68	-2 42	-2 22	-3 08
7 00	200 0	-4 19	-3 34	-2 98	-2 72	-2 66	-3 53
7 00	300 0	-4 50	-3 72	- 49	-3 36	-3 49	-4 28

Attenuation equations for Eastern and Central North America were also obtained by Toro, Abrahamson and Schneider (1997), who attempted to quantify all uncertainties involved in the prediction process. The functional form adopted by Toro ét al'(1997) is the following

$$\ln Y = C_1 + C_2 (M - 6) + C_3 (M - 6)^2 - C_4 \ln r_M - (C_5 - C_4) \max [\ln (r_M/100),0] - C_6 r_M + \varepsilon \quad (6)$$

$$r_M = [r_{jb}^2 + C_7^2]^{1/2} \quad (7)$$

in which the spectral acceleration Y is given in g's, C_j (j=1,7) denote regression coefficients, M is either Lg magnitude or moment magnitude M_o and r_{jb} is the Joyner-Boore distance to the earthquake rupture. The total uncertainty ε represents the sum of the statistical and physical uncertainties The regression coefficients for moment magnitude are given in Table 2

Fig 2 shows a comparison of median spectral accelerations for a magnitude 6 earthquake at a JB distance of 20 km Similarly, Fig 3, also adapted from Abrahamson & Shedlock (1997), presents various proposals for the median spectral acceleration in case of a strike-slip earthquake of magnitude 7 0 at a distance of 10 km in an active tectonic region. Upper and lower bounds for the 5% damped response acceleration for the same situation obtained using eqs (4) and (6) are shown in Fig 4 Source response spectra defined by eq. (4) may also be seen in Fig (5)

3.ATTENUATION EQUATIONS FOR TWO-PARAMETER STRENGTH SCALES

As an illustration of the feasibility of using the rupture area and the mean stress-drop for the prediction of earthquake motions, the following equations obtained by the author on the basis of eqs. (1-2) , by combining with well-known relations between earthquake magnitude and various relevant parameters, will be given in this section: It is of course acknowledged that this is not the best approach to obtain attenuation equations, which should be based on direct assessments of the stress-drop and the rupture area, the objective being here to put in evidence the feasibility of using such expressions in engineering applications, and some advantages of the alternative description of earthquake size or strength.

The seismic moment m_o can be related to the rupture area A in a dislocation model by means of the expression (Kanamori & Anderson, 1975):

$$m_o = \mu A D \quad (8)$$

In which μ denotes the shear modulus of the material (Lame's constant) and D the mean displacement. For a circular rupture area it may be shown that:

$$\log m_o = 1.5 \log A + \log (0.41 \Delta\sigma) \quad (9)$$

where Δσ denotes the mean stress drop, in bars , A the rupture area in 10^3 km^2 , m_o being given in dynes-cm.Using Kanamori and Anderson data base, Riera ét al'(1986) obtained semi-empirical equations relating the seimic moment to the area:

$$\log m_o = 22.36 + 1.534 \log A - 0.388 X \quad (10)$$

in which X represents a categorical variable assigned a zero value for inter-plate earthquakes and a value 1 for intra-plate earthquakes Defining as apparent stress the product of the seismic efficiency η by the mean stress $\bar{\sigma}$, a second equation relates the seimic moment to the magnitude and the apparent stress σ_a. Assuming that the expected values of these parameters are statistically different in inter and intra-plate earthquakes, the following equation was also obtained by non-linear regresion (Riera ét al', 1986)

TABLE 2

Coefficients of Toro ét al' (1997) Attenuation Equations

Freq.	Median		Weight=0.046		Weight=0.454		Weight=0.0454		Weight=0.046		Median and all cases				
(Hz)	*C1*	C2	*C1*	*C2*	*C1*	*C2*	*C1*	*C2*	*C1*	*C2*	*C3*	*C4*	*C5*	*C6*	*C7*
Midcontinent, equations using Moment Magnitude															
0.5	-0.74	1.82	-1.53	1.72	-0.99	1.82	-0.49	1.91	0.05	2.00	-0.31	0.92	0.46	0.0017	6.9
1	0.09	1.42	-0.75	1.25	-0.18	1.36	0.35	1.47	0.93	1.58	-0.20	0.90	0.49	0.0023	6.8
2.5	1.07	1.05	0.23	0.89	0.81	1.00	1.34	1.10	1.91	1.21	-0.10	0.93	0.56	0.0033	7.1
5	1.73	0.84	0.89	0.69	1.46	0.79	1.99	0.89	2.57	1.00	0.00	0.98	0.66	0.0042	7.5
10	2.37	0.81	1.53	0.65	2.10	0.76	2.64	0.86	3.21	0.97	0.00	1.10	1.02	0.0040	8.3
25	3.68	0.80	2.84	0.63	3.41	0.74	3.95	0.85	4.52	0.96	0.00	1.46	1.77	0.0013	10.5
35	4.00	0.79	3.16	0.63	3.74	0.74	4.27	0.85	4.84	0.96	0.00	1.57	1.83	0.0008	11.1
PGA	2.20	0.81	1.36	0.64	1.93	0.75	2.46	0.86	3.04	0.97	0.00	1.27	1.16	0.0021	9.3
Gulf, equations using Moment Magnitude															
0.5	-0.81	1.72	-1.6	1.58	-1.06	1.67	-0.56	1.76	-0.02	1.86	-0.26	0.74	0.71	0.0025	6.6
1	0.24	1.31	-0.6	1.15	-0.03	1.26	0.51	1.36	1.08	1.48	-0.15	0.79	0.82	0.0034	7.2
2.5	1.64	1.06	0.80	0.90	1.38	1.01	1.91	1.12	2.48	1.23	-0.08	0.99	1.27	0.0036	8.9
5	3.10	0.92	2.26	0.76	2.83	0.87	3.36	0.97	3.94	1.08	0.00	1.34	1.95	0.0017	11.4
10	5.08	1.00	4.25	0.84	4.82	0.95	5.35	1.05	5.92	1.16	0.00	1.87	2.52	0.002	14.1
25	5.19	0.91	4.35	0.74	4.92	0.86	5.46	0.96	6.03	1.07	0.00	1.96	1.96	0.0004	12.9
35	4.81	0.91	3.97	0.74	4.54	0.86	5.08	0.96	5.65	1.07	0.00	1.89	1.8	0.0008	11.9
PGA	2.91	0.92	2.07	0.75	2.64	0.86	3.18	0.97	3.75	1.08	0.00	1.49	1.61	0.0014	10.9

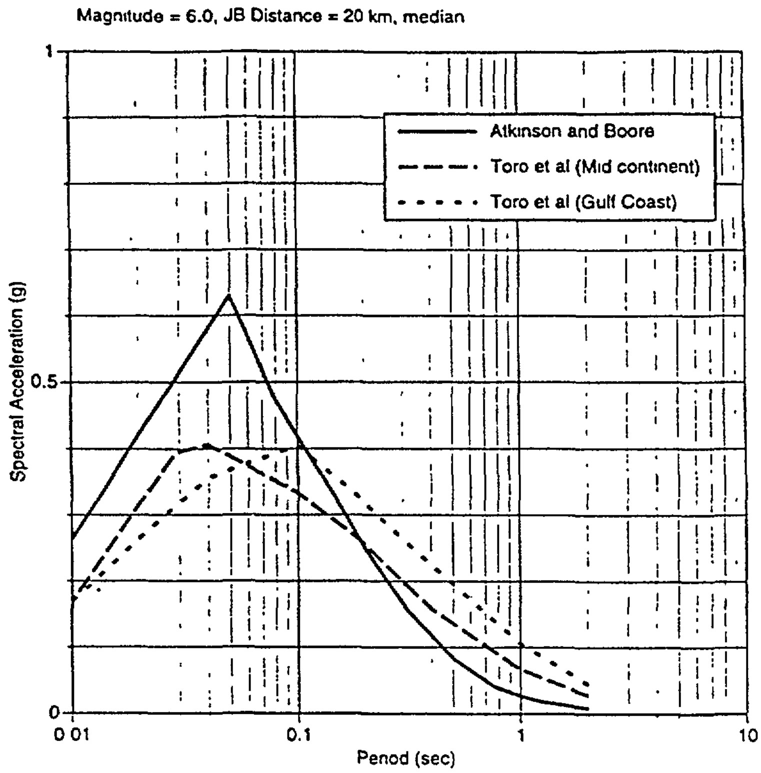


Fig. 2 Comparison of stable continental region median spectral accelerations for a magnitude 6 earthquake at a JB distance of 20 km (From Abrahamson & and Shedlock, 1997)

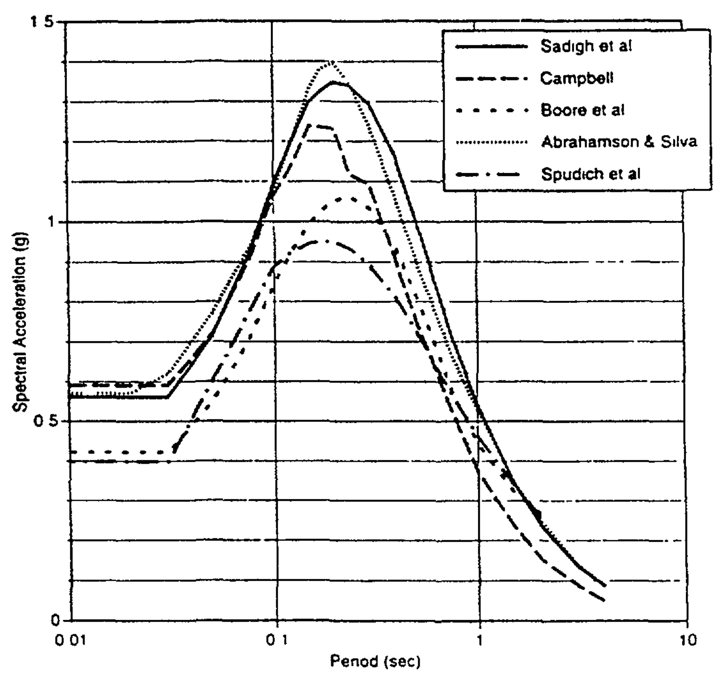


Fig 3 Comparison of the median spectral acceleration for a strike-slip earthquake of magnitude 7 0 at a distance of 10 km in an active tectonic region (From Abrahamson & Shedlock, 1997)

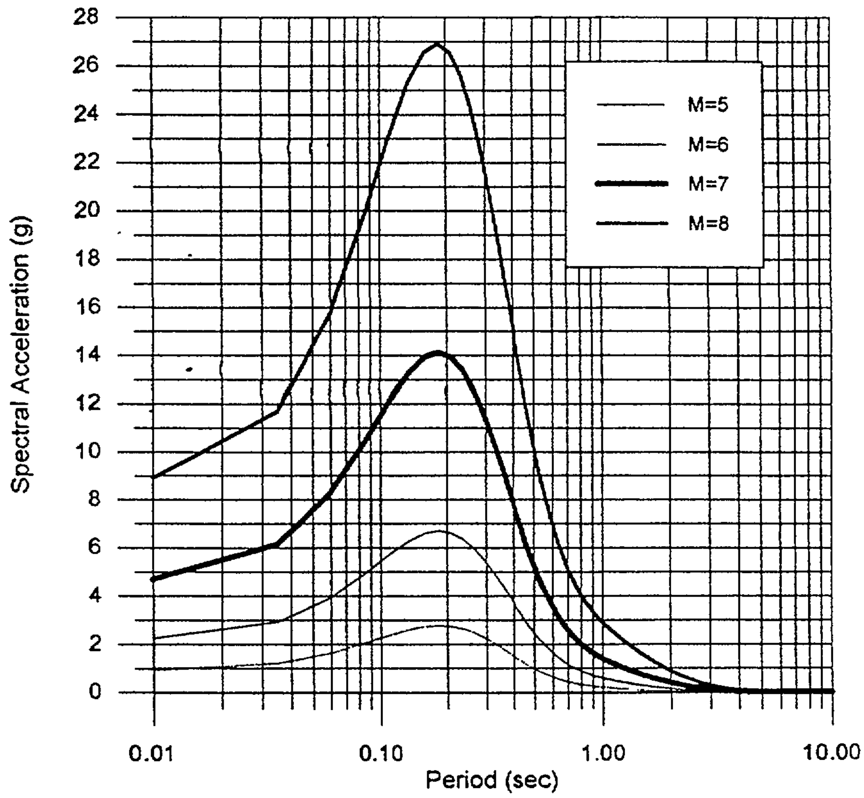


Fig 4 Source response spectra for magnitudes 5 to 8, aqcoording to Riera ét al`(1986)

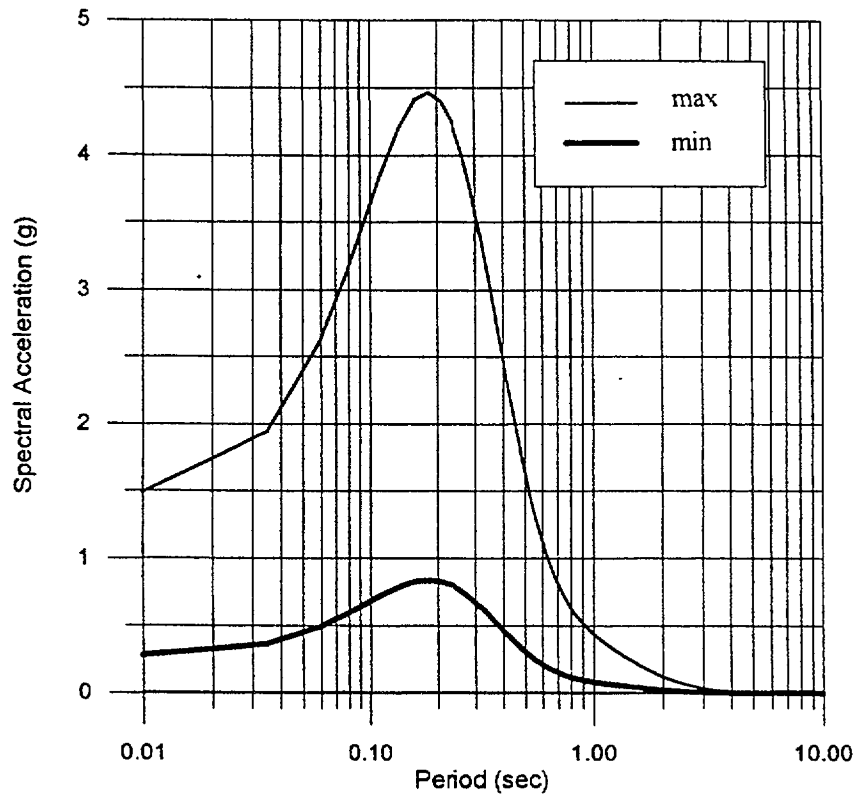


Fig 5 Upper and lower bounds for the acceleration response spectrum of a magnitude 7 0 earthquake at a distance of 10 km, according to Riera 'et al' (1986)

$$\log m_o = 15.51 + 1.53\ M + 0.483\ X \tag{11}$$

Finally, the magnitude and the rupture area are related by:

$$M = 7.455 + 0.977 \log A - 0.377\ X - 0.268\ X \log A \tag{12}$$

Thus, substituting eq.(12) in eqs. (30 and (4), permits deriving attenuation equations in terms of the rupture area for inter and intra-plate earthquakes, valid at epicentral distances larger than $A^{1/2}$. Some of these expressions are given below.

* for intra-plate earthquakes (mean $\Delta\sigma$ = 100 bars)

$$S_{ao} = 59.93\ A^{0.34} (\ e^{-1.17T}\ A^{0.098\ T} + 494\ T^{2.02}\ e^{-10\ 6\ T}) \tag{13}$$

$$\Phi = r^{[0.5 \exp(-1.1 f) - 1]}\ /\ (\ 1 + 0.408\ A^{0.29} \ln r \tag{14}$$

*for inter-plate earthquakes (mean $\Delta\sigma$ = 100 bars)

$$S_{ao} = 44.32\ A^{0.25}\ (\ e^{-1.22T} A^{0.071\ T} + 494\ T^{2.02}\ e^{-10.6\ T}\ ) \tag{15}$$

$$\Phi = r^{[05\ \exp(-1.1f)-1]}\ /\ [\ 1 + 0.314\ A^{0.21}\ \ \ln r\ ] \tag{16}$$

with the limitation r>5 km. S_{ao} represents the source acceleration spectrum and ϕ is an attenuation function that describes the decay rate of the spectral amplitude with distance from the source. T denotes the spectrum period (s) and f= 1/T the frequency (Hz). When energy dissipation due to hysteric damping or internal friction are not considered, the attenuation function can be expressed as:

$$\Phi = r^{[0.5\exp(-1.1f)-1]} \tag{17}$$

which, for high frequencies (f>5Hz) approaches the decay rate for body waves (r^{-1}). It is also well-known that the attenuation law for peak acceleration in the near-field is not similar to that in the far-field. In the near field, the spectral amplitudes depend fundamentally on the stress-drop $\Delta\sigma$. It has been suggested by Papageorgiou and Aki (1985) that there is a linear relationship between peak ground acceleration and stress-drop, i.e.:

$$ZPGA = 0.01\ \Delta\sigma \tag{18}$$

with ZPGA in g's and $\Delta\sigma$ bars. The linear relation (6) is a direct consequence of the hypotheses of material linearty. The proportionality constant was proposed by Riera and Doz (1991).

It is important to note that the parameters of equations (1) and (3), which characterize intra- or interplate earthquakes, depend on the stress-drop. The expected values calculed by Kanamori & Anderson (1975) indicate that $\Delta\sigma$ approches 100 bars in intra- and 60 bars assigned inter and interns-plate earthquakes. Since the differences between expected values of the stress-drop in intra-plate earthquakes was found by Riera 'et al'(1986) to be statistically significant, different prediction equations result for each type of earthquake. It may be more appropriate to select the attenuation equation in terms of the inferred or predicted mean stress drop, rather than on the fact of the earthquake be classified as intra- or inter-plate, the former being applicable for $\Delta\sigma$ > 100 bars.

Taking into account the equations just defined, particularized for T= 0, it is possible to calculate the peak acceleration in rock, resulting, for intra-plate earthquakes

$$(ZPGA)_0 = 59.93\, A^{0\,34}\, r^{-1} / [\, 1 + \ 0.408\, A^{0\,29} \ln r\,] \qquad (19)$$

and, for inter-plate earthquakes

$$(ZPGA)_0 = 44\,32\, A^{0\,25}\, r^{-1} / [\, 1 + 0\,314\, A^{0\,21} \ln r\,] \qquad (20)$$

with A in 10^3 km^2 and r in km, $(ZPGA)_0$ results in m/s^2. Taking into account that when $r \rightarrow 0$ equation (18) should substitute equations (19) or (20), Riera & Doz (1991) suggest a combination of these expressions in a law valid in the whole field·

$$1/a_{max} = 1 / C_1 \ + \ 1/ (C_1 + C_2\,) \qquad (21)$$

where C_1 represents the lower value and C_2 the higher value between ZPGA and $(ZPGA)_0$ If the assumption represented by eq. (18) is extented to the entire spectrum frequency range, then eq (21) may also be used to generate response acceleration and velocity spectra

As an example of the approach, the equations given in this section will be applied to the recent Great Hanshin earthquake (1995), whose magnitude was estimated as 7.2, with a mean stress drop larger than 100 bars Then from eq. 12 it may be inferred that A = 1500 km^2 . This area is compatible with an estimate based on the distribution of slip, according to Shibata (1995), from which a slightly smaller area results. Using eqs. 18, 19 and 21, the attenuation curve for $\Delta\sigma$ = 100 bars shown in Fig 6 is obtained. For purposes of comparison, the curves for mean stress drops 50 % above and below that value and predictions based on Joyner and Boore are also indicated

4 DIRECTIVITY EFFECTS

It has been repeatedly mentioned that models that imply that the earthquake induce vibrations radiate from a point source should lead to very poor predictions of ground motion at sites in the epicentral region, that is, close to the zone of energy release. Within this region the location of the site of interest in relation to the fault plane, the rupture area, the location of the hypocenter and the velocity and direction of motion of the rupture front become important factors In the immediate vecinity of the causative fault surface, the stress drop becomes a dominant factor, as discussed above

A comprehensive discussion of directivity effects in connection with attenuation equations is due to Somerville ét al'(1997) *Forward* directivity effects occur when two conditions are met the rupture front propagates towards the site, and the direction of slip is aligned with the site These conditions are frequently met in strike-slip faulting.In such case almost all the enegy radiated from the fault arrives in a single large pulse of motion Conversely, *backward* directivity effects take place when the rupture front moves *away* from the site, giving rise to the opposite effect, long duration motions having low amplitudes at long periods Directivity effects can be clearly seen in records of the 1971 San Fernando earthquake, as well as in the 1994 Northridge earthquake Fig 7, reproduced from Somerville ét al'(1997), dramatically illustrates the phenomenon in the 7.3 Landers earthquake of 1992, through the Lucerne and Joshua Tree records The information is complemented by Fig 8, from the same reference, in which a comparison between the strike normal and strike parallel responses in the forward region is presented

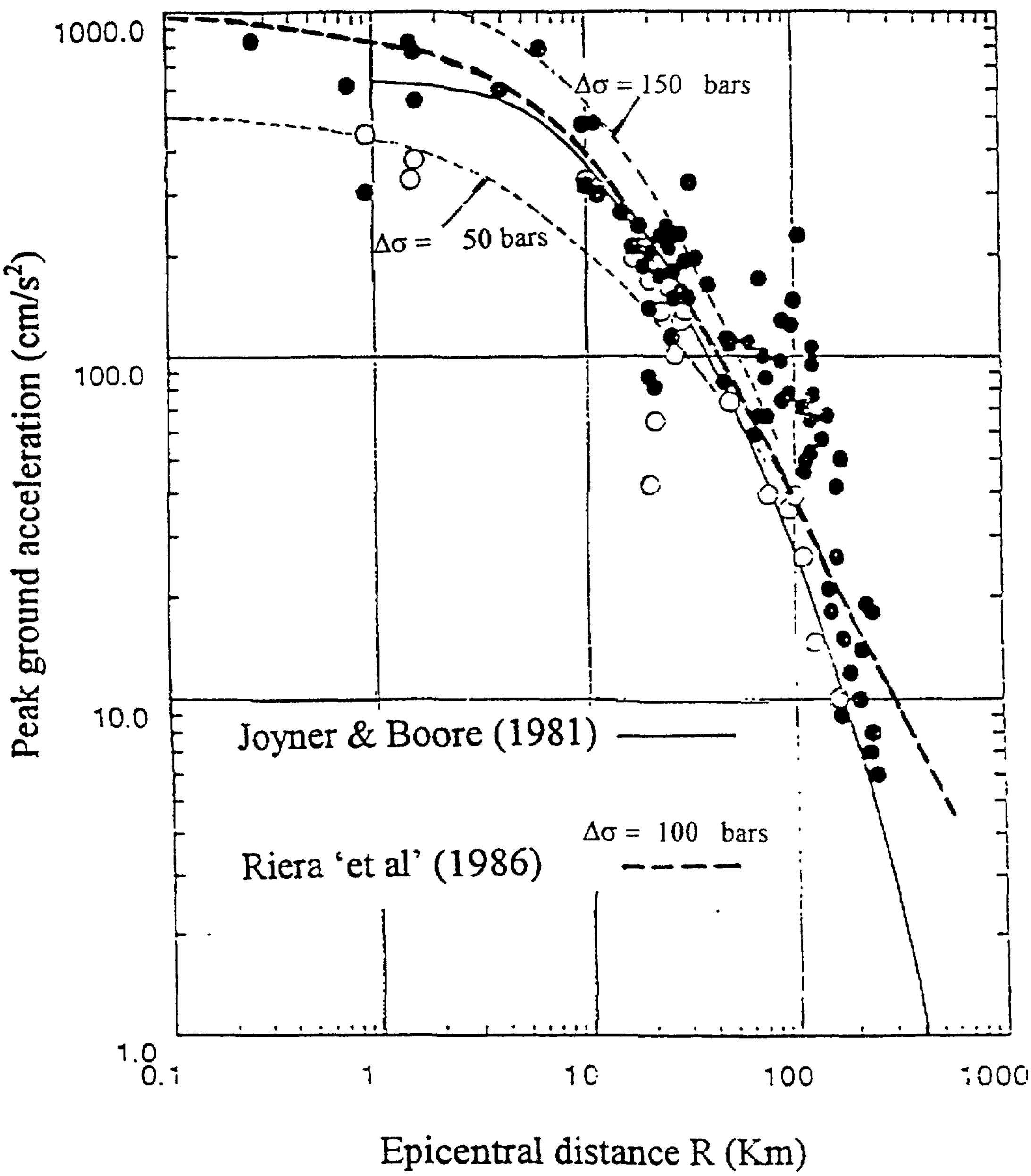


Fig 6. Attenuation of peak ground acceleration for Great Hanshin Earthquake of 1995 (Measured values furnished by Shibata, 1995)

In order to obtain criteria useful in engineering applications, Sommerville ét al'(1997) introduced the rupture directivity parameters θ and X for strike-slip faulting, and ϕ and Y for dip-slip faults, with the meaning shown in Fig. 9. By processing data from 21 earthquakes from North America, Europe and Asia, those authors arrive at the frequency dependent coefficients for modifying the acceleration response spectra shown in Fig 10.It may be seen that the response may be drastically altered for periods above 0.7 sec. Thus, the issue should be of special concern in presence of medium or soft soil layers at the site.

The preceding results constitute an additional argument in favor of seimic hazard studies based on a more detailed description of the earthquake source, rather than simply an assumed epicenter and magnitude, from which all ensuing effects must be inferred.

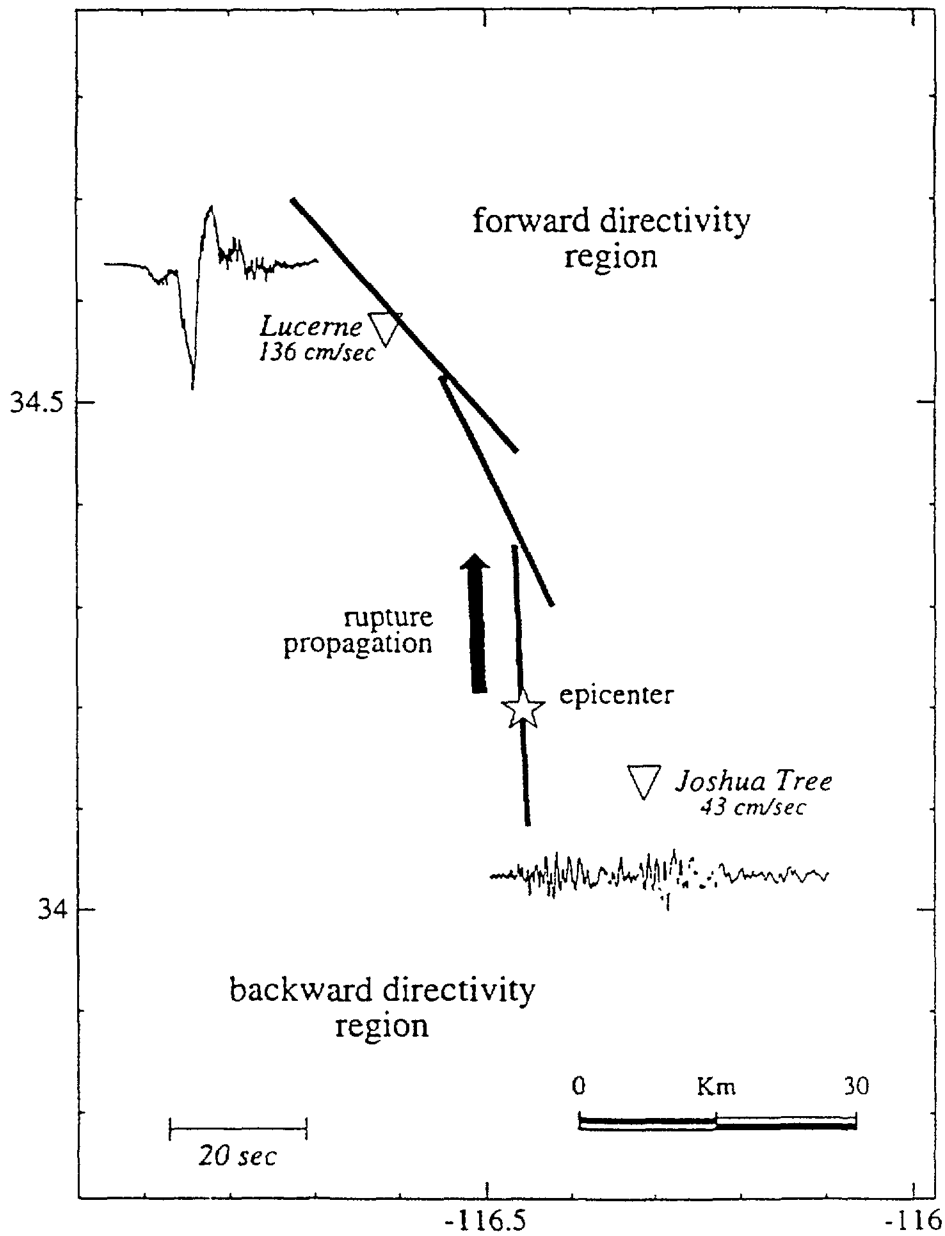


Fig 7 Map of the Landers region showing the main features of the 1992 Landers earthquake, which occurred on three fault segments. the epicenter and the location of the recording stations at Lucerne and Joshua tree (From Somerville ét al', 1997.)

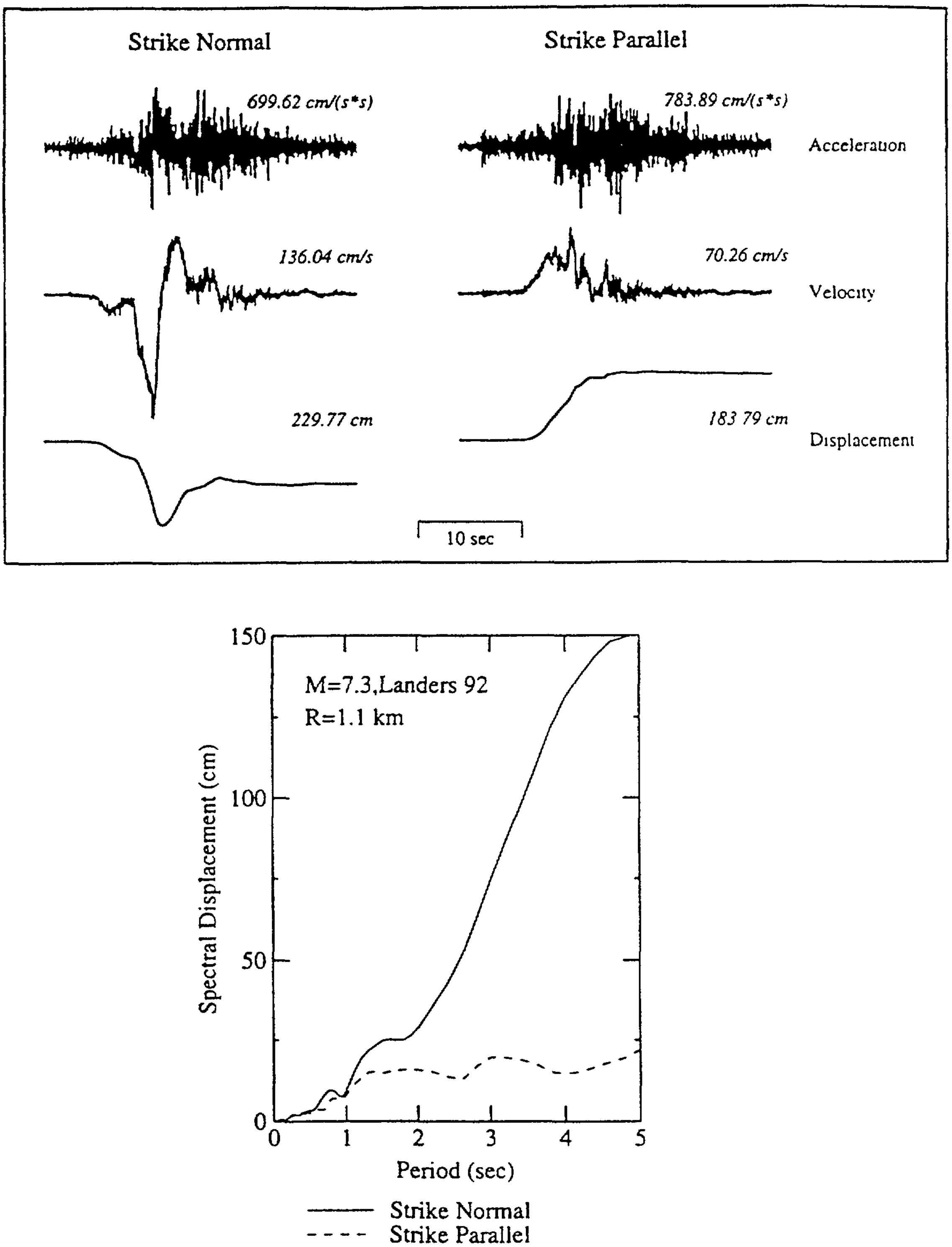


Fig 8 Directivity effects during 1992 Landers earthquake top records at stations in forward and backward directivity regions, bottom strike-normal and strike-parallel spectral displacements at Lucerne (From Somerville 'et al', 1997)

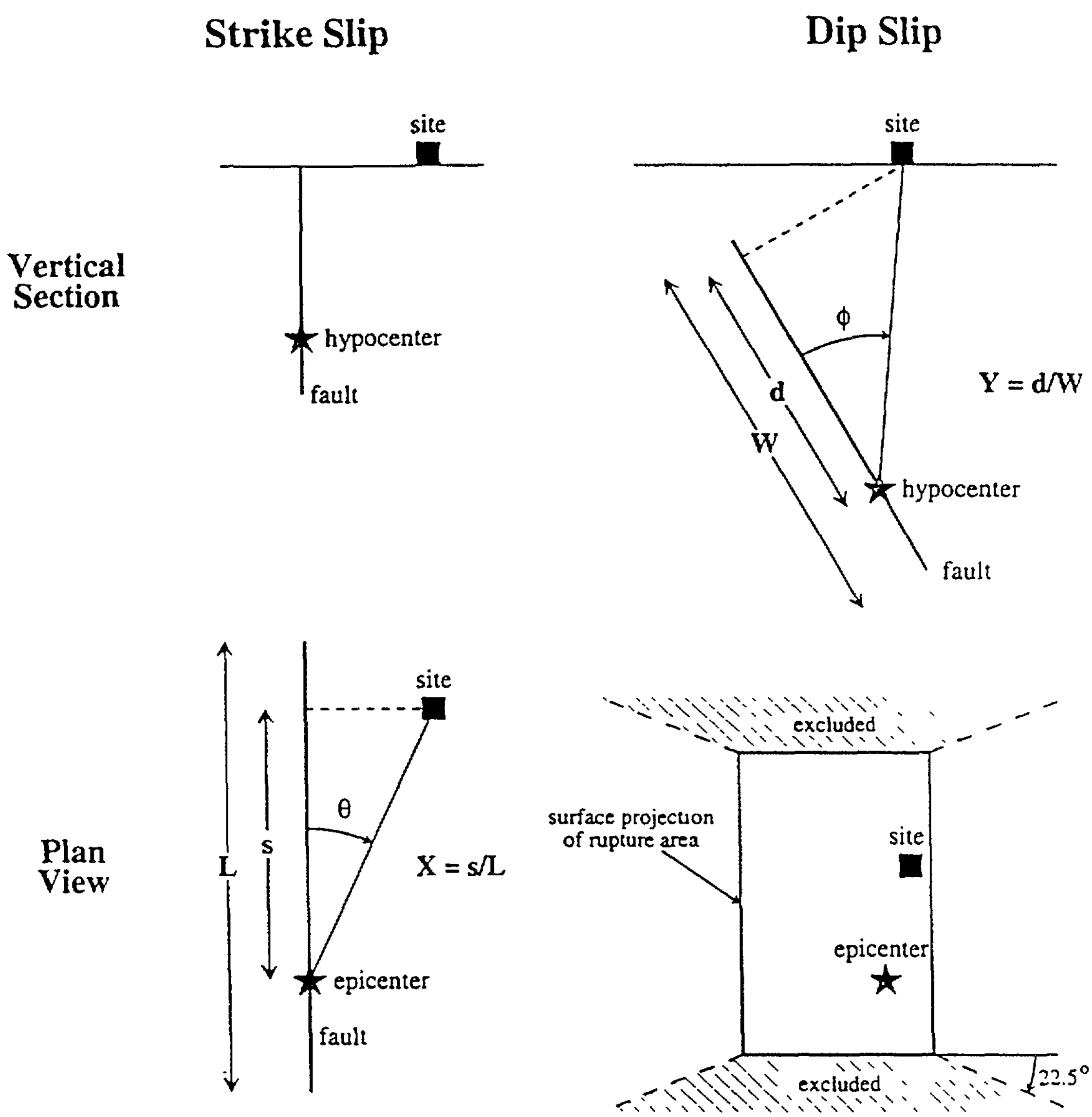
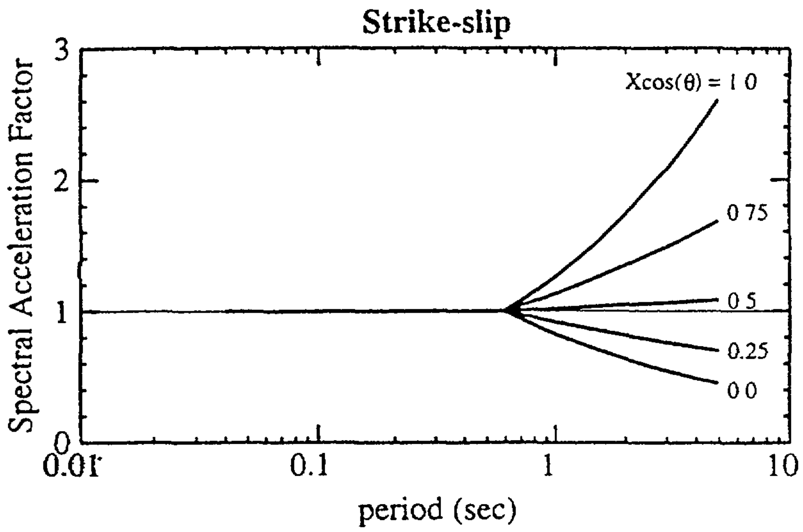


Fig 9 Definitions of rupture directivity parameters (Somerville 'et al', 1997)



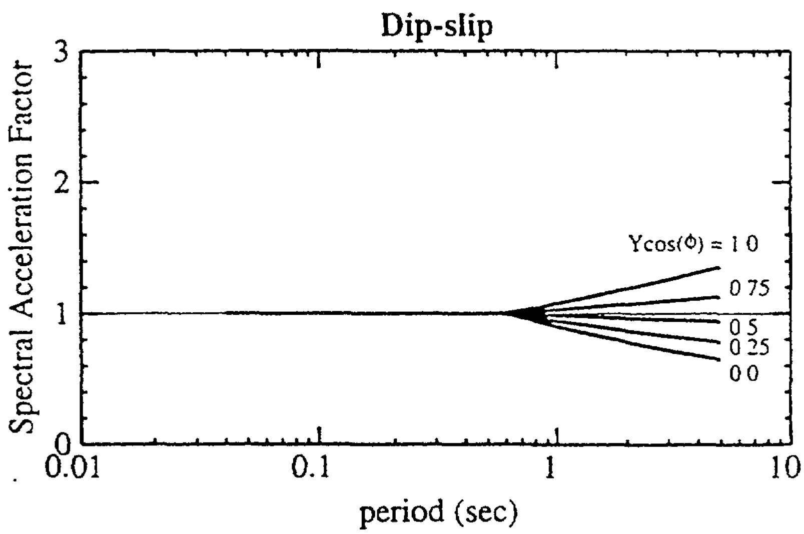


Fig 10 Empirical model of the response spectrum amplitude ratio, showing its dependence on frequency and on the directivity function , according to Somerville 'et al' (1997)

5. CONCLUSIONS

Several aspects of seismic hazard assessments of NPP connected to the use of attenuation relations were discussed. In addition to a brief overview of attenuation expressions, the feasibilty of using a two-paremeter scale to define the earthquake strength is discussed. Such a scale seems to be of paramount importance when ground motion predictions are needed in or close to the epicentral area. In this region, directivity effects may significantly influence the seismic motions, as discussed in the last section of the paper.

ACKNOWLEDGEMENTS

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SEISMIC ASSESSMENT OF A SITE USING THE TIME SERIES METHOD

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Abstract

1. To increase the safety of a **NPP** located on a seismic site, the seismic acceleration level to which the **NPP** should be qualified must be as representative as possible for that site, with a conservative degree of safety but not too exaggerated.

2. The consideration of the seismic events affecting the site as independent events and the use of statistic methods to define some safety levels with very low annual occurrence probabilities (**10^{-4}**) may lead to some exaggerations of the seismic safety level.

3. The use of some very high values for the seismic accelerations imposed by the seismic safety levels required by the hazard analysis, may lead to very expensive technical solutions that can make the plant operation more difficult and increase the maintenance costs.

4. The consideration of seismic events as a **time series** with dependence among the events produced, may lead to a more representative assessment of a **NPP** site seismic activity and consequently to a prognosis on the seismic level values to which the **NPP** would be ensured throughout its life-span. That prognosis should consider the actual seismic activity (including small earthquakes in real time) of the focuses that affect the plant site.

The method is useful for two purposes:

a) research, i.e. homogenizing the history data basis by the generation of earthquakes during periods lacking information and correlation of the information with the existing information. The aim is to perform the hazard analysis using a homogeneous data set in order to determine the seismic design data for a site;

b) operation, i.e. the performance of a prognosis on the seismic activity on a certain site and consideration of preventive measures to minimize the possible effects of an earthquake.

5. The paper proposes the application of **Autoregressive Time Series** to issue a prognosis on the seismic activity of a focus and presents the analysis on **Vrancea** focus that affects **Cernavoda NPP** site, by this method.

6. The paper also presents the manner to analyze the focus activity as per the new approach and it assesses the maximum seismic acceleration that may affect **Cernavoda NPP** throughout its life-span (~ **30 years**).

7. Development and application of new mathematical analysis method, both for long - and short - time intervals, may lead to important contributions in the process of prognosis the seismic events in the future.

1. INTRODUCTION

Earthquakes are very violent phenomena which affect people life as well as the building safety. By now, deterministic correlation regarding the moment of their occurrence and their violence has not been assessed and that is the reason why its analysis was made by statistic methods.

The statistic approach is imposed by the fact that the seismic history of a focus has presented a relatively small number of accurate determined events. Historical information are not continuous and the moment of an earthquake occurrence and especially its violence, evidence a high degree of uncertainty. For that reasons, by now, the seismic activity of a focus has been approximated by **Poisson** type models in which events, considered independent, are the annual maximum magnitudes or for certain time-interval.

If more possible alternatives for the parameters of a focus are considered, e.q. the maximum possible magnitude, focuş depth, epicentrum distance, etc. and certain levels of confidence are associated to them, one can determine the effect of that focus on a site, by the determination of hazard curves. Based on these earthquakes it is possible to determine the maximum acceleration on site, considering all the possible alternatives and their percentage of confidence.

Consideration of earthquake generation in a certain focus as completely independent elements may be quite a wrong approximation which, usually leads to overestimation.

The approach of a focus activity by means of time-series in which the events are supposed to be dependent on one another and their occurrence is generated by deterministic causes, to which aleatory causes are overlapping, is more realistic, we think.

The main problem today is whether the existing data are sufficient to assess the deterministic component and make possible a correct assessment of the model parameters both for deterministic conponent and for statistic ones.

This paper is an analysis of **Vrancea** seismic focus (the main focus which affects **Cernavoda NPP** site) applying the method of Auto-Regressive (**AR**) time-series.

The paper is aimed to evidence the possibilities of analyzing a focus by means of **AR** models. It presents several different approaches and points out the existence of an overlapping of periodical events components ranging between **2** years and **46** years, events which might be correlated to some geological phenomena regarding the earth thermodynamics and the plate tectonics or to some phenomena related to the mechanics of planets like earth tide.

Due to a lack of representative series of the input data, the paper presents only few different hypothesis which, to a certain extent, may alter the results and for that reason the analysis is considered preliminary and it should be remade by reviewing the representative package of input data.

2. SEISMIC HAZARD CURVES AT CERNAVODA NPP

Cernavoda NPP site seismicity is determined by **Vrancea** intermediate focus whose depth ranges between **90 - 150Km**, and is located at **190 Km** epicentrum distance to the **Cernavoda NPP** site evidencing a maximum credible magnitude of **7.5**, according to some authors, and **7.8** as per others.

Cernavoda NPP site is also affected by **Sabla-Dulovo**, **Galati-Tulcea** seismic area and the smaller amplitude local **Vrancea** earthquakes (see **Figure 2.1**).

To determine the seismic hazard curves on the site, **Poisson** type process which represents the probability of occurrence of at least one earthquake having the magnitude higher than **M** value, was considered [**Ref. 1**, 7].

That probability is given by the relation:

$$p(M,t) = 1 - e^{-\nu(M)t} \qquad (2.1)$$

where, $\nu(M)$ is the average annual number of earthquakes having the magnitude grater then **M**, given by the magnitude - frequency recurrence law. For **Vrancea** intermediate focus, the non-corrected magnitude - frequency recurrence law for the maximum credible magnitude is:

$$LgN(m \geq M) = 716.3 - 626.4M + 218.4\,M^2 - 38.0\,M^3 + 3.3\,M^4 - 0.1\,M^5 \qquad (2.2)$$

Figure 2.1 The seismic zones which affect Cernavoda NPP Site

The law of seismic acceleration attenuation with epicentrum distance was determined by processing the recordings made since **1977** till now and it is given by equation (2.3) for **Vrancea** intermediate focus [**Ref. 2**]:

$$Acc = e^{4.16765+1.11724M}(R_h+40)^{-1.44067}; \qquad \sigma_{\ln(Acc)} = 0.47607 \tag{2.3}$$

In case of **Sabla-Dulovo** and **Galati - Tulcea** fault, similar analyses have been done and their intermediate results are not presented herewith [**Ref. 2**].

The **Figures 2.2-2.3** present the magnitude frequency law as well as the seismic hazard curves for **Cernavoda NPP** site for medium value of **120 Km** hypocentrum depth and epicentrum distance **190 Km** using the data base of **Ref. 6**.

Analyzing the results obtained we can say that:

1. the seismic zone which determines the seismic risk for **Cernavoda NPP** is ***Vrancea*** zone. The predominant influence of intermediate **Vrancea** earthquakes in the assessment of the seismic hazard on **Cernavoda NPP** site is due to *the high frequency of earthquakes occurrence and to the high maximum magnitudes as to the other seismic zone.*

2. The value of peak ground acceleration for an annual exceeding probability of 10^{-3}, corresponding to the **DBE** design acceleration for **Cernavoda NPP** site, is **0.175 g** from *the median curve attenuation + one standard deviation* that is lower than **0.2 g** as considered in the seismic qualification of **Cernavoda NPP** Unit 1.

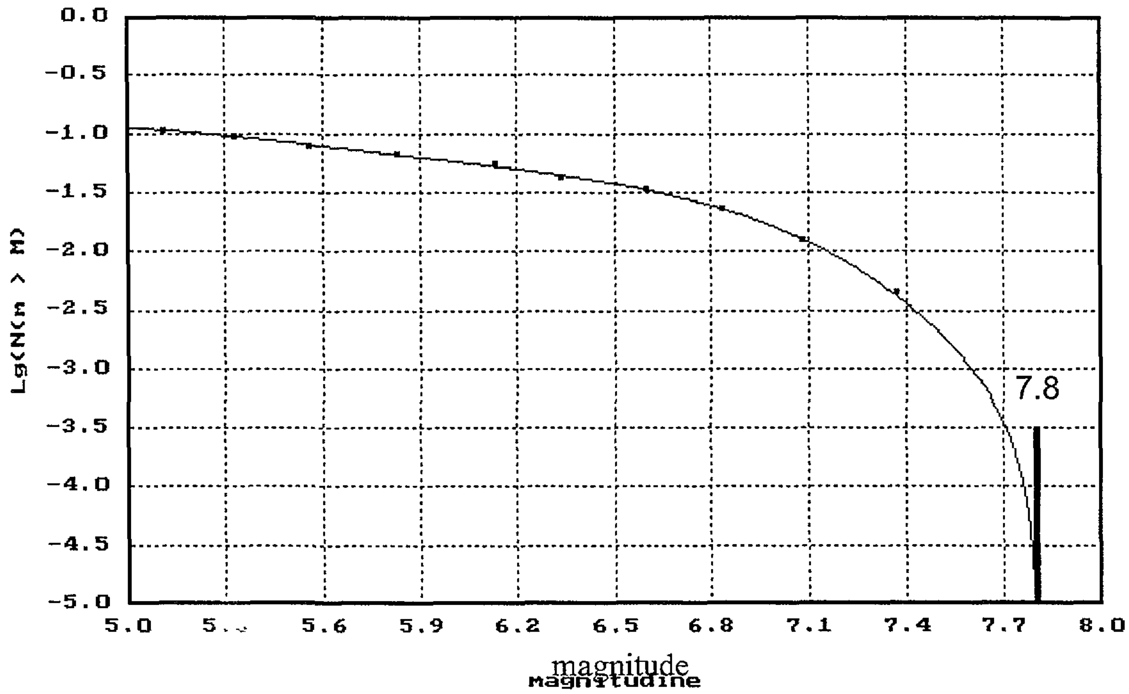

Figure 2.2 Law of the Cumulative Frequency-Magnitude for the 7.8 credible earthquake magnitude

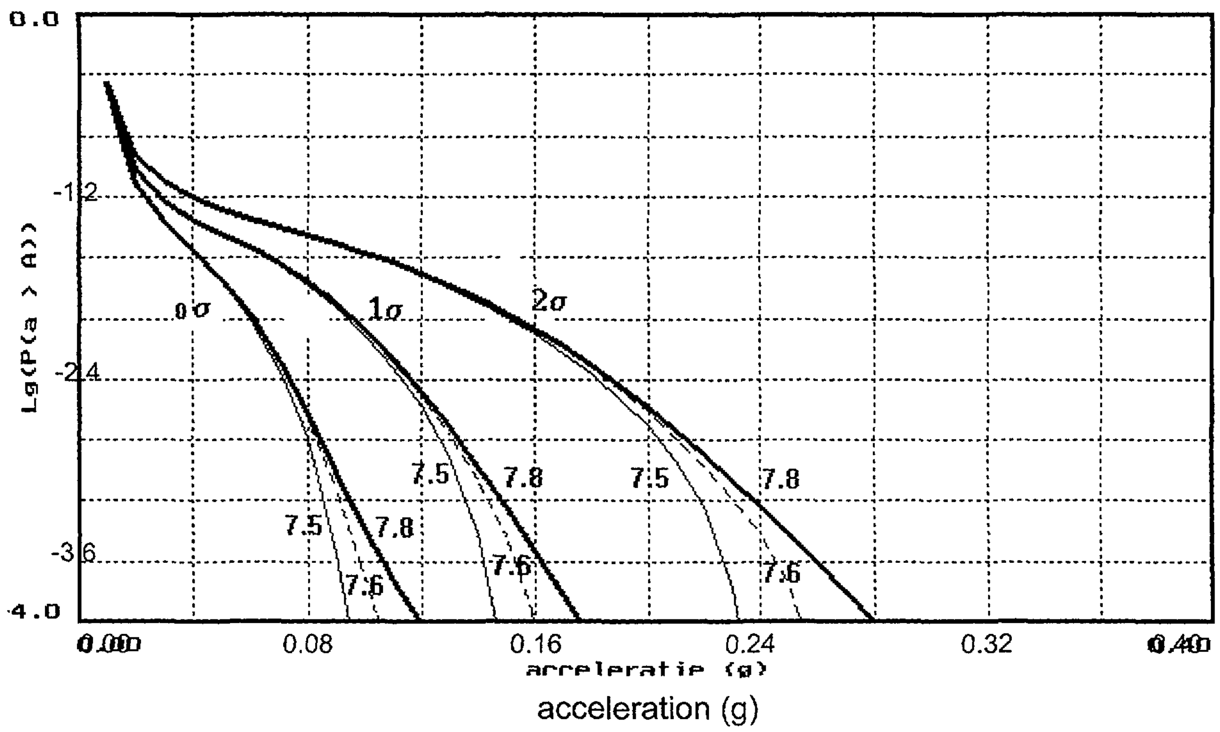

Figure 2.3 Seismic Hazard Curves for Cernavoda NPP Site for medium value: 120 Km depth and 190 Km epicentrum distance

3. ANALYSIS OF VRANCEA FOCUS ACTIVITY APPLYING THE AUTOREGRESSIVE TIME SERIES

Analyzing the seismic history of **Vrancea** focus, for the period **984-1900** it was found that there were large time-intervals in which no historical information were available. The largest time-interval corves **120** years **(1327-1446)** and makes the time series non - homogenous and thus no analysis was possible for that period in the first stage [**Ref. 6**].

The existing data, starting with the year **1900** by now, are quite homogenous and they can be applied in the analysis for that period.

Based on Auto-Regressive method, the analysis of the focus activity includes the following steps:

I. Determination and elimination from the time-series of the mean and all periodical components;

II. Determination of AR model parameters;

III Selection of AR model;

IV Prediction of events;

Here below there is a brief description of each step above.

I. Determination and elimination from time series of the mean and periodic components

The mean component of the time series is determined as an arithmetic mean of the time series and an elimination of the arithmetic mean is made for each element of the time series.

Determination of the all periodic components of the remain series, both as periods and values, is a very important stage and that is why several determination methods are applied.

a) *Determination of the period components by means of auto-correlation function*

In order to point out the periods, the auto correlation function was applied both to the initial series and to the resulted function until the periodic components became evident.

After **5** sequential applications, the component was obtained as per the **Figure 3.1** where two components are evidenced: the **2** years and the **13** years component.

b) *Determination of the periodic component using **Fourier** analysis*

Fourier analysis is a method to determine the periodic character of a time series by the detection of the periodic components.

By the application of **Fourier** transform, the existence of some components became quite obvious: **2**, **31** and **46** years. The periodic components with period grater than about 10 - 20 years (for a time-series of **93** records) are affected by computational errors and should be re-confirmed by other methods.

c) *Determination of the periodic components using a numeric method*

The numeric method determines the periods of components by the arrangement, as a table, of the time series as well as by the creation of a sub-series of constant lengths, subseries resulted

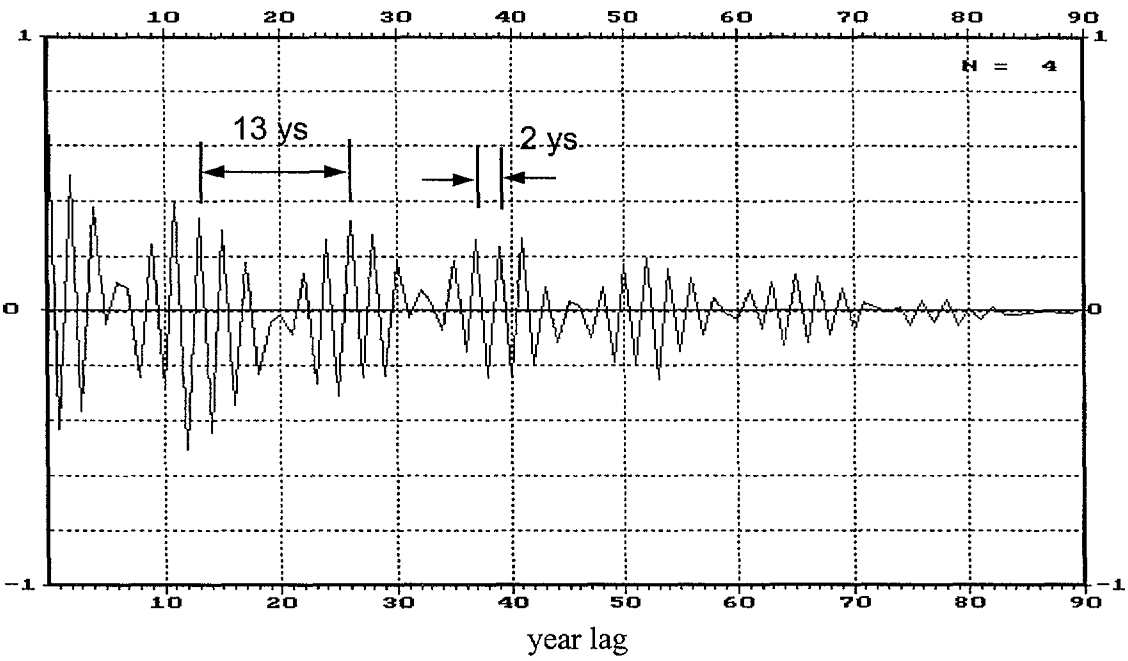

Fig 3.1 Auto-correlation function applied to the time series

from the division of the initial series by a number encompassed between **1** and the series length and the numeric processing of the time-series so obtained [**Ref.** 3].

The following periodic components were evidenced: **13, 27, 31, 41, 43** and **45** years. Large periodic components may have errors because the applied time-series has a relatively small number of events.

In case of a time series with **93** records (the case herewith), with components period larger then **30** years, there are sensible errors dependent on the increase of the detected period.

II. Determination of AR model parameters

The time-series remained after the elimination of the time-series mean and the periodic components are analyzed with **AR** model as follows.

In these models, a value **y** (earthquake magnitude value) at time **t** is produced as the sum of a linear regression on a finite number of previous values and an aleator residual component. [**Ref. 3, 4**].

If the regression is limited to **k** terms, then the equation:

$$y_t = \sum_{i=1}^{k} a_i y_{t-i} + \varepsilon_t \tag{3.1}$$

defines the so-called **Markov** model of order **k**. The $\mathbf{a}_i$ are autoregressive coefficients, and the residual $_t$, is an independent random variable uncorrelated with the $\mathbf{y}_{t-i}$ valus for **i = 1, 2,...,k**.

For a first order scheme:

$$y_t = a_1 y_{t-1} + \varepsilon_t \tag{3.2}$$

and $\mathbf{a}_1$ is given by the first auto-correlation coefficient, $\mathbf{r}_1$, of the stationary series $\mathbf{y}_t$. For the second order scheme:

$$y_t = a_1 y_{t-1} + a_2 y_{t-2} + \varepsilon_t \tag{3.3}$$

and $\mathbf{a}_1$, $\mathbf{a}_2$ are given by:

$$a_1 = \frac{r_1(1 - r_2)}{(1 - r_1^2)}; a_2 = \frac{(r_2 - r_1^2)}{(1 - r_1^2)} \tag{3.4}$$

where $\mathbf{r}_1$, $\mathbf{r}_2$ are the first and second-order auto-correlation coefficients of the stationary series $\mathbf{y}_t$. The residuals $_t$ are found from:

$$\varepsilon_t = y_t - a_1 y_{t-1} \tag{3.5}$$

for the first-order scheme, and:

$$\varepsilon_t = y_t - a_1 y_{t-1} - a_2 y_{t-2} \tag{3.6}$$

for the second-order scheme.

III. Selection of AR model

An important problem of fitting a parametric model to a time series is how to choose the best order of approximation. For purely **AR** models it can be solved rather easily in most cases by applying the criteria [**Ref.** 4]:

$$AIC(k) = n \cdot \log \lambda^2 (k) + 2k \tag{4}$$

for **k = 0, 1,...,k$_m$**,

where: **k** - the order of the current approximating model;

k$_m$ - maximum order which should be specified in advance;

n - the length of time-series;

^{2}**(k)** - the estimate of $_t$ for the current model of order **k**.

The optimal order is one for which **AIC(k)** attains its minimal value.

IV. Prediction of events

In the alternatives subjected to analysis, the prediction on the seismic activity is performed using the average component, determined by the application of AR model [**Ref. 4**], to which the periodic components and the time-series mean are added. To those values, we can add a generated gaussian aleatory value of mean zero and the dispersion determined from the remained time - series (see paragraph II).

4. RESULTS

By the application of the above presented method, the seismic activity of **Vrancea** focus for a time-series encompassing the time-interval **1901- 1993** was analyzed under the following hypotheses:

- for the years in which data were not available, an earthquake having the magnitude equal to the minimum detectable value throughout the period, namely value **4**, was considered in the analyses;

- for the years in which more earthquakes existed, the earthquakes were considered equivalent to an earthquake which released an amount of energy equal to the sum of energies generated in that respective year.

Figures 4.1-4.2 show an unidimensional case, in which the annual maximum magnitude represents the time-series.

Moreover, a bidimensional case is presented herewith, a case in which the variables of time-series signify the time-interval between two subsequent earthquakes, namely, the magnitude of the earthquake occurred after each of these time-intervals. It is a case which, theoretically eliminates some lack of information in the initial data (**Figures 4.3 - 4.4**).

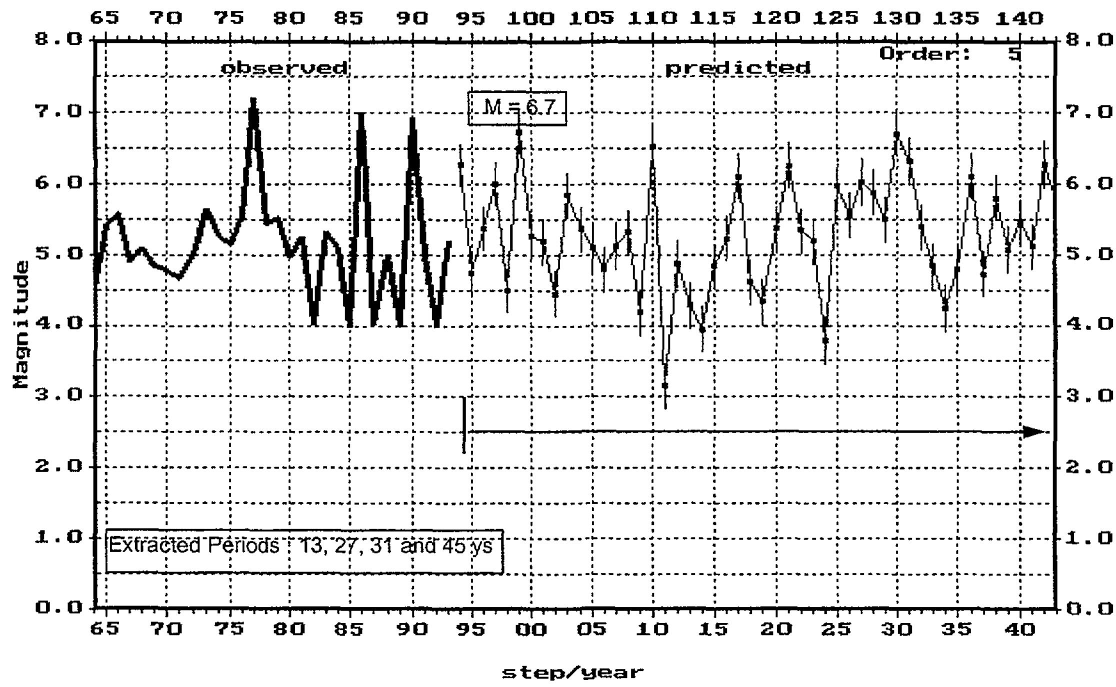

Fig. 4.1. Estimating preliminary analysis. Time - series: 1901 - 1993

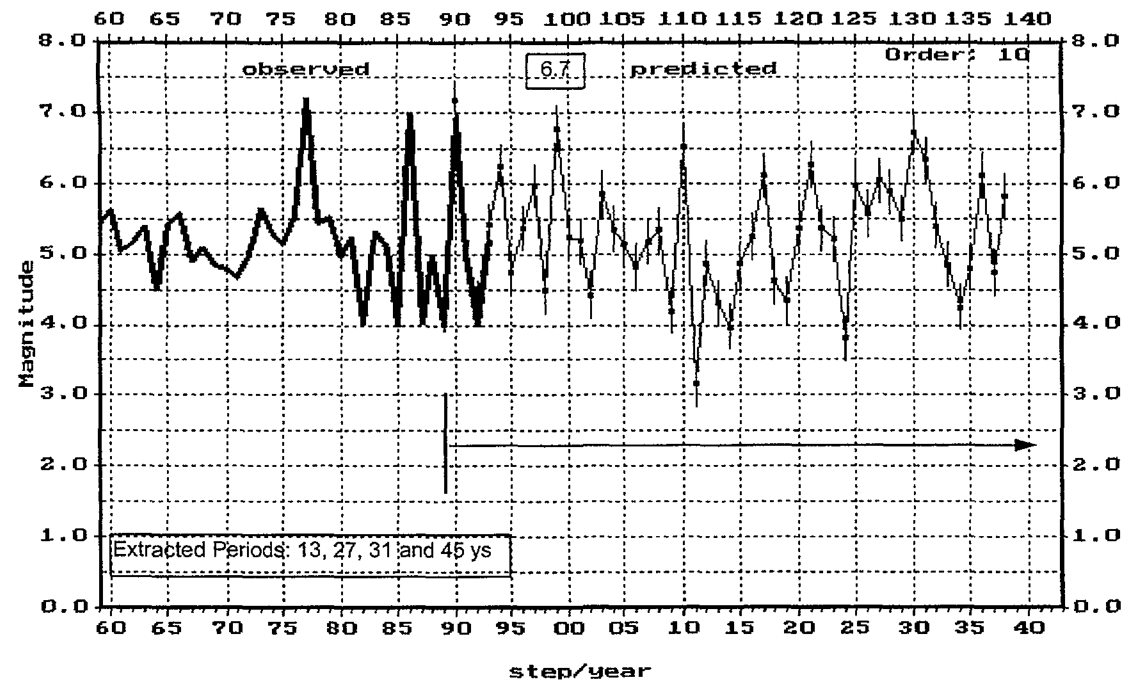

Fig. 4.2. Estimating preliminary analysis. Time - series: 1901 - 1988.

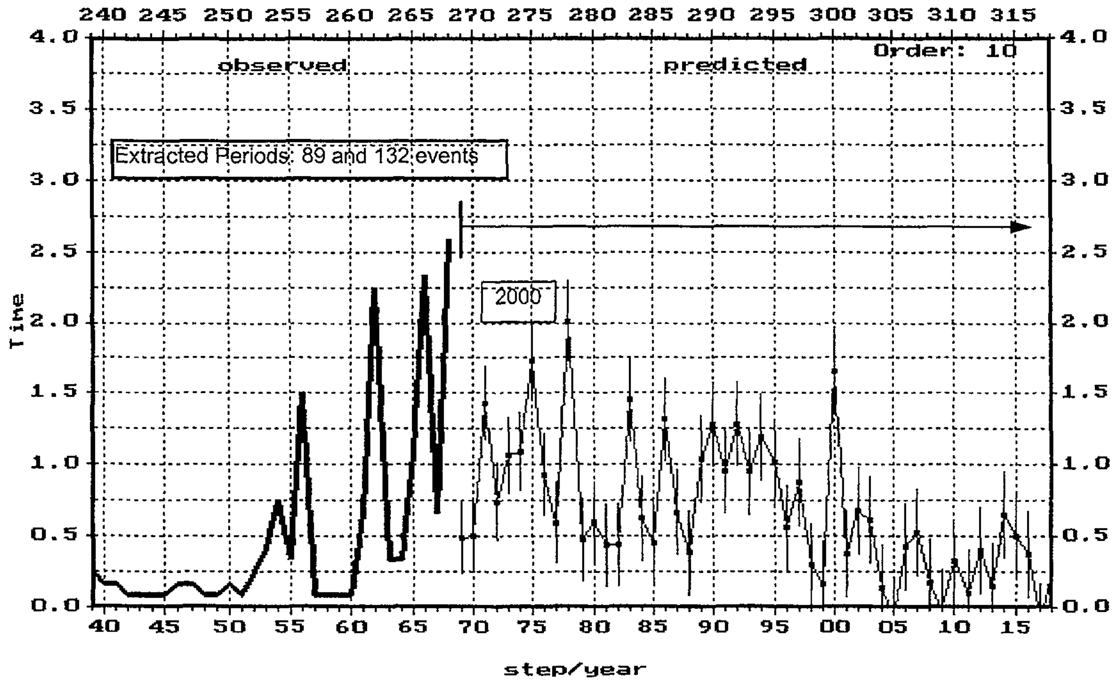

Fig. 4.3. Estimating preliminary analysis. Time - series: 1901 - 1993. Bidimensional case

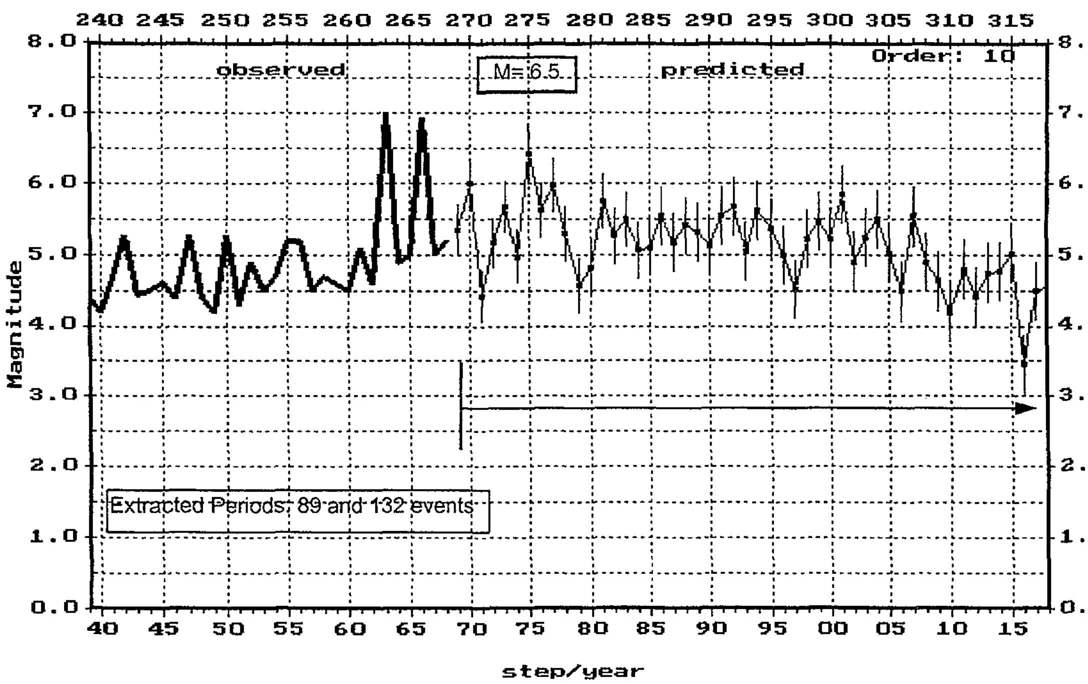

Fig. 4.4. Estimating preliminary analysis. Time - series: 1901 - 1993. Bidimensional case

5. CONCLUSIONS

This paper is a first attempt to predict the seismic activity of **Vrancea** focus based on time-series method.

Analyses for **Vrancea** focus were made using the time - series of the earthquake magnitude during the period **1901 - 1993** for which the estimation error is about **0.5** units of magnitude, according to some authors. Due to the lack of data, the analysis was done considering earthquakes of *equivalent* annual magnitudes in order to take into consideration the whole energy released during one year interval.

Following to those analyses, some conclusions could be drawn:

- **Vrancea** focus seismic activity is the result of the overlapping of some periodic components having as a basis, periodic components with the periods of about **13, 27, 31, 41, 43, 45** years;

- Although the duration of these periodic components as well as the magnitude of the components can be affected by a series of errors, such as: series length, computational method, etc., these components are quite clearly pointed out in the paper and their existence might be correlated with some phenomena related to the earth thermo - dynamics, earth tides, etc.

According to this first analysis, in **Vrancea** region, an earthquake of **6.7** magnitude might be generated in **1999** (from **Fig.4.1-4.2**); or an earthquake of **6.5** magnitude might be generated in **2000** (from **Fig.4.3-4.4**).

The magnitude average dispersion is about of **0.5 - 0.8** units of magnitude caused by the uncertainties of the initial input data, analysis method, etc.

These predictions are quite reliable because the earthquakes under investigations during **5** years (**1989 - 1993**, see **Fig.4.2**) have shown quite a coincidence with the predicted earthquakes.

Results obtained can be considered representative for the following reasons:

- the data used in the analyses are most complete by now, in the sense that small magnitude earthquakes, generated by **Vrancea** focus and available to us, have also been considered;

- the mathematical model allows processing of a large amount of information;

- the results, obtained for **Vrancea** focus by the simulation of a prognosis during **20 years** ago, showed a good fitness with the actual seismic activity of **Vrancea** focus from that data until today.

Finally, here are some proposals in order to update the **Vrancea** seismic activity data:

- continuation of researches by the above method, both by enlarging the time - series length, in the sense of considering a smaller time - interval of samples (i.e. **monthly** time - intervals) and / or by enlarging the period of research (e.q. starting with the year **1800**);

- filling in the years for which no information was available, with events generated by overlapping the periodic components established for full-data time periods;

- detailed bi-dimensional analysis using representative data;

- performance of similar analyses for at least **3** significant focus points on the **Earth** (possibly located in **USA, JAPAN** and **IRAN**) in order to determine whether those focus evidence deterministic periodic components and whether a part of them coincides with **Vrancea** focus periodic components. That would conform the hypothesis issued in this paper;

- correlation of time - duration and magnitude of periodic components with deterministic phenomena already known in the earth thermo - dynamics, earth tide dynamics forces, etc;

Based on the analyses performed, new hypotheses regarding the energy accumulation and release mechanism can be developed but that will be presented in a future stage.

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EFFECTS OF INPUT STRUCTURAL DATES FOR DISPLACEMENTS AND INTERNAL FORCES OF STRUCTURES IN CASE OF EARTHQUAKE

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Abstract

This paper analyses the effects of uncertainties in the modulus of elasticity of the constructional material, soil stiffness and the mass of structure on models corresponding to two typical structures in the Paks Nuclear Power Plant. The structure has been modelled as a beam model, and in computation of soil springs, a stiff foundation has been taken into account. Analyses show that masses must be taken into account as correctly as possible, but the effects of soil stiffness are sharply different with flexible and rigid structures. This effect in the case of flexible buildings is less important than in the case of rigid-box-like structures.

1. INTRODUCTION

In earthquake computation of structures, structural models of increasing preciseness are created, and engineers try to take the interactions between structure and soil into account approaching the reality as close as possible. For performing computations with models of high degree of freedom, not linear due to frequency-dependent soil stiffness, efficient computations methods have been elaborated. The decision on exciting spectra necessary for earthquake calculation is based on extensive analyses. Afterwards, some basic data must be given as input parameters at the beginning of computations. The task seems to be simple as modulus of elasticity of the reinforced concrete structure or the soil (soil strata) are well-known parameters. However, real values actually occurring are not known, values chosen by engineers based on various considerations will be surely others than the real ones. The same applies to the mass of the structure. No model can be precise enough to accurately demonstrate the masses computable from the dead weight of the structure, weight of the auxiliary structures (coverings, etc.) and technological equipment. In static tests, uncertainties due to these inaccurate parameters can be handled easily (e.g. applying appropriate safety factors with loads). However, in earthquake examinations, discrepancies of the above mentioned characteristics affect the dynamic properties of the system. If another frequency belongs to a given oscillation pattern, another value of the exciting spectrum must be used in computations. This value may be either larger or smaller than the original one depending on the location of the given frequency in the spectrum curve.

From the above follows that results of the dynamic computations must be handled and interpreted with proper caution because of the uncertainties in the input parameters.

2. TEST PARAMETERS AND MODELS

Effect of alteration of the modulus of elasticity of the reinforced concrete structures was examined so that calculations were carried out not only for the design data, i. e. values determined on the basis of related codes but also for three quarters and four thirds of the corresponding value:

$$0.75 \times E_{giv} < E < 1.33 \times E_{giv}$$

For mass characteristics, a large difference like with the modulus of elasticity cannot occur in a careful examination, therefore, the test interval was as follows:

$$0.9 \times M_{giv} < M < 1.1 \times M_{giv}$$

At the same time, in the case of soil stiffness much greater differences were foreseen because there is really a much greater uncertainty in these data. Furthermore, analysable effects can be only awaited for these marked differences.
Therefore:

$$0.1 \times R_{giv} < R < 10 \times R_{giv}$$

In the above relations E_{giv} , M_{giv} and R_{giv} are the given parameters, while E, M and R are the parameters the analyses were carried out with.

Presumably, effect of deviations in the individual parameters will be different for a flexible structure and a structure that can be regarded as a rigid box. In accordance with this, the ventilation chimney (Fig. 1) in Paks NPP as a flexible structure and an auxiliary building as a rigid box (Fig. 2) were chosen for tests.

Out of the horizontal and vertical response spectra applied in the tests, the horizontal response spectrum can be seen in Fig. 3.

The structure has been modelled as a beam model, and in computation of soil springs, a stiff foundation has been taken into account.

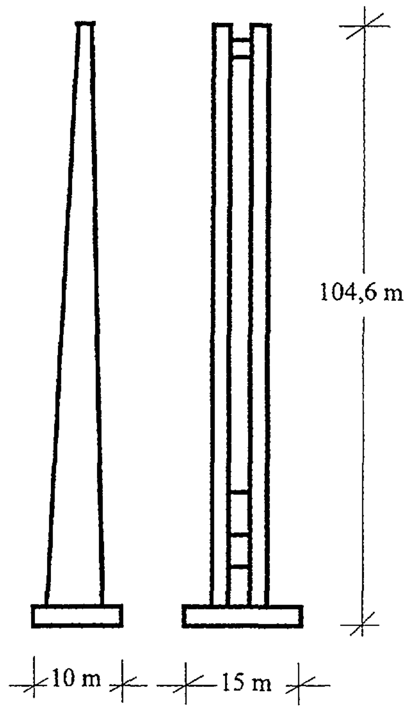

Figure 1. Flexible reinforced concrete structure: Ventilation chimney

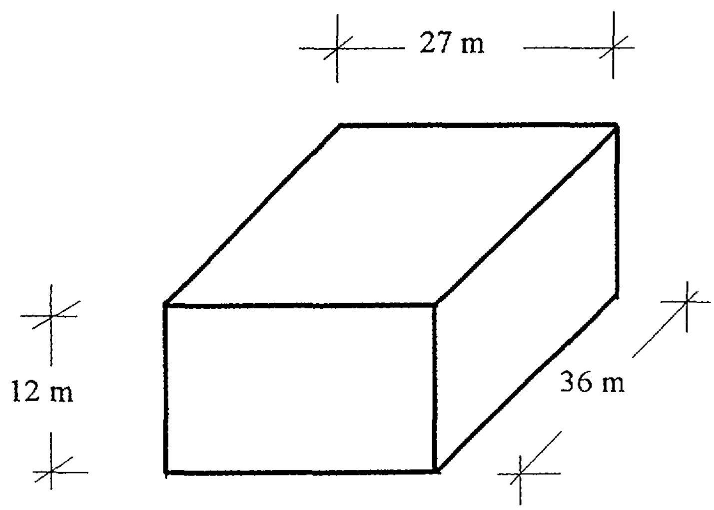

Figure 2. Reinforced concrete building with thick walls and slabs: Auxiliary building concrete block

3. RESULTS OF THE VENTILATION CHIMNEY

Fig. 4 shows the beam model of the chimney. The comparative test has been carried out for the horizontal displacement in point 1 at the chimney top, for vertical displacement in point 2 in the middle of the base plate. Internal forces were analysed in beam section 3 (in clamp cross-section of the chimney), girder 4 connecting both chimneys, and girder 5 in the base plate.

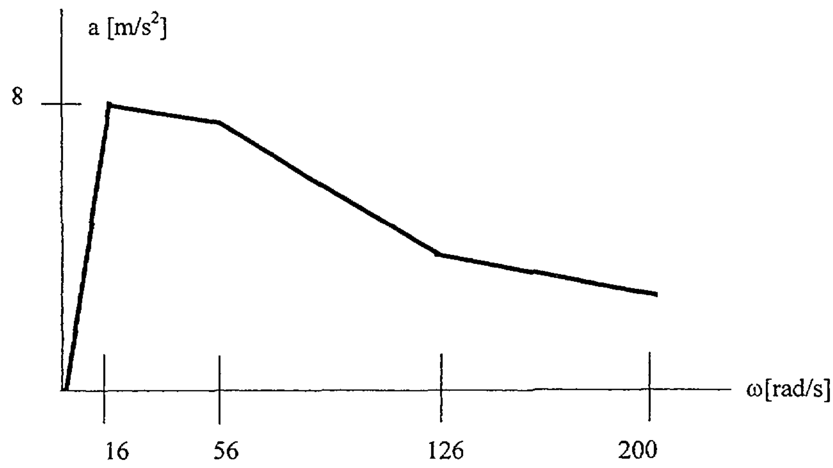

Figure 3. Horizontal Response Spectrum

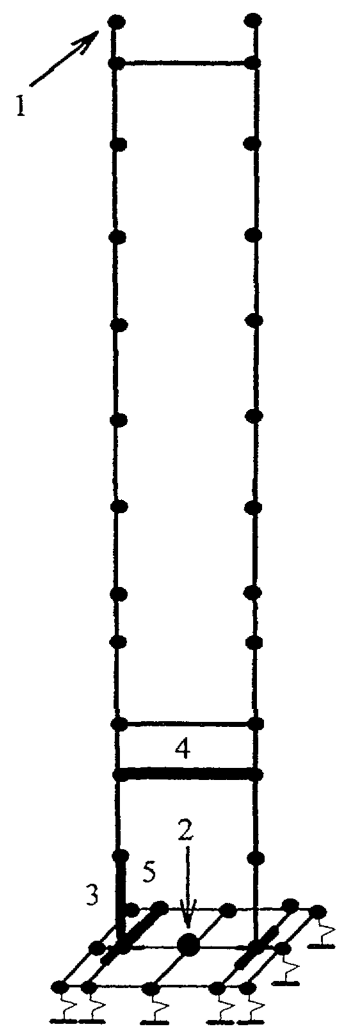

Figure 4. Beam model of ventilation chimney

At first, the eigenvector value necessary for the appropriate accuracy was analysed. If all eigenvectors belonging to the complete spectrum shown in Fig. 3 will be included in the tests, 42 eigenvectors must be used for the given structure because

$$\omega_1 = 1.37 \text{ rad/s}, \quad \omega_{42} = 208.7 \text{ rad/s}.$$

Table 1 demonstrates that horizontal displacement of the chimney top can be already obtained rather accurately with 3 eigenvectors, while calculation of appropriate accuracy of the vertical displacement of the middle point in the base plate requires at least 10 eigenvectors.

Bending moment and shear forces can be computed rather precisely with 10 eigenvectors, but accurate normal forces result only from more than 20 eigenvectors. Finally, in comparative tests 30 eigenvectors were included. Fig. 5 displays the location of the natural circular frequencies in the response spectrum. It reveals that for the flexible structure the first natural circular frequencies belong to the fast increasing section of the spectrum, and the eigenvectors included in the computations correspond to natural circular frequencies belonging to high values of the spectrum curve.

Table 1. Accuracy of displacements and internal forces at ventilation chimney

place of analysis	kind of value	number of eigen vectors						
		3	5	10	15	20	30	42
1	H. disp.	99,2	100,0	100,0	100,0	100,0	100,0	100,0
2	V. disp.	90,7	90,7	97,1	100,0	100,0	100,0	100,0
3	N	83,9	83,9	85,8	99,8	99,8	100,0	100,0
	T	49,1	49,1	96,2	97,7	97,7	100,0	100,0
	M	85,8	85,8	99,5	99,9	99,9	100,0	100,0
4	N	7,1	7,1	67,8	90,6	90,6	99,4	100,0
	T	84,6	84,6	99,6	99,7	99,7	100,0	100,0
	M	84,8	84,8	99,7	99,7	99,7	100,0	100,0
5	N	27,2	67,0	94,3	94,3	99,6	100,0	100,0
	T	82,7	91,7	95,6	99,9	100,0	100,0	100,0
	M	82,7	91,9	95,9	99,9	100,0	100,0	100,0

$\omega_1 = 1{,}4$ $\omega_{10} = 22{,}5$ $\omega_{15} = 38{,}1$ $\omega_{30} = 119{,}9$ rad/sec

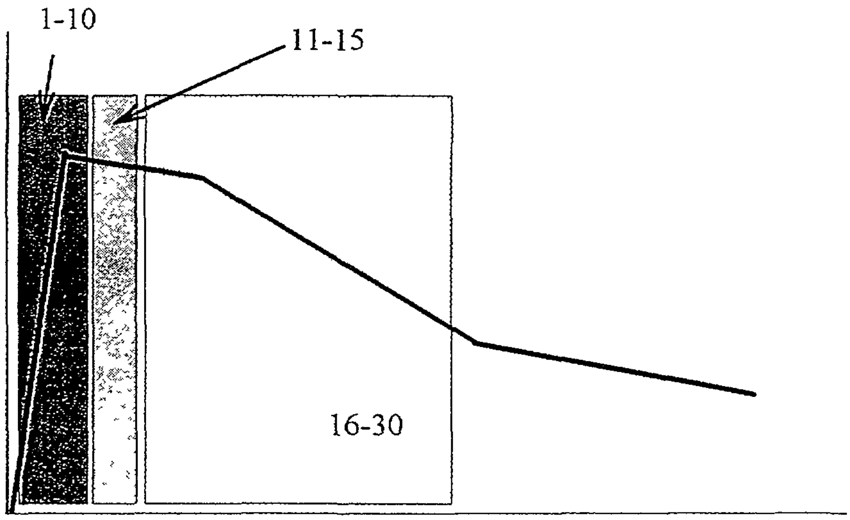

Figure 5. Natural circular frequencies in the spectrum at ventilation chimney

Fig. 6 shows the role of differences in modulus of elasticity of reinforced concrete in the solution. It can be seen that with decreasing modulus horizontal displacement increases, while vertical displacement and internal forces decrease. While modulus was diminished by 25% (or increased by 33%) in comparison to the value taken originally, changes in internal forces made no more than 15%.

Fig. 7 underlines the importance of precise choice of masses. It can be seen that a mass growth of 10% may even result in a 10% increase of internal forces.

Fig. 8 demonstrates the effect of soil stiffness. Difference was deliberately chosen not really. In spite of the fact that soil stiffness was reduced to 10% and enlarged to tenfold value, a maximum change of 25% in internal forces and horizontal displacement was observable. Simultaneously, vertical displacement increased, then decreased five times the original value. This means that in the case of flexible structures, mistakes in soil characteristics little influence the displacements and internal forces occurring due to flexibility. This small effect of soil stiffness change can be explained by the location of the natural circular frequencies in the spectrum. The location of the first ten most important natural circular frequencies in the spectrum hardly depends on the soil parameters (Fig. 9).

4. RESULTS OF THE ANALYSES OF THE AUXILIARY

Beam model of the building is depicted in Fig. 10. Comparative tests were here carried out for the horizontal displacement in point 1 in the upper plane of the structure, for the vertical displacement in point 2 in the bottom plane. Internal forces were examined in beam section 3 in the

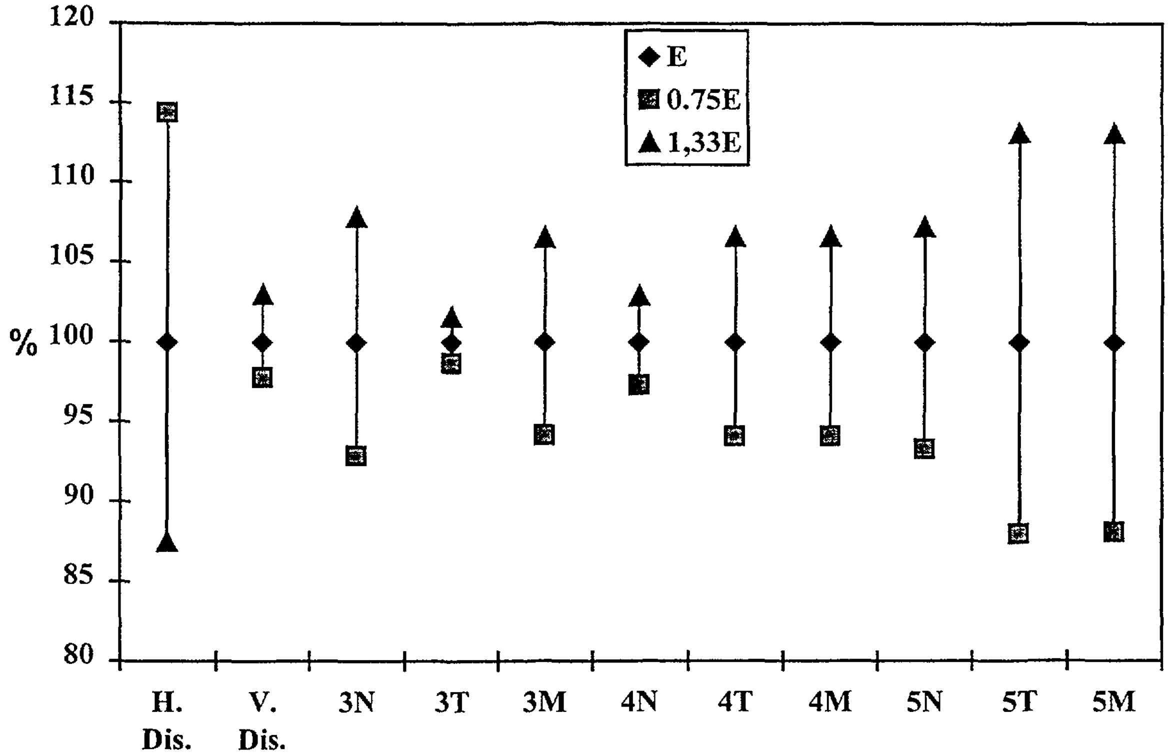

Figure 6. Influence of modulus of elasticity at ventilation chimney

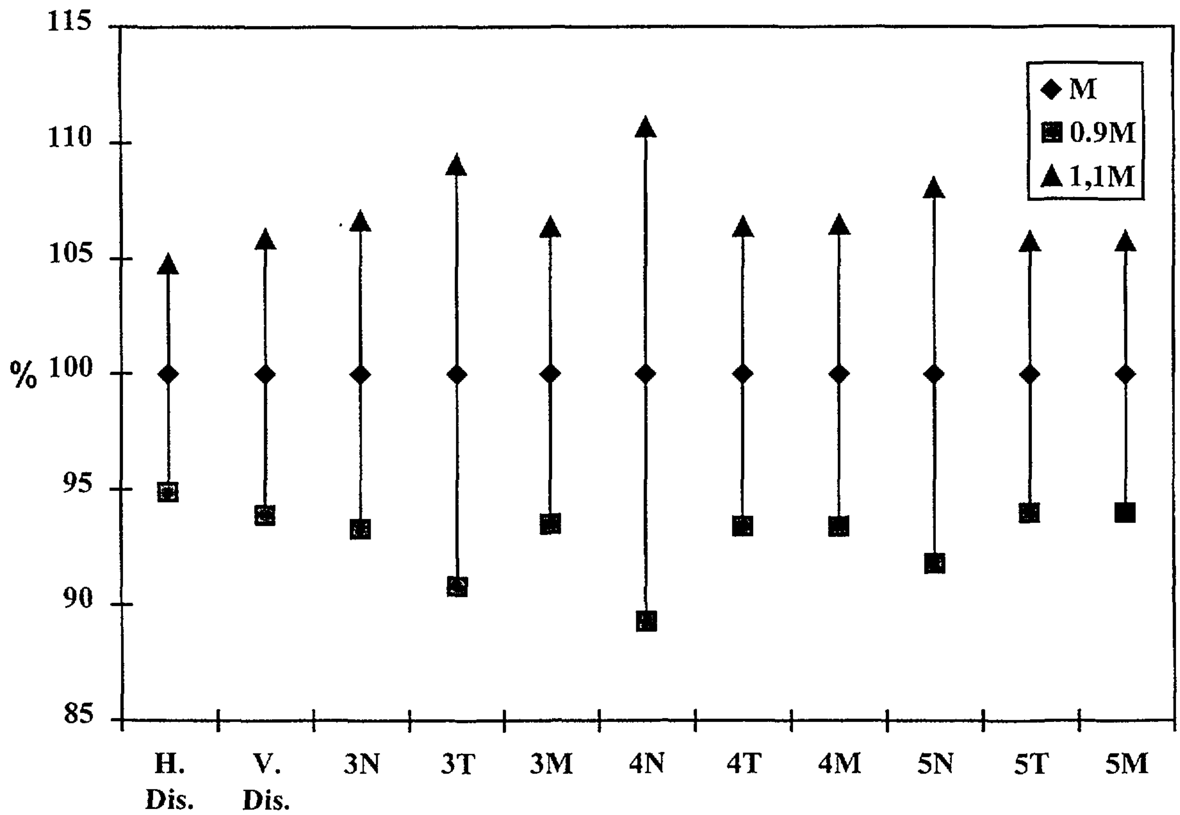

Figure 7. Influence of mass at ventilation chimney

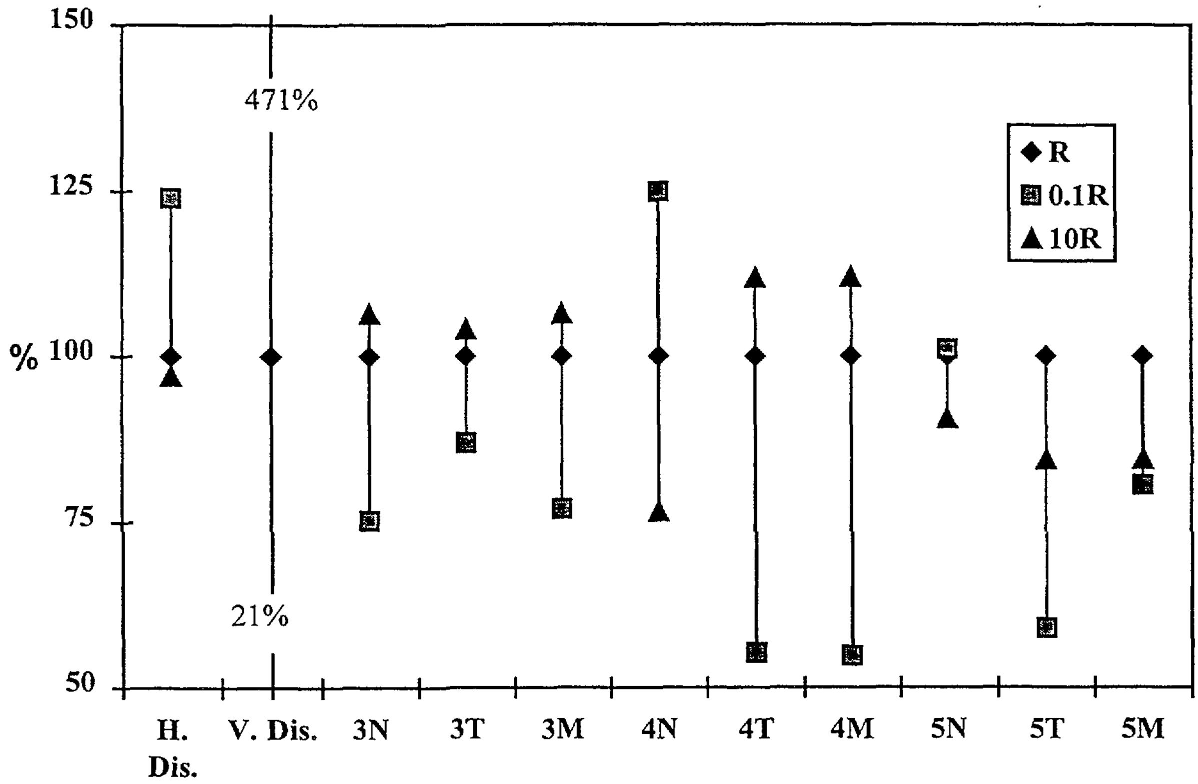

Figure 8. Influence of soil stiffness at ventilation chimney

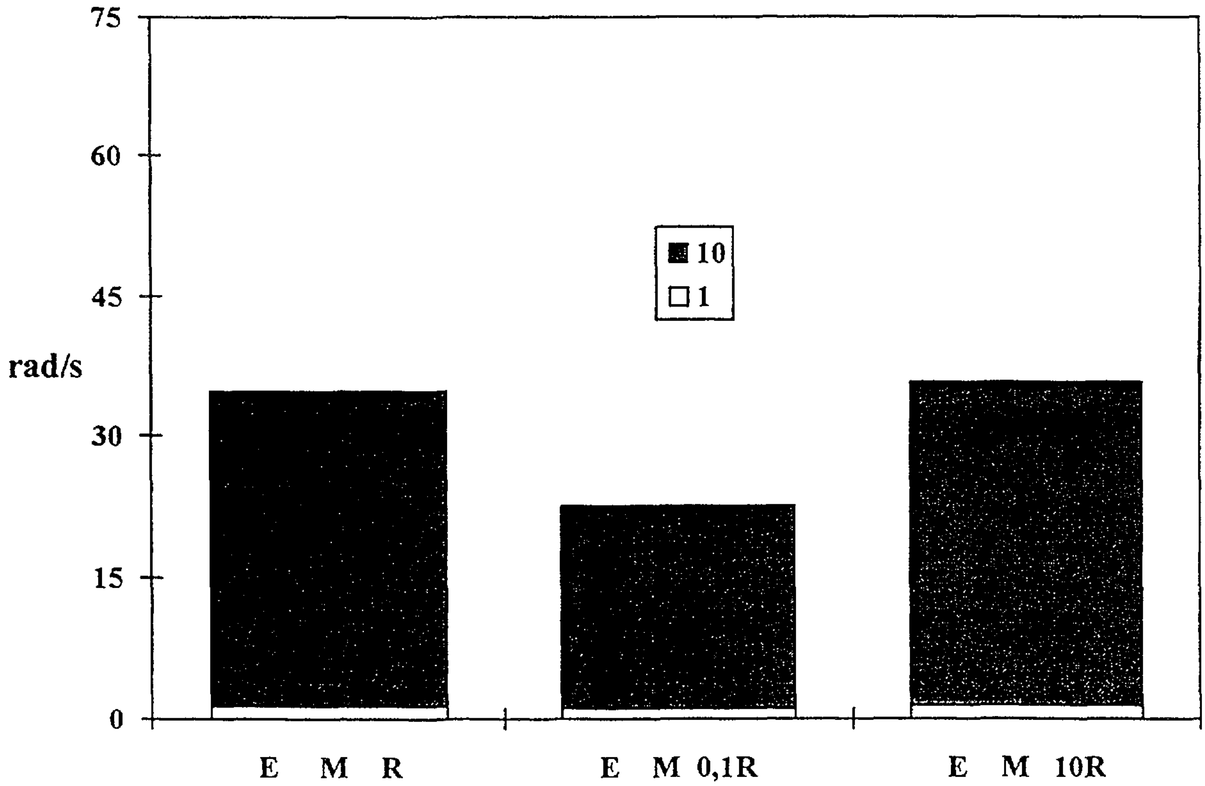

Figure 9. Location of the first ten natural circular frequencies at ventilation chimney

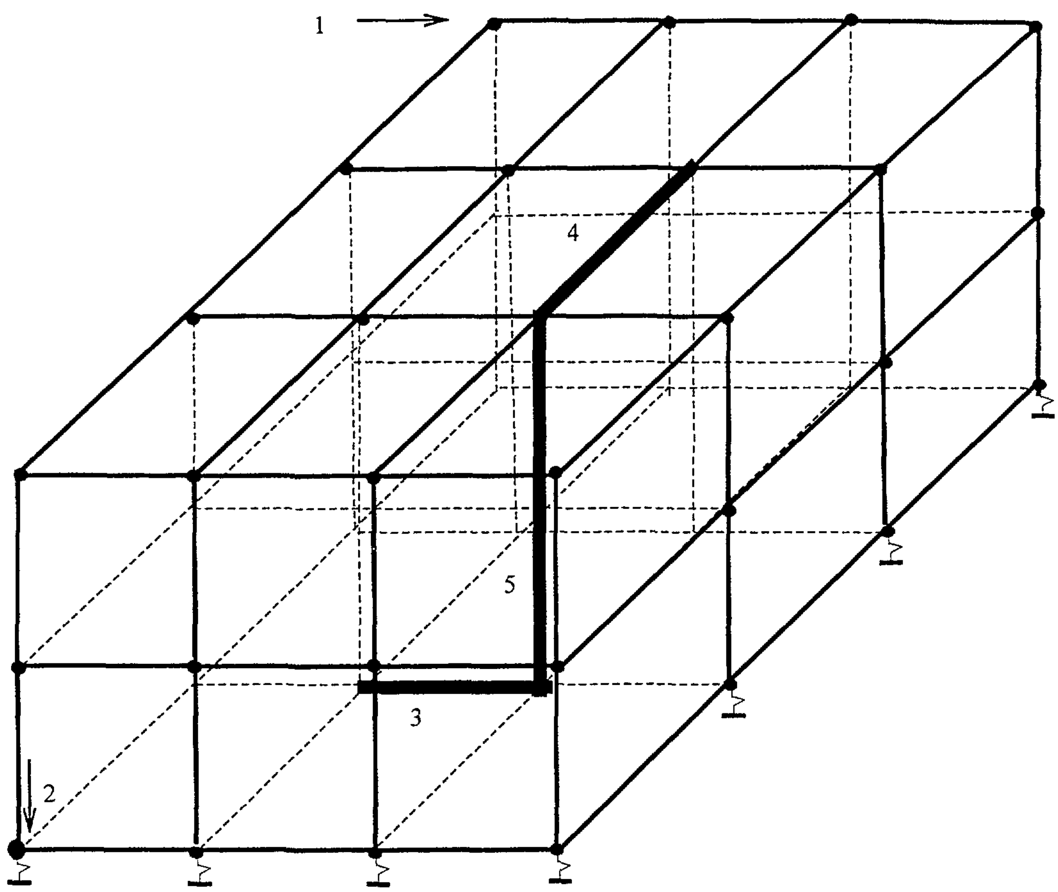

Figure 10. Beam model of auxiliary building concrete block

base plate, beam 5 in the wall and beam 4 in the upper plate. The whole spectrum covered now 44 eigenvectors.

$$\omega_1 = 9.63 \text{ rad/s}, \quad \omega_{44} = 203.7 \text{ rad/s}.$$

On the basis of Table 2 it can be stated that use of 10 eigenvectors results in computation of both displacements and internal forces with the appropriate accuracy. Fig. 11 shows that the first seven natural circular frequencies belong to the upper part of the fast growing section of the spectrum curve, and eigenvectors to be included in computations correspond in this case again to the natural circular frequencies belonging to the high values of the spectrum curve.

In Fig. 12 the effects of soil stiffness alteration are demonstrated. Unlike the flexible structure, now there is a considerable change both in internal forces and horizontal displacement, i. e. in rigid-box-like structures, the importance of mistakes in soil characteristics is much larger than with flexible structures. The more important role of soil stiffness alteration is explained by the location of the natural circular frequencies in the spectrum. Fig. 13 illustrates that the location of the most important first ten natural circular frequencies immensely depends on the soil parameters.

Table 2. Accuracy of displacements and internal forces at auxiliary building concrete block

place of analysis	kind of value	number of eigen vectors				
		5	7	10	20	30
1	H. disp.	100,0	100,0	100,0	100,0	100,0
2	V. disp.	82,4	100,0	100,0	100,0	100,0
3	N	68,2	98,7	98,9	100,0	100,0
	T	98,4	99,9	99,9	100,0	100,0
	M	99,7	99,9	99,9	100,0	100,0
4	N	92,9	97,7	99,4	100,0	100,0
	T	98,5	99,1	99,2	100,0	100,0
	M	98,7	99,4	100,0	100,0	100,0
5	N	30,9	99,8	100,0	100,0	100,0
	T	99,8	99,9	100,0	100,0	100,0
	M	99,8	99,9	99,9	100,0	100,0

$\omega_1 = 9,6$ $\omega_7 = 27,7$ $\omega_{10} = 37,3$ $\omega_{20} = 58,9$ $\omega_{30} = 129,6$ rad/s

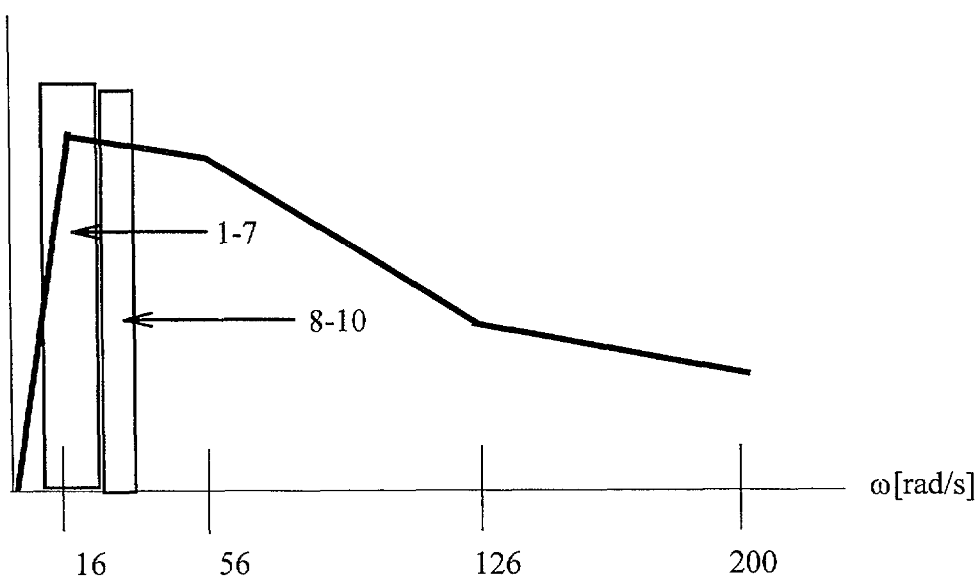

Figure 11. Natural circular frequencies in the spectrum at auxiliary building concrete block

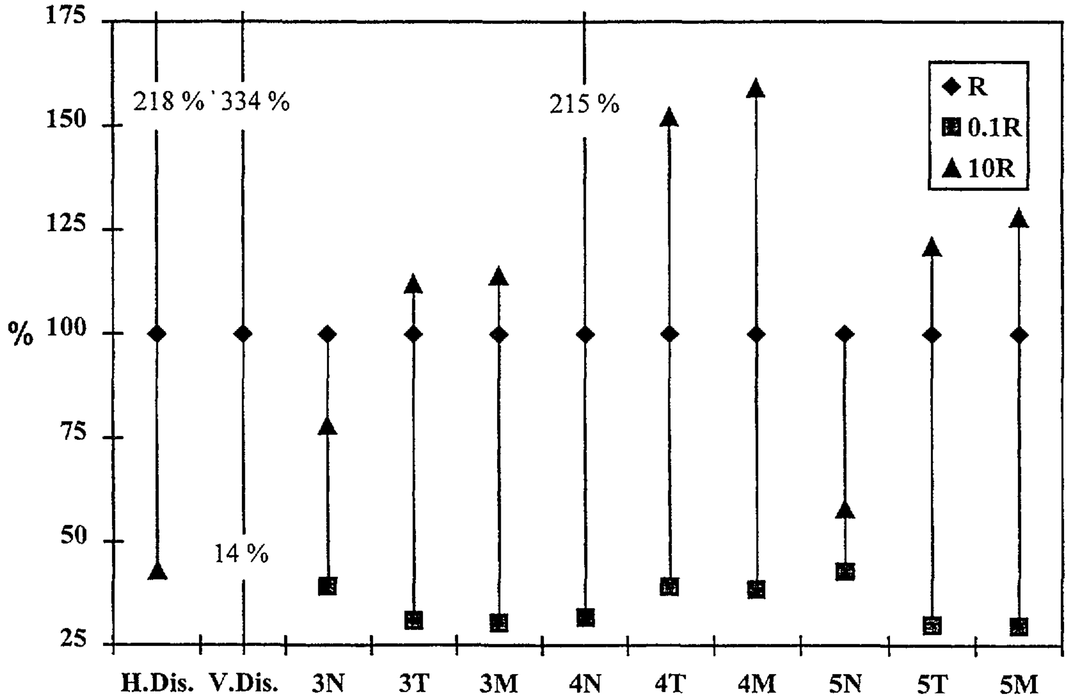

Figure 12. Influence of soil stiffness at auxiliary building concrete block

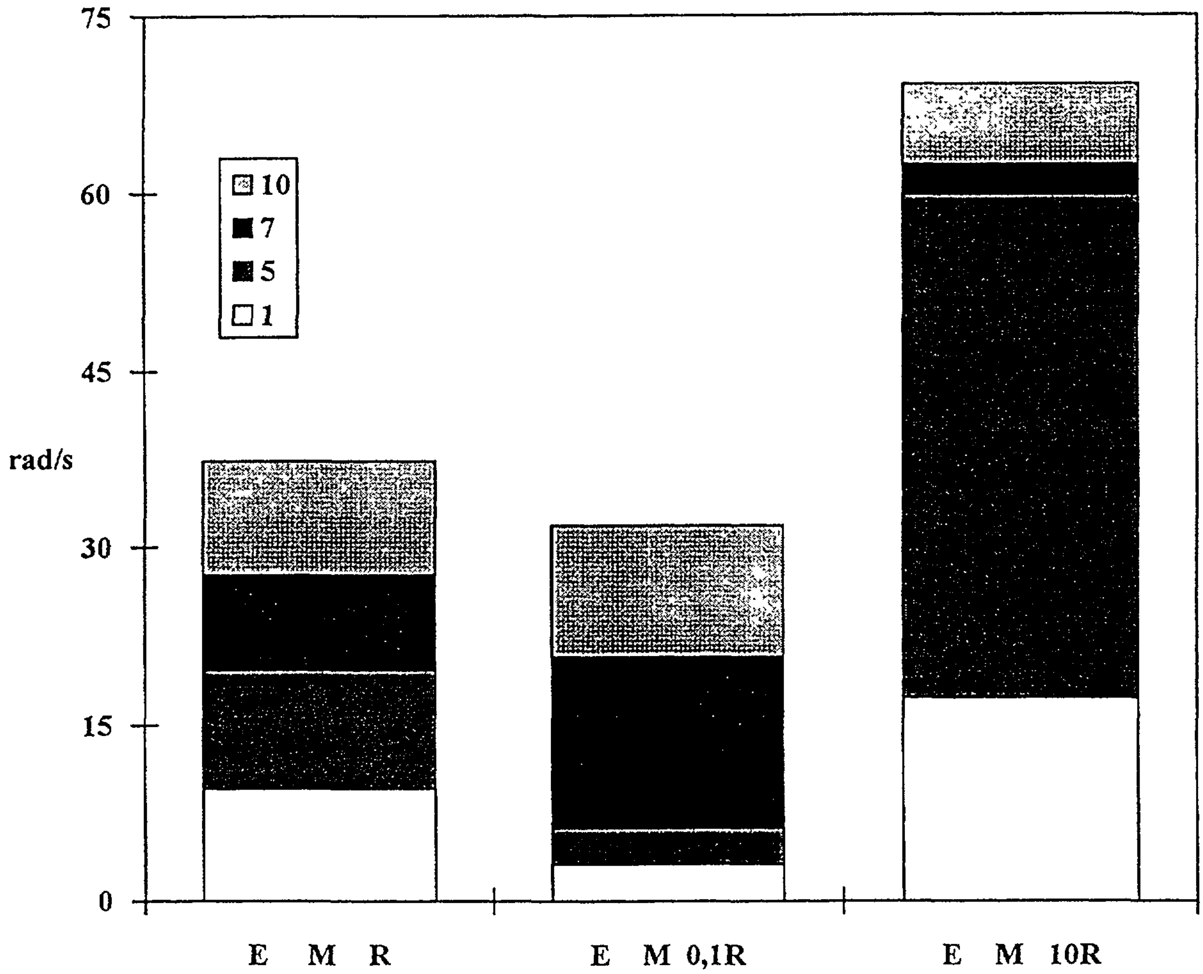

Figure 13. Location of the first ten natural circular frequencies at at auxiliary building concrete block

CONCLUSIONS

Analyses show that masses must be taken into account as correctly as possible because mistakes appearing in displacements and internal forces are proportional to mistakes made in computation of masses. At the same time, it is a comforting result that mistakes in modulus of elasticity of the structural material appear in a much less degree in internal forces, i. e. the results obtainable by values given in code specifications are close to the ones obtained by the actual modulus of elasticity.

Effects of soil stiffness are sharply different with flexible and rigid structures. In the latter case, mistakes made in soil stiffness computation lead to a significant discrepancy with the real results. This effect is, however, in the case of flexible buildings not important.

ACKNOWLEDGEMENTS

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SEISMIC EVALUATION AND STRENGTHENING OF BOHUNICE NUCLEAR POWER PLANT STRUCTURES

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Abstract

A seismic assessment and strengthening investigation is being performed for selected structures at the Bohunice V1 Nuclear Power Plant in Slovakia. Structures covered in this paper include the reactor building complex and the emergency generator station. The emergency generator station is emphasized in the paper as work is nearly complete while work on the reactor building complex is ongoing at this time. Seismic evaluation and strengthening work is being performed by a cooperative effort of Siemens and EQE along with local contractors. Seismic input is the interim Review Level Earthquake (horizontal peak ground acceleration of 0.3g).

The Bohunice V1 reactor building complex is a WWER 440/230 nuclear power plant that was originally built in the mid-1970s but had extensive seismic upgrades in 1991. Siemens has performed three dimensional dynamic analyses of the reactor building complex to develop seismic demand in structural elements. EQE is assessing seismic capacities of structural elements and developing strengthening schemes, where needed. Based on recent seismic response analyses for the interim Review Level Earthquake which account for soil-structure interaction in a rigorous manner, the 1991 seismic upgrade has been found to be inadequate in both member/connection strength and in providing complete load paths to the foundation. Additional strengthening is being developed.

The emergency generator station was built in the 1970s and is a two-story unreinforced brick masonry (URM) shear wall building above grade with a one story reinforced concrete shear wall basement below grade. Seismic analyses and testing of the URM walls has been performed to assess the need for building strengthening. Required structural strengthening for in-plane forces consists of revised and additional vertical steel framing and connections, stiffening of horizontal roof bracing, and steel connections between the roof and supporting walls and pointing of two interior transverse URM walls. Out-of-plane forces require the addition of vertical steel members attached to the URM walls with large openings and/or excessive height to thickness ratios. A practical strengthening solution combining the capacity of the existing structure, as determined by masonry testing, with the capacity of new structural elements has been achieved.

INTRODUCTION

In July 1996, Elektrarne Bohunice (EBO) entered into a contract with REKON, a joint venture of Siemens and Vuje, for major reconstruction and additions to the safety systems of Bohunice V1 located in Slovakia. The seismic evaluation and any required strengthening is being performed by a cooperative effort of Siemens, EQE, and local contractors.

The seismic evaluation and strengthening program is based on the intent of the "Seismic Qualification and Design Procedure - Part A: Civil Structure" for the EBO-V1 project (Reference 1). This procedure summarizes design criteria, procedures and application rules for seismic design and evaluation of structures in compliance with the IAEA Technical Guidelines for Bohunice and links them with the applicable national and international codes and standards. This procedure is applicable to new or upgraded Seismic Class 1 (SC1) structures or members. The criteria for seismic capacity was modified to include the results of the material testing for the existing URM walls of the emergency generator station.

Final assessment of the site seismic hazard is being performed by SAV (Slovak Academy of Sciences). Interim site-specific ground response spectra were adopted based on simplified probabilistic seismic hazard analysis. The resulting Interim Review Level Earthquake (iRLE) is the basis for seismic evaluation and design. The iRLE is shown in Figure 1.

REACTOR BUILDING COMPLEX

The reactor building complex consists of several structures connected above grade but supported on independent foundations. Individual structures comprising the reactor building complex include the reactor building/reactor hall, turbine hall, transverse electrical building, longitudinal electrical building, and ventilation hall as illustrated by the plan view shown in Figure 2. The reactor building complex was originally built in the mid-1970s but had extensive seismic

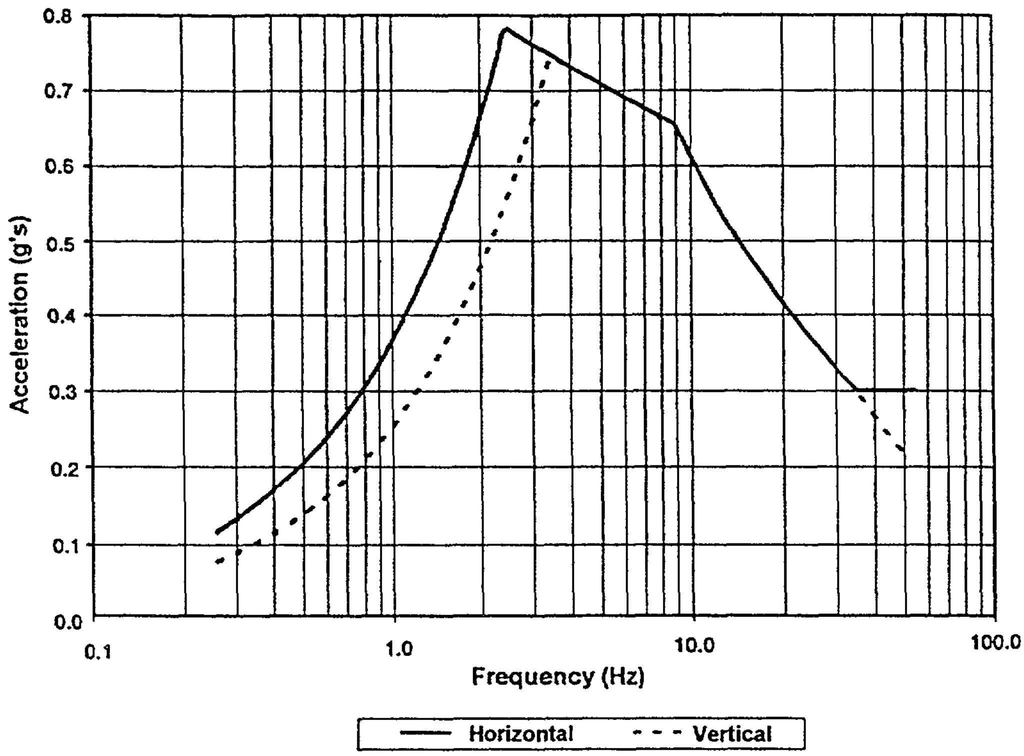


Figure 1: iRLE Ground Response Spectrum

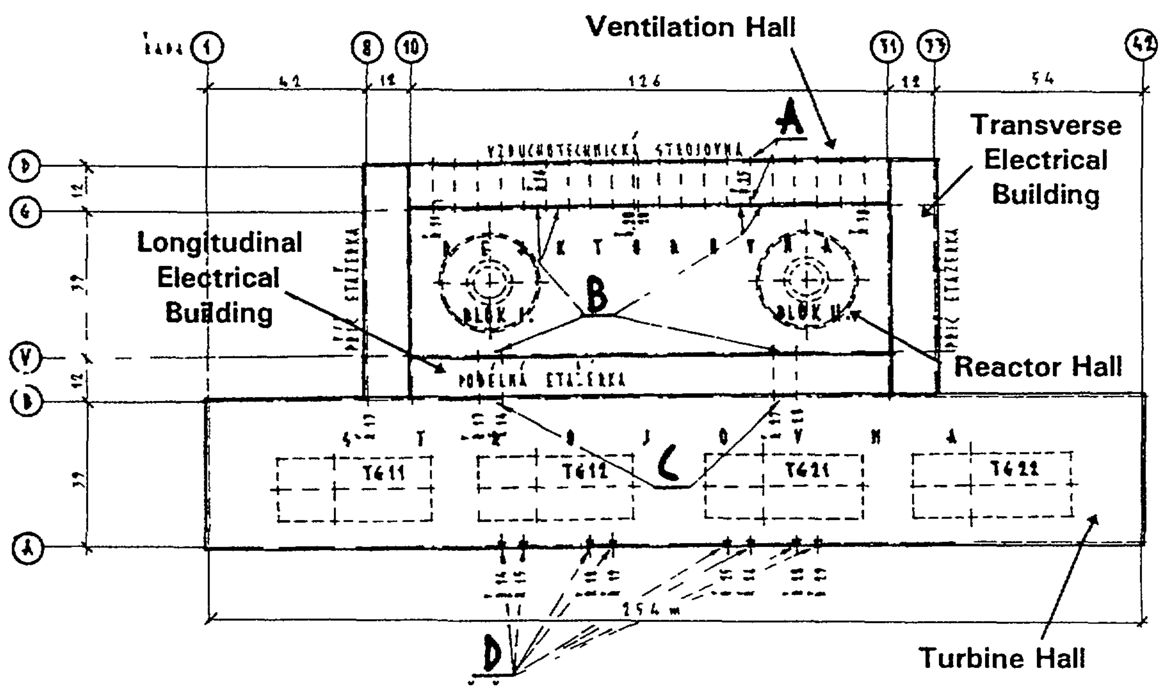


Figure 2: Reactor Building Complex Plan View

upgrades in 1991. The reactor building has very rigid reinforced concrete shear wall construction at lower elevations with a steel frame reactor hall above. In addition, a steel frame turbine hall and electrical buildings are adjacent to the reactor building. Lateral force resisting systems consist of steel diagonal bracing at the roof to distribute loads to steel frames. The original design for lateral resistance was transverse steel moment frames and longitudinal steel braced frames. Recent (1991) seismic upgrades include addition of bracing members in the transverse direction and concrete shear walls in the longitudinal direction.. In addition, the 1991 upgrades included substantial strengthening of the transverse electrical building consisting of increased connections to the reactor building concrete for seismic resistance in one horizontal direction and roof and wall steel diagonal bracing for seismic resistance in the other horizontal direction.

Siemens has performed detailed three dimensional dynamic analyses of the reactor building complex for the purpose of determining in-structure response spectra and seismic structural response. A large model representing one of the Bohunice V1 units has been developed as shown in Figure 3. The reactor building complex actually consists of two nearly identical units. There is a separation joint between the two units and only one unit is included in the structural model. Frequency domain soil-structure analyses accounting for independent foundation motion have been performed using the computer program SASSI with the iRLE as seismic input.

EQE is using the seismic demand determined from SASSI analyses to assess the adequacy of the structure and to develop strengthening measures, where they are needed Seismic demand in the elements of the reactor building complex determined for the recent seismic response analyses appear to be significantly larger than the seismic demand used to develop the 1991 seismic upgrades. Increased seismic demand is due to the use of the interim Review Level Earthquake and to soil-structure interaction behavior associated with independent foundation motions. As a result, the 1991 seismic upgrades have been found to be inadequate in some areas. For transverse seismic behavior, the strength of new steel diagonal members and their connections added between the roof of the reactor hall and the roof of the ventilation hall are not adequate for the seismic loads. In addition, there does not appear to be a reasonable and complete load paths to the foundation in

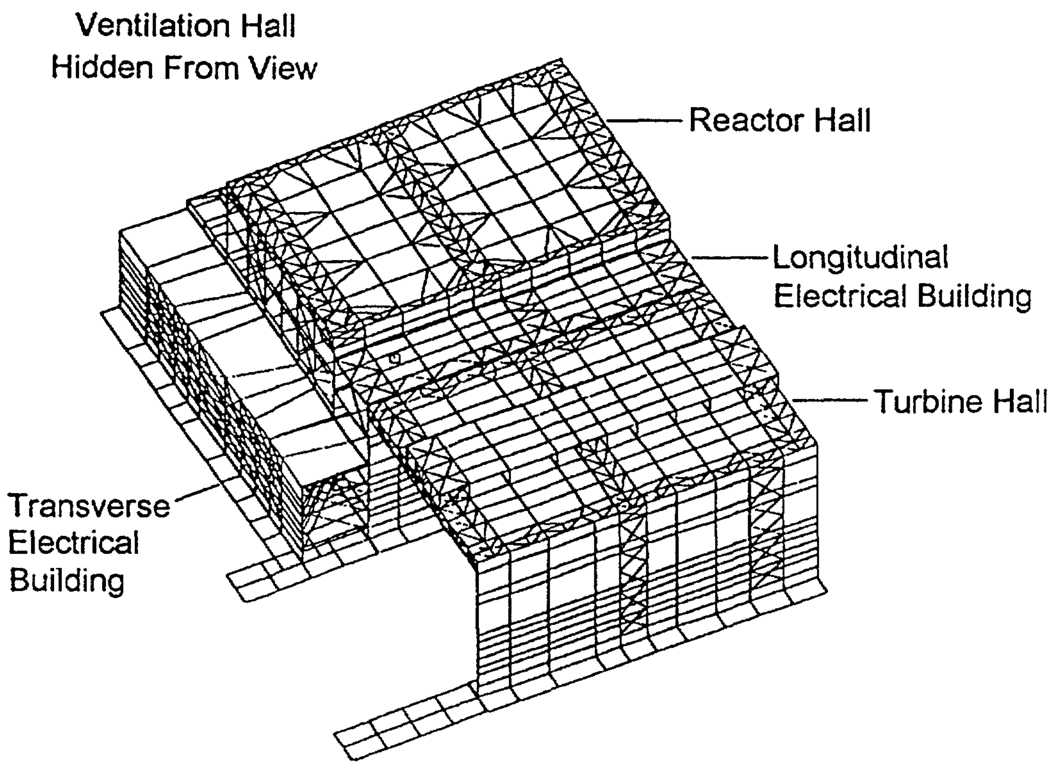


Figure 3: SASSI Structural Model of Reactor Building Complex

certain areas. Examples include provision for the reaction forces from upgrade diagonal bracing, connection of the reactor hall upgrade concrete shear walls to the supporting concrete floor, and continuity of column doubler plates down to the foundation in the turbine hall. The development of additional seismic strengthening measures based on the most recent seismic analyses is ongoing at this time.

EMERGENCY GENERATOR STATION

The Emergency Generator Station/Building (Figure 4) is a two-story unreinforced brick masonry (URM) shear wall building above grade with a one-story reinforced concrete shear wall basement below grade. The plan dimensions are approximately 54m x 19m, with an average height to top of sloping roof of approximately 10m. The station was constructed circa 1970. The basement is located approximately 3.85m below the first floor which is at grade. The second floor/mezzanine is located approximately 3.7m above the first floor. The floor area at the basement and first floor is approximately 1026 m^2. The second floor/mezzanine has a floor area of approximately 558 m^2. The total floor area is approximately 2610 m^2.

The vertical load-carrying system consists of steel beams and girders with a metal deck and rigid insulation at the roof; steel framing with a cast-in-place concrete slab at the second floor/mezzanine and first floor; spread footing and concrete slab on soil at the basement. Vertical element support for the first floor consists of reinforced concrete walls; vertical element support for the second floor mezzanine and roof consists of URM walls.

The lateral force-resisting system consists of steel bracing and a flexible metal deck diaphragm at the roof and rigid concrete slab diaphragms at the second floor/ mezzanine and first floor. The diaphragms transfer their forces to interior and exterior load bearing URM walls above grade on three sides only and reinforced concrete shear walls below grade. The existing station does not have a vertical lateral force-resisting system along the north longitudinal wall (column line A).

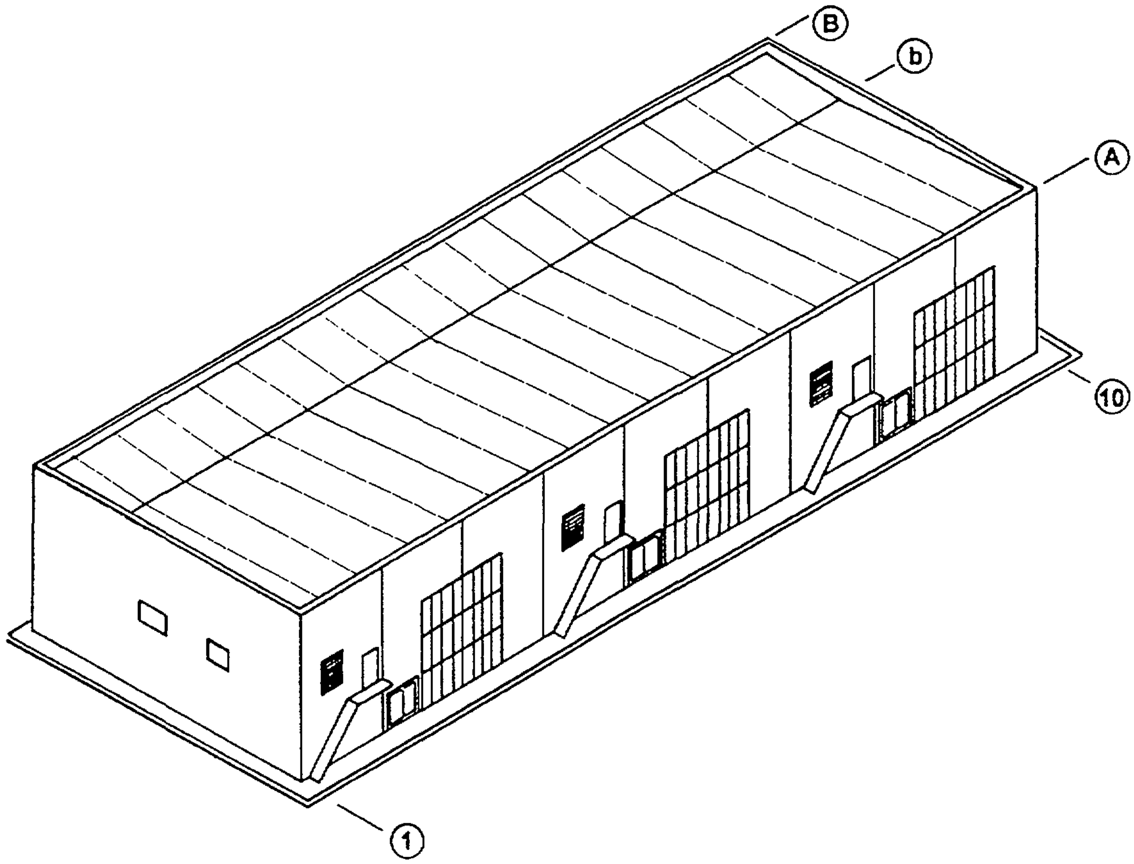


Figure 4: Emergency Generator Stations/Building Isometric

It is of interest to note that the crane bays which service the emergency generators were constructed of an independent steel frame prior to the construction of the surrounding above-grade building. The URM walls were then constructed to incorporate the existing steel elements. The crane bay steel framing is a braced frame in the transverse direction and a moment frame in the longitudinal direction.

The emergency generators are Earthquake Klass 1 (EKI) and are thus required to remain operational after a major seismic event. The station/building that houses these generators must be able to experience the designated review level earthquake without compromising the operation of the emergency generators.

The scope of work for the seismic evaluation of the Emergency Generator Station was divided into four (4) major tasks:

1. Review/evaluation of available construction documents to establish an understanding of the vertical load-carrying and lateral force-resisting systems.
2. Site visit to document the existing conditions and verify general conformance with the available construction documents. Additional site visits were also performed to verify conceptual constructibility for required seismic strengthening.
3. Seismic analysis of the existing station/building to identify deficiencies and determine required strengthening to provide sufficient resistance to withstand the review level earthquake without compromising the operation of the emergency generators.
4. Preparation of construction drawings to implement the required strengthening.

Description/Assumptions For Computer Model

A three-dimensional dynamic model of the Emergency Generator Station was developed using SAP 90 (Reference 7), a general structural analysis computer program appropriate for three-dimensional finite element analysis of many building types. Critical lateral force-resisting building elements were evaluated and demand-to-capacity (D/C) ratios were calculated. Member capacities are based on the referenced standards including the results of the material testing for the existing URM walls.

The finite element model for the Diesel Generator Station was generated taking into account all specific features of the building, based on the available drawings and site observations. The fixed-based model of the above-grade structure uses shell elements to represent both in-plane and out-of-plane shear and bending behavior of the brick walls, including existing openings and irregularities. The stiffness of the brick elements was adjusted to account for degradation during a seismic event. Steel elements were modeled using frame elements with special attention directed to assigning proper end release codes to properly model the expected performance.

The existing facade panels at building line A were modeled as masses only because they do not contribute to the lateral force-resisting system. The masses of the existing traveling cranes were modeled close to line A where a mounting ladder and parking positions are located.

The fixed-base response spectrum analysis was generated using the "Interim Review Level Earthquake with 5% damping." Forty modes were calculated using the Ritz analysis method. Note that modes at frequencies of 4.6, 8.6, and 11.2 hz are characteristic for the movement of the building in the longitudinal direction and modes at frequencies of 7.4, 7.9, 10.7, 11.6, and 13.7 hz are characteristic for the movement of the building in the transverse direction. Vertical participation is represented in modes at frequencies of 20.5, 34.9, and 36.1 hz.

Material Testing

Tests were conducted to determine the constructed strength of the existing URM walls for the Emergency Generator Station. The in-place shear tests were performed as per U.S. Uniform Building Code (UBC) standards (Reference 6) and modified write-up from SEAOC proceedings. See Figure 5 for an illustration of the procedure for in-place shear test.

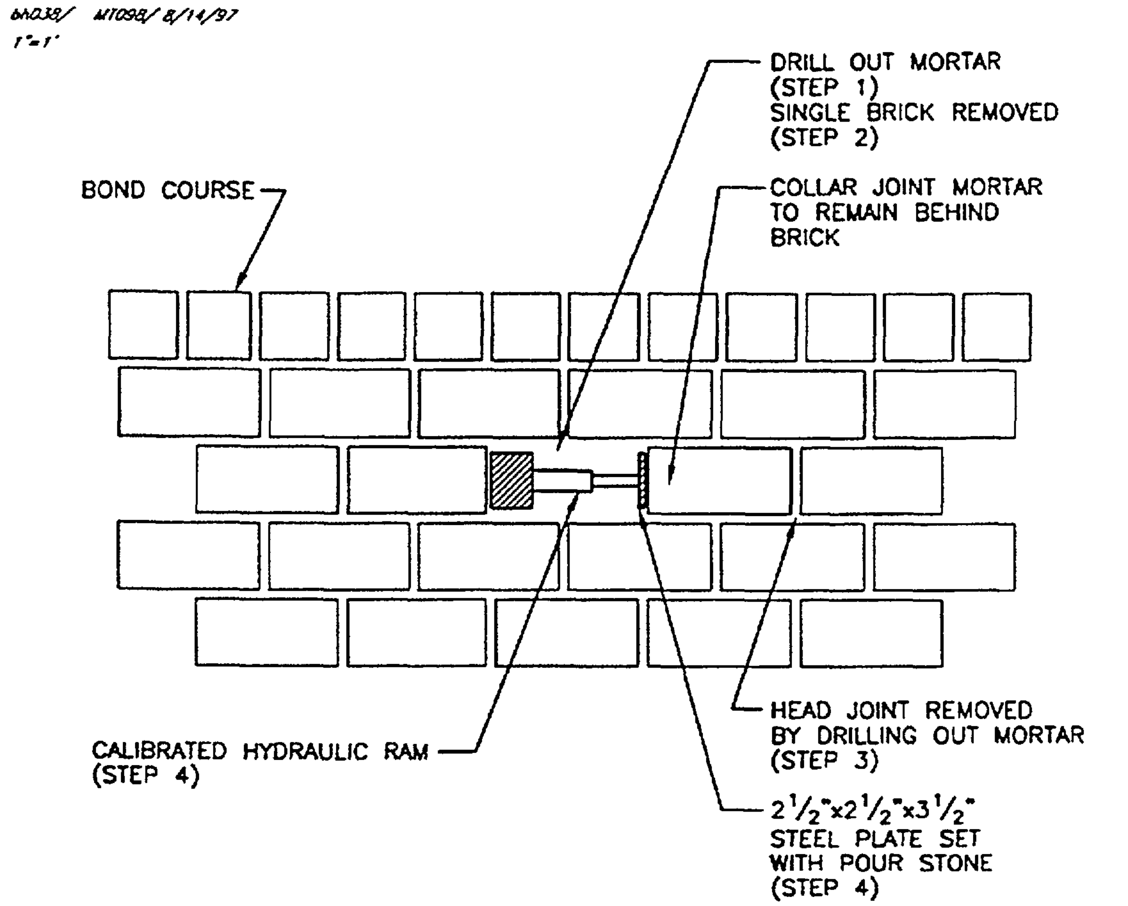
The capacity of the existing brick walls to resist seismic lateral shear forces may be determined from an evaluation of the in-place shear tests. A total of 20 tests were performed as listed in Table 1. Figure 6 is an example of test results. The values for elastic capacity are developed herein for use in determination of demand/capacity values to evaluate the existing walls.

The shear strength of the tested mortar may be considered to consist of two components. The first component is equivalent to a cohesion contribution and thus is independent of axial load. The second component is equivalent to a friction contribution and thus is dependent on axial load.

The elastic capacity is determined from an evaluation of the force-displacement curves from the in-place shear tests as shown in Figure 6. It is based upon a determination of the linear portion of the curve and a maximum offset of approximately twice the linear displacement. The cohesion component or mortar strength (V_{eto}) for elastic capacity is calculated as shown below:

- $V_{eto} = (V_{test}/A_b) - (P_D{+}L)$ where
- V_{test} is the value of test load
- A_b is the bedded and collar area of the mortar subjected to shear load
- $P_D{+}L$ is the dead plus live load on the tested brick. There was no live load present at the time of the test

PRELIMINARY
NOT FOR CONSTRUCTION



PROCEDURE:

STEP 1 Existing mortar drilled out with 5/16" diameter masonry drill x 4" long.

STEP 2 Remove brick.

STEP 3 Drill out head joint mortar x 4" deep.

STEP 4 Install jack and test.

STEP 5 "V", mortar (lbs./sq. in.) = $\dfrac{\text{P (load in lbs.)}}{\text{2x bedded area plus collar area}}$

Figure 5: Procedure for In-Place Shear Test

The total elastic capacity (V_{et}) mortar shear stress including both cohesion and friction components is computed as 56% of V_{eto} plus 75% of the dead load stress. The reduction in recorded values accounts for statistical variations in recordings and vertical seismic components of ground motion. These are empirical values being considered for UBC standards now being developed.

TABLE 1. TEST RESULTS SHEAR STRENGTH URM EMERGENCY GENERATOR STATION BUILDING (530) ELASTIC CAPACITY

WALL Grid Line	WALL Level Elev.	TEST LOC. No.	Wall Const Type [8]	DIMENSIONS Height Above Test (H) (m)	DIMENSIONS Wall Thick-ness (cm)	BRICK DIMENSIONS Width (W) (cm)	Length (L) (cm)	Thick-ness (T) (cm)	Collar Jt. COVE-RAGE (%)	Bedded Area [2]	Collar Area [2]	Shear Area [2] (cm2)	Actual In-Site Dead Load on Test [1] kPa	[7] psi	Dead Load at Mid-HT [9] kPa	[7] psi	Elastic Capacity Gauge (bar)	Force (kN)	Mortar Stress Veto [5] kPa	Mortar Stress Veto [5] psi	Mortar Stress Vet [6] kPa	Mortar Stress Vet [6] psi
4	0	1	2BK	8.80	37	11.30	23.50	11	80%	265.55	206.80	737.90	118.71	17.23	120.73	17.52	300	37.68	391.93	56.88	310.03	45.00
7	3.7	2	1BK	5.24	37	34.00	27.30	11	0%	928.20	0.00	1856.40	70.68	10.26	47.89	6.95	95	20.18	38.02	5.52	57.21	8.30
1	0	3	2BK	8.43	37	11.30	24.00	11	70%	271.20	184.80	727.20	113.71	16.50	120.73	17.52	225	33.00	340.08	49.36	280.99	40.78
1	3.7	4	1BK	5.06	37	32.30	23.30	11	0%	752.59	0.00	1505.18	68.26	9.91	47.89	6.95	450	95.58	566.75	82.26	353.30	51.28
B	3.7	5	2BK	6.04	40	11.00	24.00	11	80%	264.00	211.20	739.20	81.48	11.83	47.89	6.95	380	47.75	564.49	81.93	352.03	51.09
B	0	6	2BK	9.04	40	11.30	24.00	11	100%	271.20	264.00	806.40	121.94	17.70	120.73	17.52	325	40.00	374.09	54.29	300.04	43.55
B	0	7	2BK	9.04	40	11.30	23.50	11	100%	265.55	258.50	789.60	121.94	17.70	120.73	17.52	225	34.56	315.75	45.83	267.37	38.80
B	3.7	8	1BK	0.30	36	31.50	23.30	11	0%	733.95	0.00	1467.90	4.05	0.59	47.89	6.95	280	60.00	404.70	58.74	262.55	38.11
b	0	9	2BK	8.79	40	11.40	23.80	11	20%	271.32	52.36	595.00	118.57	17.21	120.73	17.52	325	40.00	553.70	80.36	400.62	58.14
b	3.7	10	1BK	4.74	37	32.40	23.30	11	0%	754.92	0.00	1509.84	63.94	9.28	47.89	6.95	280	60.00	333.45	48.40	222.65	32.31
4	3.7	11	1BK	4.75	34	32.00	23.50	11	0%	752.00	0.00	1504.00	64.07	9.30	47.89	6.95	450	93.44	557.20	80.87	347.95	50.50
B	0	12	2BK	9.04	36	11.50	24.00	11	90%	276.00	237.60	789.60	121.94	17.70	120.73	17.52	180	24.00	182.01	26.42	192.47	27.93
B	3.7	13	1BK	5.69	36	32.00	24.00	11	0%	768.00	0.00	1536.00	76.75	11.14	47.89	6.95	325	72.27	393.75	57.15	256.42	37.22
b	0	14	2BK	8.79	40	11.50	24.00	11	0%	276.00	0.00	552.00	118.57	17.21	120.73	17.52	150	18.84	222.73	32.33	215.28	31.24
b	3.7	15	2BK	4.59	36	11.50	23.00	11	100%	264.50	253.00	782.00	61.92	8.99	47.89	6.95	350	43.96	500.23	72.60	316.05	45.87
b	3.7	16	1BK	4.74	35.5	31.50	24.00	11	0%	756.00	0.00	1512.00	63.94	9.28	47.89	6.95	475	101.94	610.27	88.57	377.66	54.81
2	3.7	17	1BK	4.75	36	32.50	24.50	11	0%	796.25	0.00	1592.50	64.07	9.30	47.89	6.95	250	55.00	281.29	40.83	193.44	28.08
5	0	18	1BK	4.75	40	32.00	24.00	11	0%	768.00	0.00	1536.00	64.07	9.30	47.89	6.95	325	65.00	359.10	52.12	237.01	34.40
1	0	19	1BK	5.13	35.5	32.00	23.00	11	0%	736.00	0.00	1472.00	69.20	10.04	47.89	6.95	520	110.43	681.00	98.84	417.28	60.56
8	0	20	1BK	4.75	37	32.00	24.00	11	0%	768.00	0.00	1536.00	64.07	9.30	47.89	6.95	220	45.00	228.89	33.22	164.10	23.82

1) Density of Brick = 1250 kg/m3
   Dead Load Density = 1250 kg/m3 (9.81m/sec2) (kn/1000N) = 12.263 KN/m3
   Dead Load Stress = 12.263 KN (Height) (F)
   Where F = Factor for plaster on one side or wall = 1.1
   Note: Live Load
   Also: 12263 N/m3 * (m/3.278 ft)3 * (LB/4.45) = 78.23 LB/FT3
   With "F" for plaster = 78.23 * 1.1 = 86.06 LB/FT3
2) Shear Area = 2 (Bedded area) + Collar Area
   Bedded Area = WL
   Collar Area = TL(% Coverage)
3) Working Stress Mortar shear test values (Vto)
   Vto = (Force/Shear Area)(10,000) - Dead Load Stress
4) Working Stress Mortar shear strength (Vt)
   Vt = 0.80 Vto
5) Elastic Capacity Mortar shear test values (Veto)
   Veto = (Force/Shear Area)(10,000) - Dead Load stress
6) Elastic Capacity Mortar shear strength (Vet)
   Vet = 0.56 Veto + 0.75 Dead Load Stress at Mid-HT (h)
7) psi = kPa/6.89
8) Wall Construction
   1 BK = One brick thick wall
   2 BK = Two brick thick wall
9) Dead Load Stress =
   12.263 KN (h) (f)
   Where: F = 1.1, see note 1
   and h = Mid-HT of wall
   = (3.7 - 0) (1/2) +
   (10.8 - 3.7)
   = 8.95 at lower elev.
   h = (10.8 - 3.7) (1/2)
   = 3.55 at upper elev.

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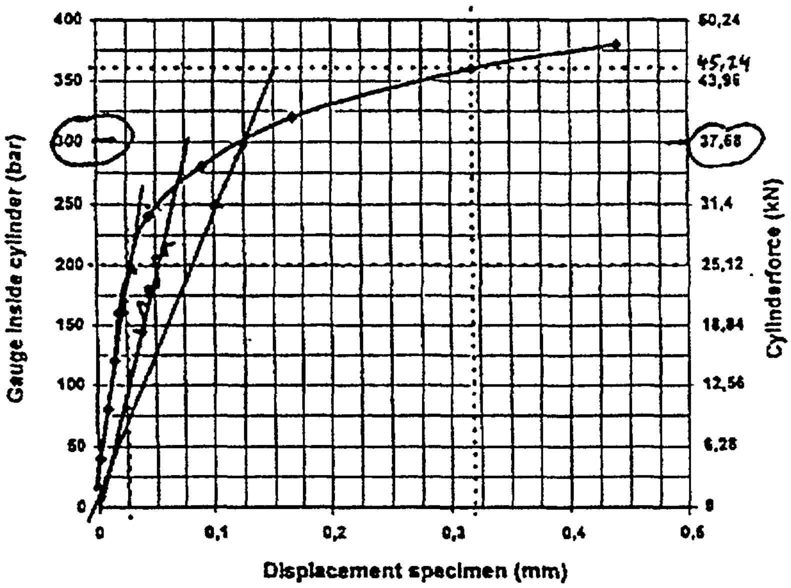
SIEMENS REKON Bohunice V1	IN-PLACE SHEAR (IPS) TEST DIESELBUILDING (530)	Index Page 1/1
	Table 1: TEST RESULTS TEST LOCATION Nr.: 1 (Internal)	Date: 24.04.97

USED CALIBRATED CYLINDER: Type: H8-40 [X] (max. 8 t) Type: H12-160 [ ] (max. 12 t)

WALL DIMENSIONS		BRICK DIMENSIONS			COLLAR JT.	IPS DATA		
						1) FIRST READING	FIRST FLAKING	ULTIMATE GAUGE
Height Above Test (m)	Wall Thick-ness (cm)	With (cm)	Lengh (cm)	Bedded Area (cm²)	COVERAGE (%)	Gauge (bar)	Gauge (bar)	Gauge (bar)
9,3	37	11,3	23,5	265,6	80	200	360	—

1) Movement = 0,001" = 0,025mm

	IPS DATA	
	Gauge (bar)	Displace-ment Speciemen (mm)
	40	0,002
	80	0,009
	120	0,015
	160	0,019
X	200	0,028
	240	0,044
	280	0,090
	320	0,167
X	360	0,319
	380	0,44



Signature: Prof. Dr. Bajza
Slovak Technical University

Signature: Grief
SIEMENS/KWU

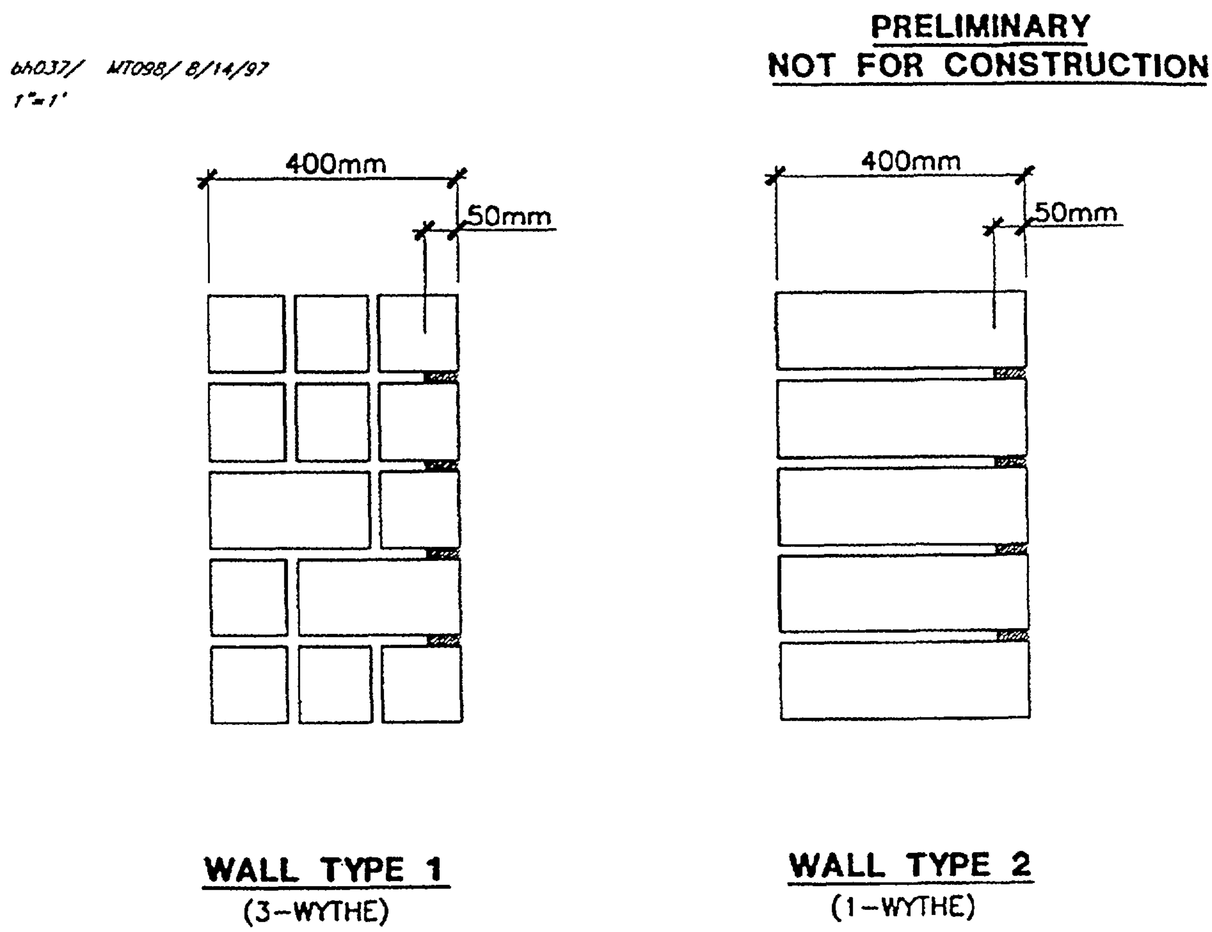
Figure 6: In Place Shear Test

Note that some of the values for elastic capacity are below the preferred lower limit of 207 kpa (30 psi). Test location 12 is located in the longitudinal wall on line B and has an elastic capacity below 207 kpa (30 psi); however, additional test locations also on wall line B and at the same elevation (i.e., test locations 6 and 7) have values above 207 kpa (30 psi). The average value for all three test locations is above 207 kpa (30 psi).

Values for elastic capacity at test locations 2, 17 and 18 are also below the preferred lower value of 207 kpa (30 psi); test location 2 is significantly below the preferred lower value. From an evaluation of demand/capacity ratios, it is required to point these interior transverse walls using 1994 UBC standards as shown in Figure 7. The requirements to point URM walls are:

1. **Joint Preparation** - The old or deteriorated mortar joint shall be cut out, by means of a toothing chisel or non-impact power tool, to a uniform minimum depth of 2 inches (50mm) until sound mortar is reached. Care shall be taken not to damage the brick edges. After cutting is complete, all loose material shall be removed with a brush, air or water stream.

2. **Mortar Preparation** - The mortar mix shall be Type N or Type S proportioned as required by the construction specifications. The pointing mortar shall be prehydrated by first thoroughly mixing all ingredients dry and then mixing again, adding only enough water to produce a damp unworkable mix which will retain its form when pressed into a ball. The mortar shall be kept in a damp condition for one



PROCEDURE:

1. Remove a minimum of 50mm of existing mortar until sound mortar is reached.
2. New mortar mix shall be Type N or Type S.
3. Tightly pack damp new mortar into joint in layers not exceeding 8mm in depth until it is filled.

Figure 7: Pointing of URM Walls

and one-half hours; then sufficient water shall be added to bring it to a consistency that is somewhat drier than conventional masonry mortar.

3. **Packing** - The joint into which the mortar is to be packed shall be damp but without freestanding water. The mortar shall be tightly packed into the joint in layers not exceeding 1/4 inch (6.4mm) in depth until it is filled; then it shall be tooled to a smooth surface to match the original profile.

Structural Evaluation

The seismic analysis of the Emergency Generator Station/Building found several concerns in the existing lateral force-resisting system of the building. These concerns include the lack of a complete lateral force-resisting system and insufficient capacity in the existing lateral force-resisting system to comply with Reference 1. These concerns are described in the following paragraphs.

The lateral force-resisting system along the exterior wall at wall line A is insufficient to resist the expected lateral forces. A complete vertical lateral force-resisting system is required along line A using some of the existing vertical steel columns modified to accommodate new steel diagonal members. Strengthening of the steel roof diagonals is required to allow the elements to carry both compression and tension loads and lateral forces.

The connections of the roof to the URM walls in both the longitudinal and transverse directions do not have sufficient capacity to transfer the roof diaphragm forces into the supporting URM walls. In addition, the roof diaphragm appears to have a horizontal separation adjacent to the wall at line b (interior longitudinal wall). Additional steel diagonal members and connections/attachments are required.

The summary of results (demand/capacity ratios) for in-plane shear forces in the walls is presented in Table 2. The average elastic capacity for each wall was based upon the results presented in Table 1 "Elastic Capacity". The demand for each wall was obtained from a three-dimensional computer analysis as described above and presented as in-plane shear stress iso-contour plots, see Figure 8. Note that Figure 8 shows in-plane shear stress iso-contour plot for Wall B, similar plots were generated for all lateral force resisting shear walls. The results show that the wall on grid line 7 and the lower portion of the wall on grid line 8 require pointing as previously discussed.

The ability of the URM walls to withstand out-of-plane forces was also considered. The capacity for each wall is generated in Table 3 based upon the theory and equations developed in Reference 2. Capacity is expressed as a coefficient, C_p which is the wall out-of-plane capacity divided by wall weight. C_p values range from about 0.4 to 12 in Table 3. The demand for each wall is obtained from the theory and equations developed in Reference 8 and essentially represents the dynamic behavior of a singly supported wall spanning between the roof and floor. For all walls, the demand is estimated to be 0.63 times the wall weight. The results show that solid walls are adequate for out-of-plane forces (except for the wall on grid line 10 which has an exceptionally large height to thickness ratio). However, it is recommended that "strongback" members be installed adjacent to large openings to assure stability.

Demand/capacity ratios for other structural members and connections were also computed. The capacity of the various elements is generated essentially by manual calculations while the demand is essentially determined from the three-dimensional computer analysis. The results show that the roof diagonal bracing members must be strengthened. This will also allow these members to act in both tension and compression and better distribute the lateral forces.

TABLE 2. SUMMARY OF RESULTS [D/C RATIO] WALLA IN-PLANE SHEAR FORCES ELASTIC CAPACITY

Wall		Demand (D)		Capacity (C)		Ratio	Compliance	Comment
Grid line	Level	PSI (1)	KPa (2)	PSI (3)	KPa (2)	D/C	Y/N	
B	0	16	112	36.76	253	0.44	Y	
	3.7	12	84	42.14	290	0.29	Y	
b	0	25	175	44.69	308	0.57	Y	
	3.7	22	154	44.33	305	0.50	Y	
1	0	23	161	40.78	281	0.57	Y	
	3.7	17	119	55.92	385	0.31	Y	
2	0	20	140	28.08	193	0.72	Y	
	3.7	14	98	28.08	193	0.51	Y	
4	0	20	140	45.00	310	0.45	Y	
	3.7	17	119	50.50	348	0.34	Y	
5	0	24	168	34.40	237	0.71	Y	
	3.7	17	119	34.51	238	0.50	Y	
7	0	20	140	8.30	57	2.44	N	Point per UBC STD 21-8
	3.7	18	126	8.30	57	2.20	N	Point per UBC STD 21-8
8	0	25	175	23.82	164	1.06	N	Point per UBC STD 21-8
	3.7	15	105	23.82	164	0.64	Y	
10	0	15	105	23.82	164	0.64	Y	
	3.7	14	98	23.82	164	0.60	Y	

(1) See Figure 11, Sample/Example In-Plane Shear Stress Iso-Contour

(2) Pa = 6.89 (psi)

(3) See Table 1

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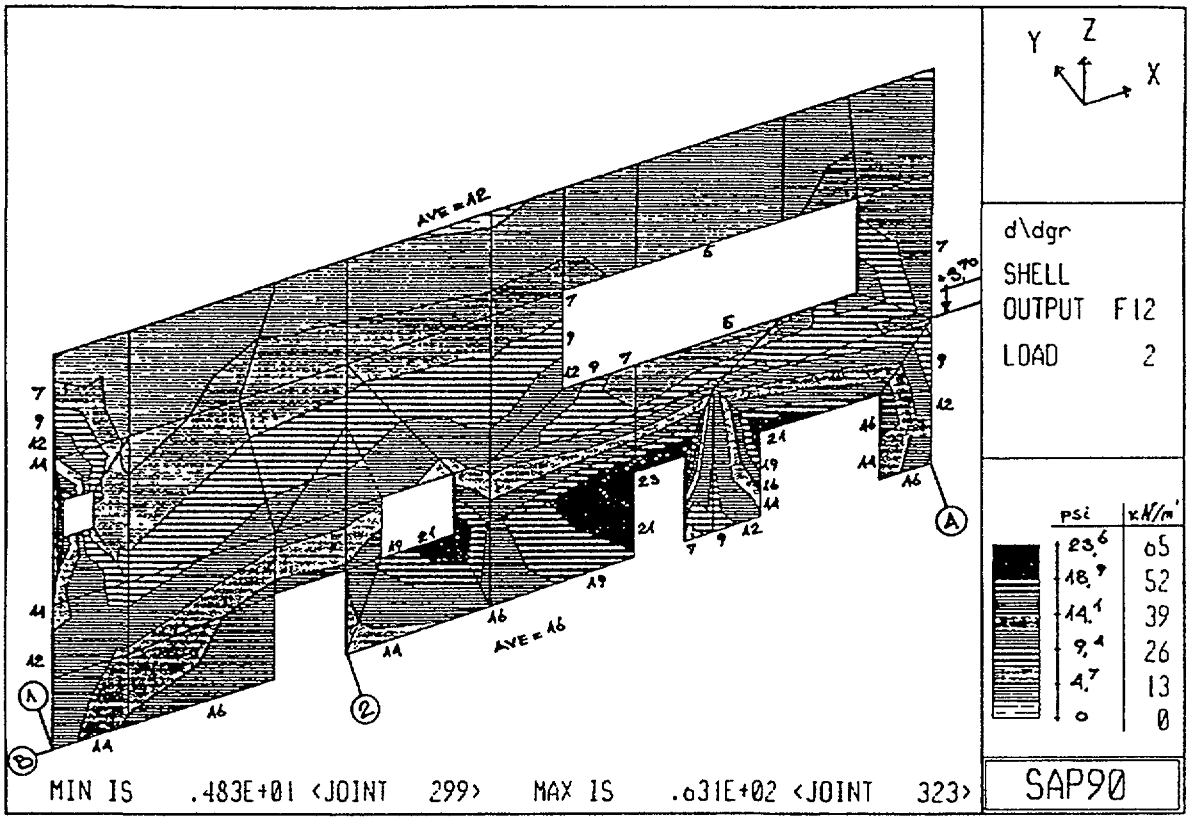

Figure 8: In Plane Shear Stress Iso Contour

Structural Strengthening/Conclusions

The deficiencies found from the seismic evaluation of the Emergency Generator Station lateral force-resisting system can be corrected by providing a supplemental lateral force-resisting system, and through strengthening of the existing system. The strengthening concept basically includes providing new steel diagonal vertical bracing members along the exterior wall at building line A; additional steel connections/ attachments at the roof to the supporting URM walls, vertical steel "strongbacks" at wall lines B, b and 10, stiffening of horizontal roof bracing and pointing of two interior transverse walls.

The five specific problem areas in the existing lateral force-resisting system of the Emergency Generator Station/Building, along with strengthening measures are described in the following paragraphs.

1. Strengthening of roof diagonals (i.e. add angle shape to make box shape element from existing single angle). This strengthening is required for all horizontal diagonal roof elements. Details are also shown to provide shear transfer between the apparent horizontal discontinuity in the roof diaphragm parallel to building line b, see Figure 9.
2. Connections of the roof diaphragm to the supporting vertical URM walls, see Figure 10.
3. Additional steel diagonal braces and connections to existing steel columns. Note that there are two sets of diagonal braces; one for the vertical lateral force-resisting system which extends from the roof to the first floor in three bays and the other set of braces is for a "drag strut" to connect the mezzanine(s) to the vertical lateral force-resisting systems located in three different bays, see Figure 11.

TABLE 3. SUMMARY OF RESULTS [C_P RATIO]: CAPACITY WALLS FOR OUT OF PLANE FORCES — REF. 2

Wall		Data								
Grid line	Level	Height h [m]	Thickness t [m]	Ratio h/t (3)	Weight (1) Wt [psf]	$D/D_{CR} = 2$ R_1	R_2 (4)	I	Pressure(2) w [psf]	C_P w/Wt
B	0	3.6	0.38	9.47	100	0.90	0.61	0.067	1,119	10.5
	3.7	6.1	0.38	16.05	100	0.77	0.61	0.031	261	2.45
6	0	3.6	0.38	9.47	100	0.90	0.61	0.067	1,119	10.5
	3.7	5.94	0.38	15.63	100	0.78	0.61	0.032	281	2.63
1	0	3.6	0.4	9	105	0.91	0.61	0.074	1,314	11.7
	3.7	6.54	0.4	16.35	105	0.76	0.61	0.03	246	2.19
2	0	3.6	0.4	9	105	0.91	0.61	0.074	1,314	11.7
	3.7	6.54	0.4	16.35	105	0.76	0.61	0.03	246	2.19
4	0	3.6	0.4	9	105	0.91	0.61	0.074	1,314	11.7
	3.7	6.54	0.4	16.35	105	0.76	0.61	0.03	246	2.19
5	0	3.6	0.4	9	105	0.91	0.61	0.074	1,314	11.7
	3.7	6.54	0.4	16.35	105	0.76	0.61	0.03	246	2.19
7	0	3.6	0.4	9	105	0.91	0.61	0.074	1,314	11.7
	3.7	6.54	0.4	16.35	105	0.76	0.61	0.03	246	2.19
8	0	3.6	0.4	9	105	0.91	0.61	0.074	1,314	11.7
	3.7	6.54	0.4	16.35	105	0.76	0.61	0.03	246	2.19
10	0	10.24	0.38	26.95	100	0.58	0.61	0.011	41	0.386
	3.7	6.1	0.38	16.05	100	0.77	0.61	0.031	261	2.45

Constants: $f'm = 1{,}000$ $k_1 = 0.50$ $k_2 = 0.30$ $S_{max} = 0.004$

Density = 1,250 kg / m³ ≃ 80 LB/FT³

(1) $$W_t = \frac{80\ \text{lb}}{FY^3} \times t(m) * \frac{3.28\ FT}{m} = PSF$$

(2) $W = (2f'm\ I\ R_1\ R_2 / h/t) * 144 = psf$

(3) Critical h/t = 21.7

(4) Minimum Value for R_2 per Ref. 2

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4. Additional "strongback" members are also recommended to be installed on walls along lines B and b due to the number of large openings and along the wall at line 10 due to its excessive h/t ratio, see Figures 12 and 13.

5. Pointing of the URM wall along grid line 7 and at the lower portion of the URM wall along grid line 8, see Figure 7.

Implementation of these recommendations will provide the increased lateral force-resisting capacity required by Reference 1 for the Emergency Generator Station to withstand the iRLE such that the emergency generators remain operational.

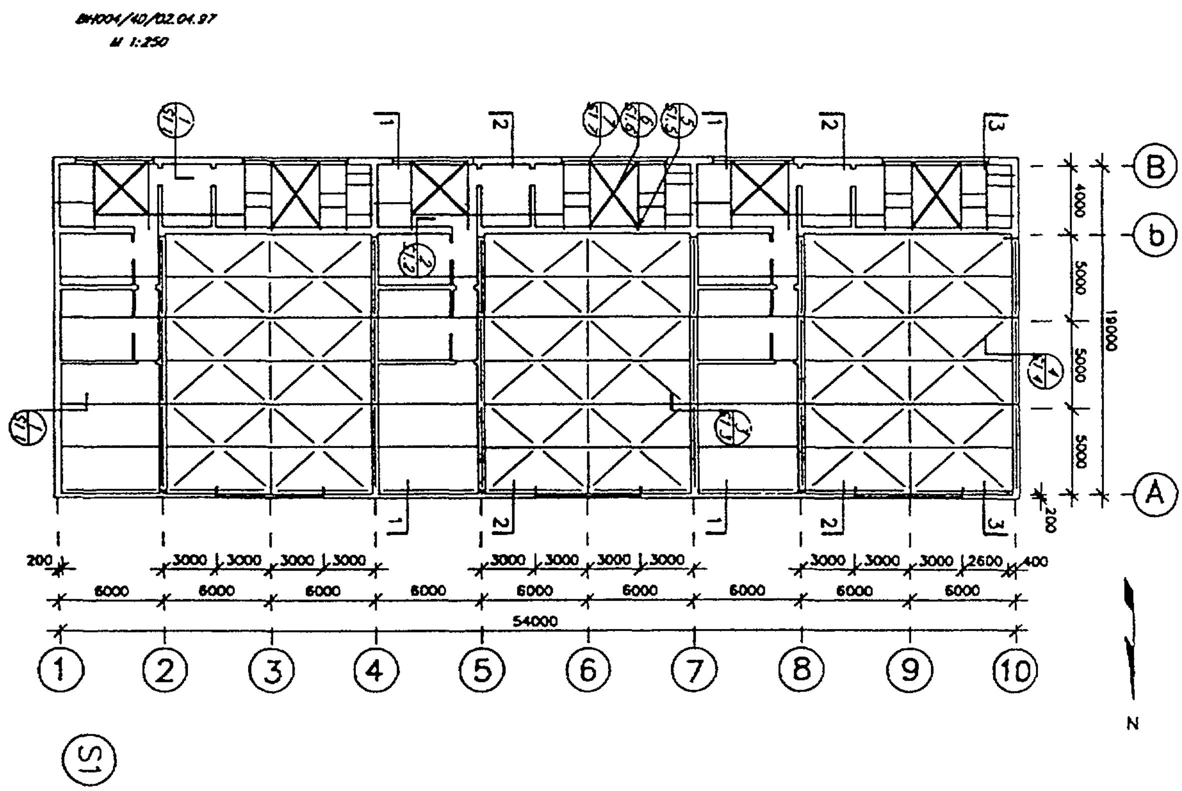

Figure 9: Level +10.24, Roof

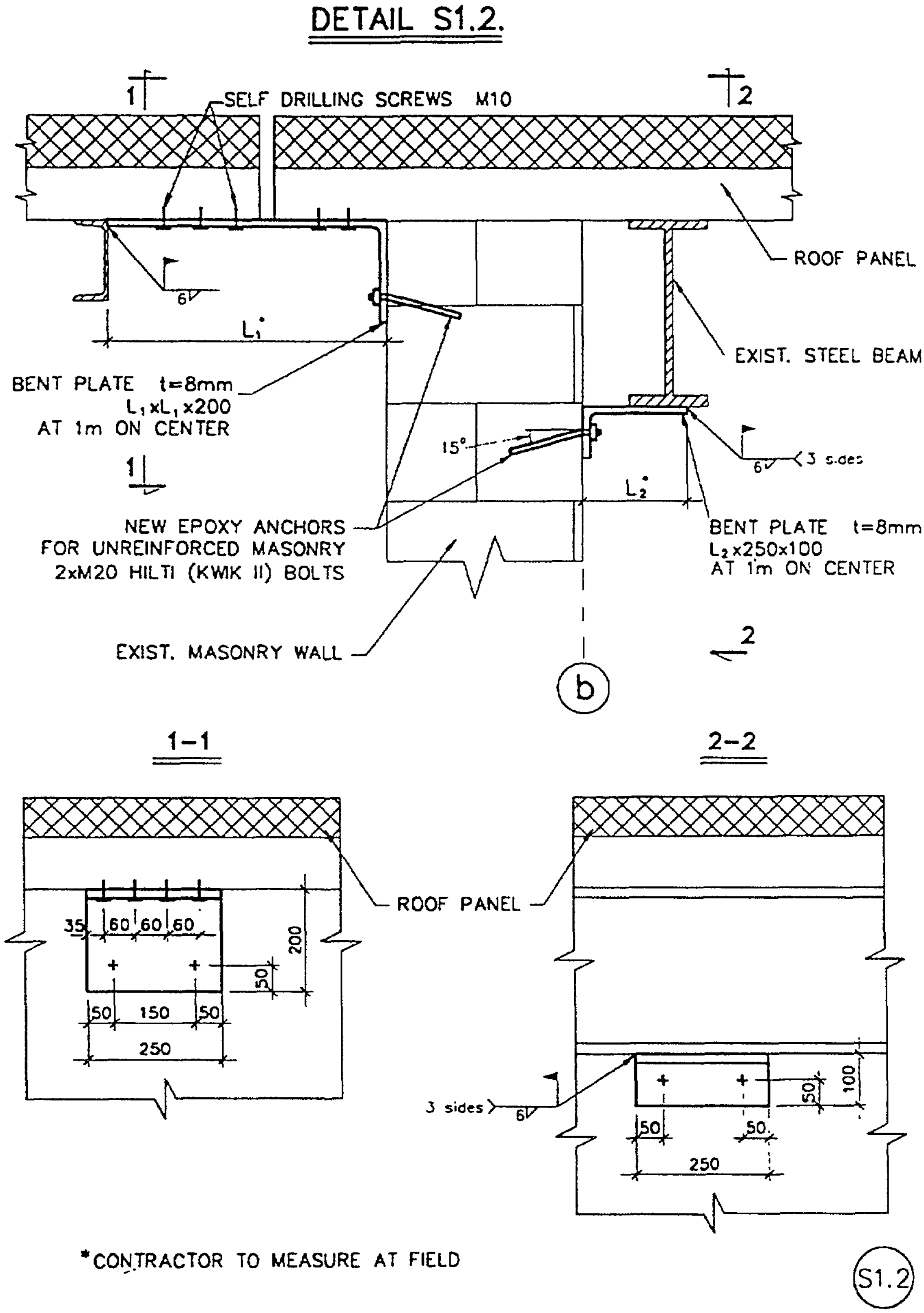

Figure 10: Detail S1.2, At Roof

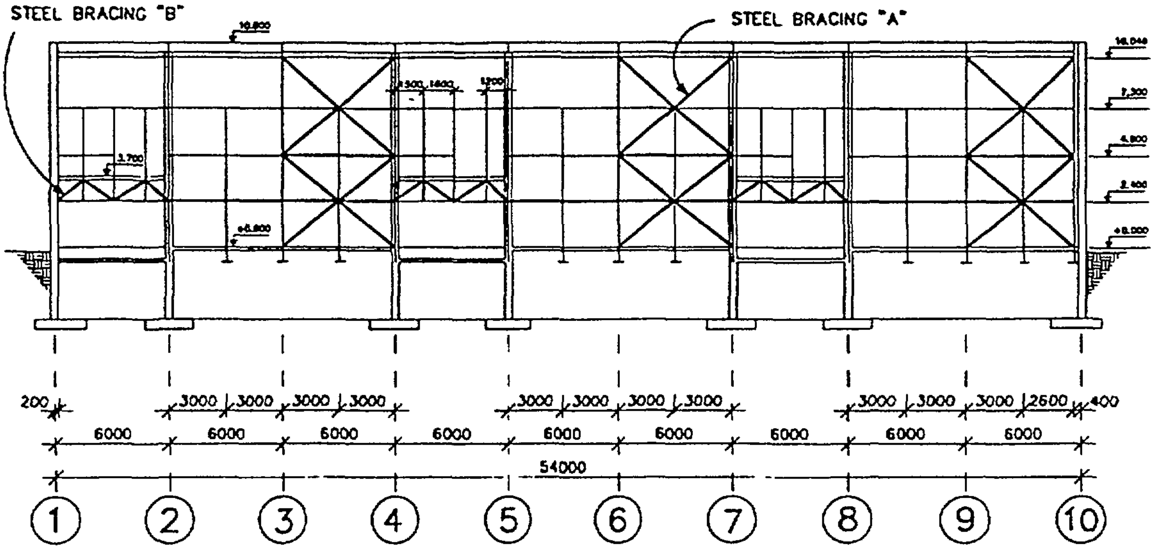

Figure 11: Exterior Wall Elevation At Line "A", Looking South

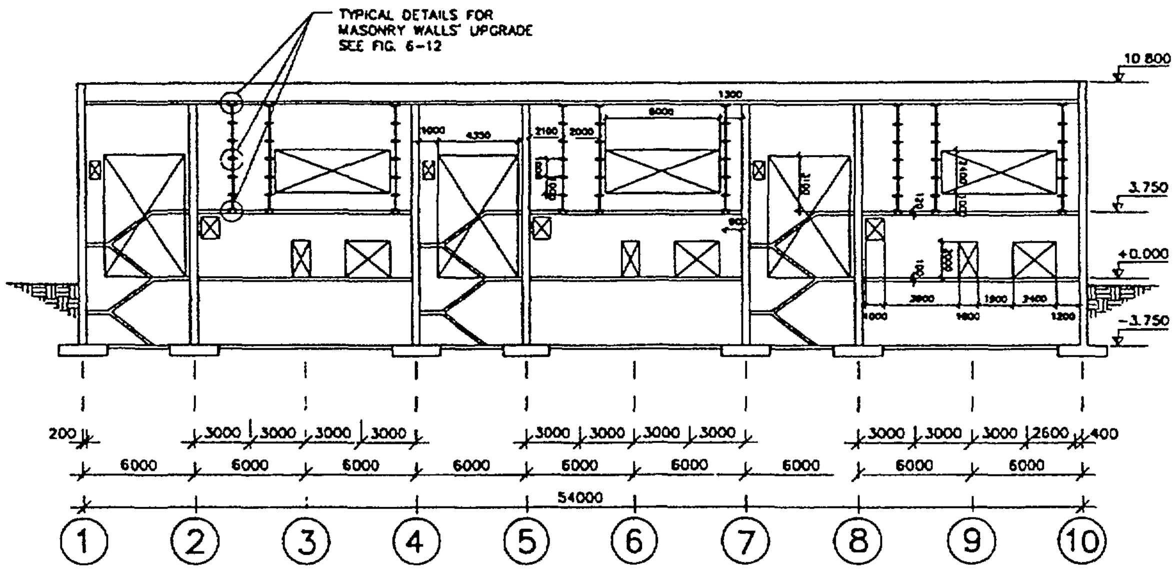

Figure 12: Exterior Wall Elevation At Line "B", Looking South

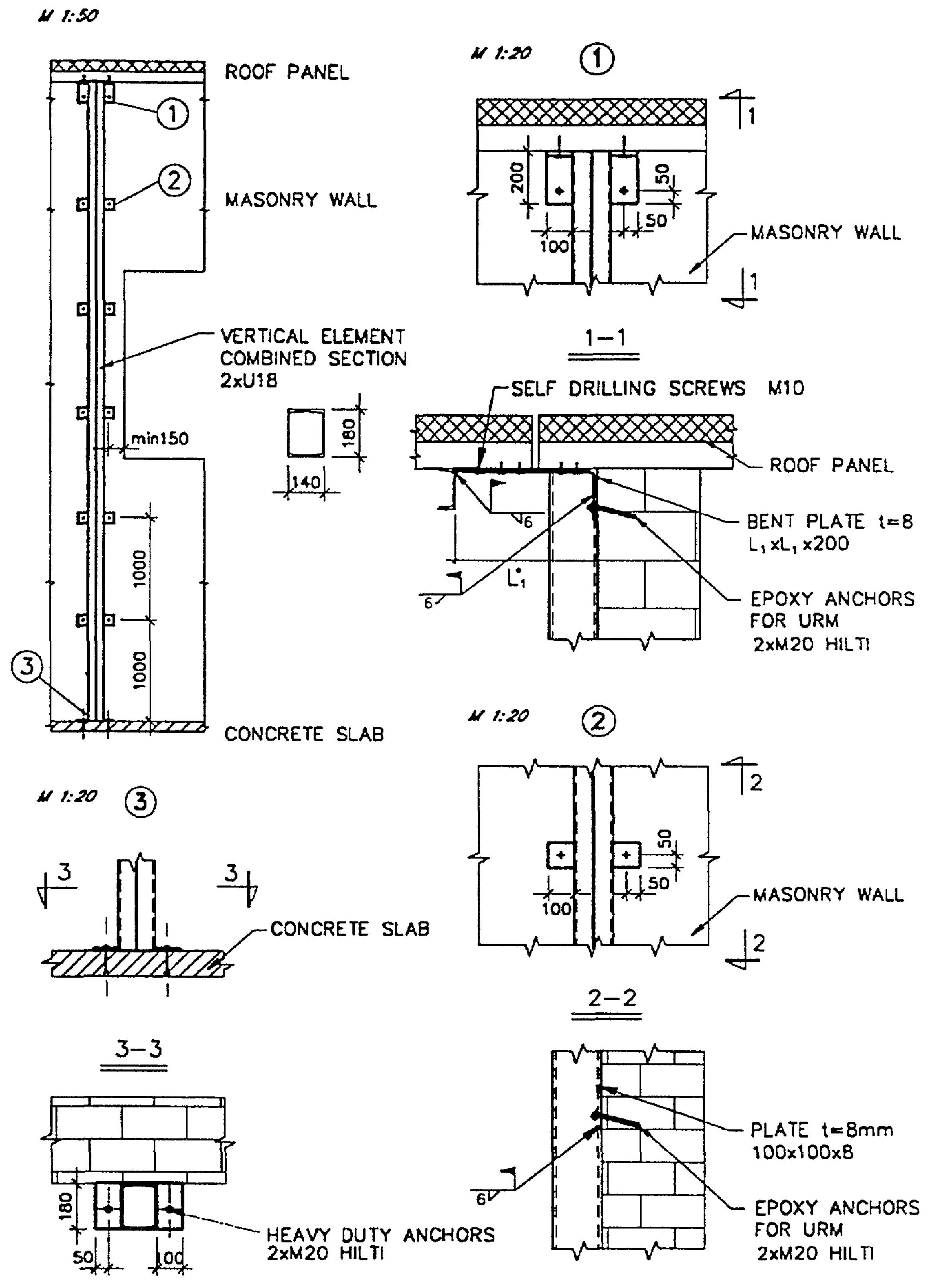

Figure 13: Typical Details for Masonry Walls' Upgrade

ACKNOWLEDGEMENTS

The work presented in this paper is part of an ongoing seismic evaluation of the EBO nuclear facility located in Bohunice, Slovakia. The support of EBO is greatly appreciated. The in-place shear tests were performed by Professor Dr. Baiza, Department of Material Engineering, University of Bratislava and technical staff from Siemens. Assistance from the University and Siemens is gratefully acknowledged.

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XA0100508

SEISMIC UPGRADING OF THE SPENT FUEL STORAGE BUILDING AT "KOZLODUY" NPP

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Abstract

The Spent Fuel Storage Building at Kozloduy NPP site has been analysed for new review level earthquake with 0.2g peak ground acceleration (compared to the initial design basis earthquake with 0.1g PGA). The preliminary seismic analysis of the existing building structure using the 5% site specific response spectrum showed the need of seismic structural upgrading.

Two upgrading concepts were evaluated on the basis of several factors. The main factor considered was preventing the collapse of the hall structure and the travelling cranes on the fuel storage area during and after a SSE.

A three dimensional finite element model was created for the investigation of the seismic response of the existing structure and for the design of the building upgrading. The modelling of the heavy travelling crane and its sub-crane structure was one of the key points. Different configurations of the new upgrading and strengthening structures were investigated.

Some interesting conclusions have been drawn from the experience in analysing and upgrading of such a complex industrial structure, comprised of elements with substantial differences in material, rigidity, construction and general behaviour.

1 INTRODUCTION

The Spent Fuel Storage Building is a cast in place and precast high-bay industrial type building constructed in 1986 at Kozloduy NPP site. The building is approximately rectangular measuring about 78 by 46 meters in plan and rises 37 meters above ground. Figure 1 shows the current configuration of the building. It is comprised of two main parts - Main Hall (between rows B and Г), with high-bay (between axes 1 and 7) and low-bay (between axes 7 and 14) and Auxiliary Building (between rows A and B). There are two travelling cranes to handle the spent fuel - a high capacity crane (160t) servicing the high-bay and a light capacity crane (16t) servicing both bays. The structure is composed by two construction types: the precast concrete construction above Elevation 7.20m, and the monolithic concrete block below this elevation where the spent fuel pools are housed and which supports the precast building elements. Figure 2 shows a typical section of the building at the transition between high and low-bays at axis 7.

The existing lateral load resisting system of the Main Hall superstructure consists of cantilevered precast columns in the transverse direction and frame resisting frames precast columns and beams as well as interior concrete shear walls in the longitudinal direction.

The existing structure has been designed for an anticipated seismic input motion with 0.1g peak ground acceleration. With the application of stronger safety requirements for NPPs it came clear that the facilities of Kozloduy NPP should be designed to withstand safe shutdown earthquake (SSE) with 0.2g peak ground acceleration. Considerable efforts have been made by experts from different institutions, essentially supported by IAEA, to develop more adequate seismic input characteristics for the seismic qualification of facilities associated with the safe shutdown of the plant. This included development of a free field Response Spectrum for Kozloduy site, anchored at 0.2g PGA, which was approved by IAEA [1], Figure 3.

The preliminary seismic analysis of the existing building structure using the 5% site specific response spectrum showed the need of seismic structural upgrading.

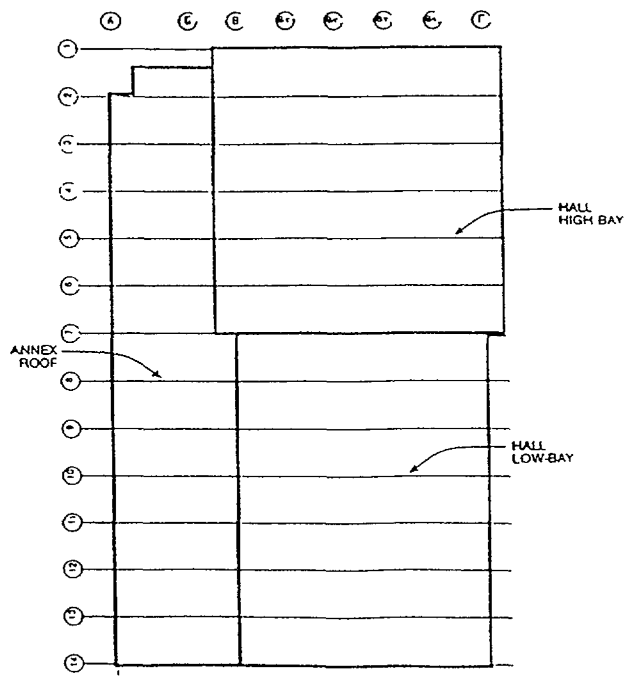

Figure 1 Plan View of the Spent
Fuel Storage Building

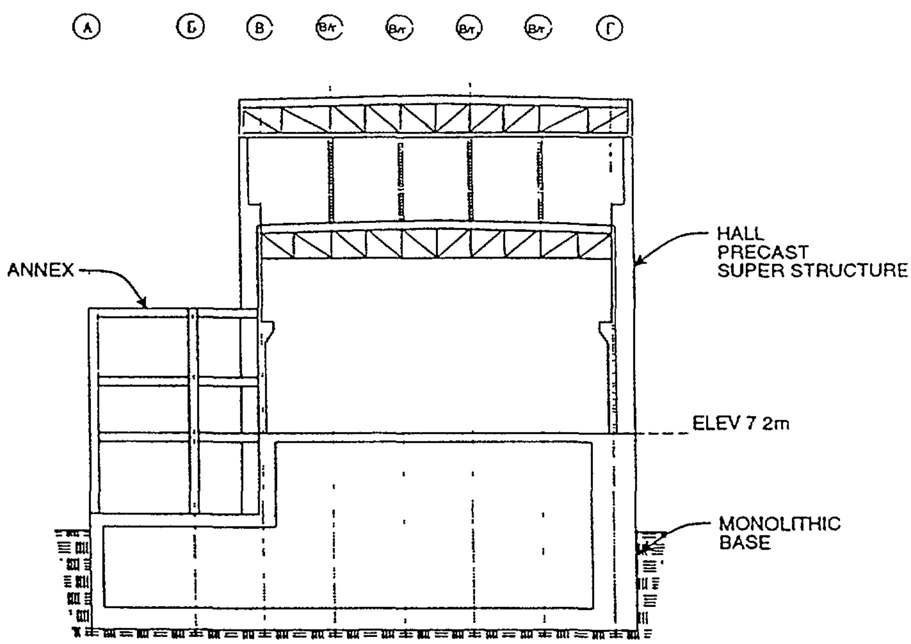

Figure 2 Vertical Cross Section at Axis 7

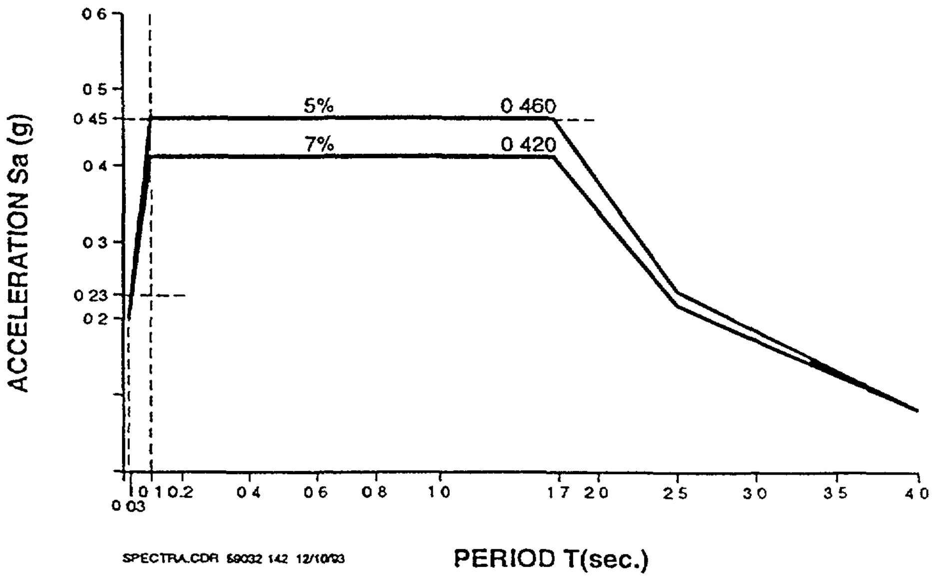

Figure 3 Kozloduy NPP Site Specific Response Spectrum

2 UPGRADING CONCEPTS

Two upgrading concepts were evaluated on the basis of several factors - one based on cast in situ reinforced concrete shear walls along the end walls of the building and the other based on new vertical steel braces A key factor for the selection of upgrading concept was the client's requirement for quick and easy for implementation building upgrades that will impose no breaks in the technological process EQE-International defined acceptance criteria based on the USNRC and UBC requirements for analyses, member and connections detailing [2]. Relative construction costs were considered as well as the client's requirements for the building upgrading schedule Walkdown notes were used as a base to assess the applicability of the proposed strengthening concepts Finally, the main factor considered was preventing the collapse of the hall structure and the travelling cranes on the fuel storage area during and after an SSE.

The acceptance criteria for design was that structural elements must have capacity to resist the combined effect of gravity and earthquake loads. This is expressed as the ratio of demand forces to capacity forces of the structural elements of the lateral load-resisting system, which is called Inelastic Demand Ratio (IDR). The inelastic demand ratios used for the assessment of existing elements and for the design of new ones in this case are shown in Table 1

The strengthening concept based on new steel braces was chosen The general advantage of the steel structure upgrading scheme is that the construction work will be carried out outside of the spent fuel pools area.

The main upgrading elements in the transverse direction are new external steel braces supporting the columns of the hall, which are attached down to the auxiliary building, Figure 4 It was estimated that their assembly and erection will be possible and comparatively easy on the roof of the auxiliary building. Bracing upgrades are also made between the columns on axis 1 On axis 7 at the transactional area between the high-bay and low-bay roofs a new steel truss is arranged, Figure 2.

In the longitudinal direction, vertical steel braces along rows B and Г are arranged, Figure 5 Doubled braces are used at the high-bay part of the building to resist the longitudinal seismic loads

Reinforced concrete shear walls will also be built up in different locations in the auxiliary building and in the inner structure of the hall, Figure 6.

A decision was made to remove the facing precast concrete panels on axis 7 between rows B and Г above the low-bay roof, because they can fall down directly on the spent fuel pools through the low-bay roof All heavy facing panels along row Г between axes 1 to 7 above Elevation 24 00m

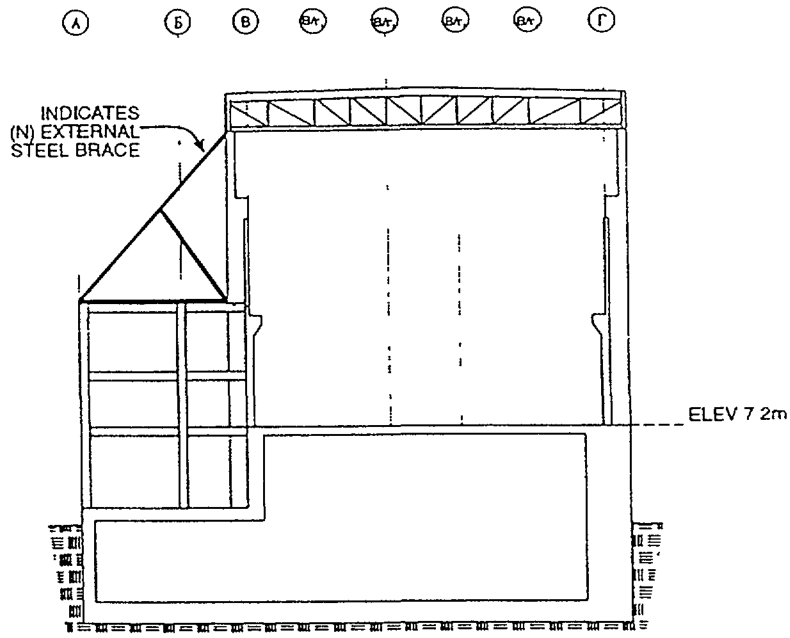

Figure 4 Transverse External Steel Brace at High-Bay Section

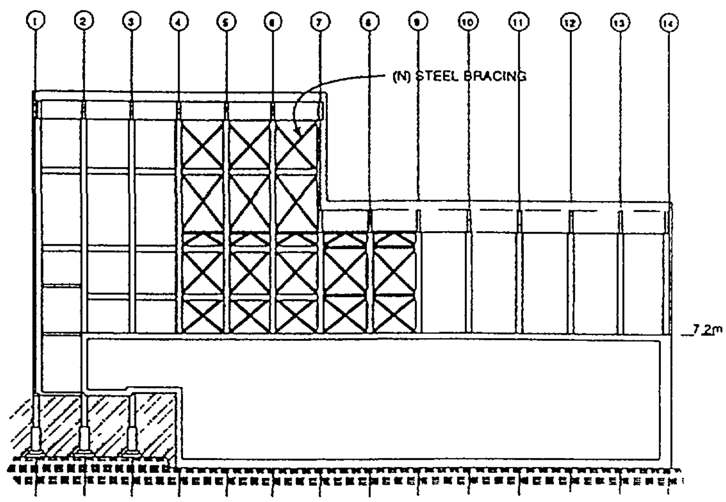

Figure 5 Longitudinal Vertical Steel Braces

Table 1 Inelastic Demand Rations (IDR)

Building System	Elements	Inelastic Demand Ratios	Notes
Steel Braced Frames	Diagonals Precast Columns Precast Beams Braces	3 1 1 1	only tension
Concrete Walls	Shear Flexure	1 5 1 75	
External Braces	Struts Connections	1 1	

will be removed also, in order to decrease some of the masses on the higher levels and thus to reduce the stress in the lateral resisting systems' elements induced by seismic loads. All removed heavy concrete panels are to be substituted by lightweight panels made of corrugated steel sheets and polyurethane thermal insulation.

Figures 6 and 7 show the location of the new work and main strengthening elements. The actual structural element locations were finalised during the detailed analysis and final design phase.

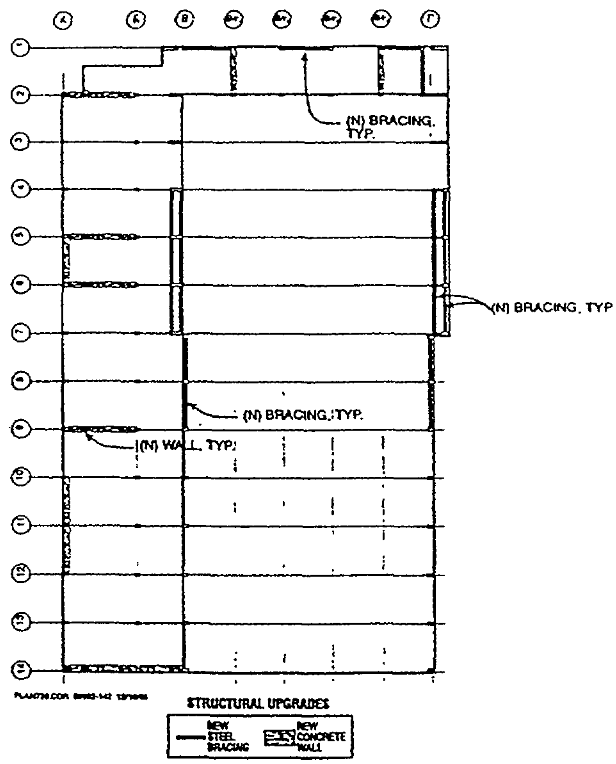

Figure 6 Plan of Structural Upgrades at Elevation 7.2m

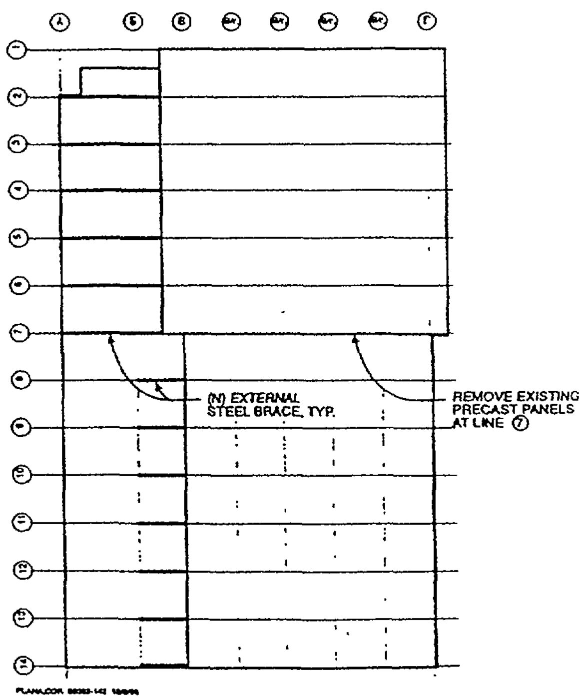

Figure 7 Plan of Structural Upgrades at Roof Level

3 CREATING A 3-D FINITE ELEMENT MODEL FOR FINAL DESIGN

A three dimensional SAP90 model was created for the investigation of the seismic response of the existing structure and for the design of the building upgrading. A seismic analyst, structural and civil engineering team was formed and the benefits of their joint effort were utilised to accelerate the process of analysis, member sizing and connection details design, [3].

3.1 INVESTIGATION ON THE STRUCTURAL BEHAVIOUR

Investigations of different elements of the model were carried out with the goal to develop envelope of the seismic forces. Different configurations of the external steel braces were investigated. Two lateral supporting sets of elements were finally chosen - one against heads of the high-bay columns and one against columns transaction on Elevation 27.75m. For these braces the inelastic demand ratio was defined equal to 1.0, Table 1.

The longitudinal vertical bracing along columns on rows B and Γ was arranged to span 2-4 bays. The investigation was carried out to identify the most reasonable span of bracing. The three spans braced were chosen. After that the moment resistance of the longitudinal high bay girders for rotations around vertical axis was investigated. It was decided to put shear force transferring elements on the girders at Elevation 27.75m to make possible resistance through vertical braces of the transverse seismic loads also.

The modelling of the heavy travelling crane and its sub-crane structure was one of the key points of this work. Its mass can develop very large seismic forces, but it cannot cause movements of the supporting nodes in opposite directions, so the crane also supports the lateral resistance of the adjacent columns by distributing the forces to more than two pairs of columns.

The crane was modelled in two manners. The first was to put beam type elements between columns on the corresponding axes. In the second, more precisely, the crane beams, crane car and sub-crane ways were modelled. The influence of the crane location along the high-bay hall was also investigated. Three principal locations were assumed: between axes 1-3, 3-5, 5-7. The critical crane location for lateral bracing elements was found to be between axes 5 to 7 where it is often located during operation.

The sub-crane structure transfers vertical and horizontal transversal and longitudinal crane loads. For transversal horizontal and seismic loads, the sub-crane structures are vulnerable. That is why they were proposed to be upgraded to a box type steel frame structure. The detailed modelling of the crane was done mainly for the purposes of evaluating the sub-crane structure upgrading [4].

The modelling of masses took in consideration the real gravity loads distribution, established as a result of the walkdown. Numerous runs were done improving the mass distribution, just to make it closer to reality and, at the same time, reasonable to be processed. It was found during the investigation that panel masses on the outer columns on row Γ above Elevation 24.00m and axis 14 above Elevation 18.00m induce very large seismic forces. It was decided to remove the heavy panels there and replace them with lightweight ones, as mentioned above. The effect of the masses on the building cladding sides was investigated also and was decided that the panels can be left there.

3.2 FINAL RUN

As a result of the above mentioned investigations, the SAP90 model was refined and tuned to properly assess the seismic response of the upgraded structure, Figure 8. Upper limit member seismic forces were assessed from the analyses.

The final runs were carried out with 5% site specific response spectra. After that deconvoluted 5% response spectra down to the foundation level were used as input for comparative study. The deconvoluted response spectra were prepared using SHAKE'91 code analysis [5]. The benefit of using this seismic input motion was reduction of approximately 10% of the member seismic forces which did not practically affect the new sized braces and upgrades of the existing elements.

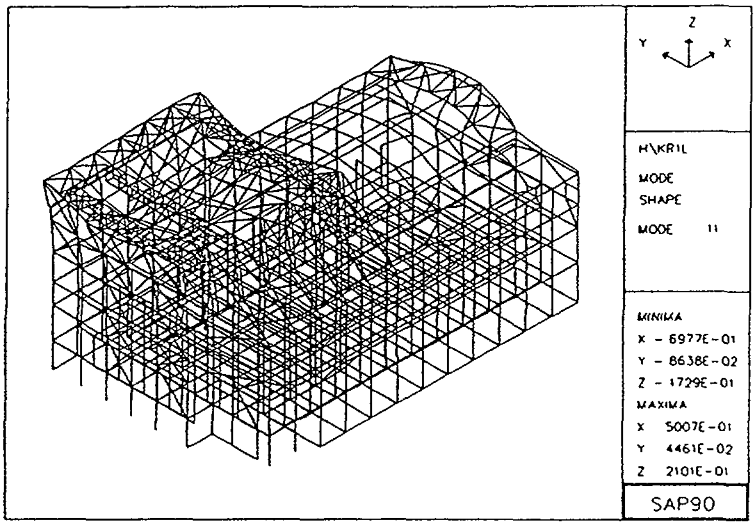

Figure 8 SAP90 Structural Model

3.3 LOADING COMBINATIONS

The principal load combination used was:

DL + 0,25.LL + CRL + E

DL - dead load (includes equipment loads);
LL - live load (includes snow loads);
CRL - crane loads;
E - seismic loads.

3.4 SIZING OF NEW ELEMENTS

The sizing of new bracing elements was carried out. The sizing calculations were done according to the Bulgarian Code [6]. The steel profiles for the upgrading elements were chosen from the available on the Bulgarian market. The new shear walls were sized to transfer the seismic forces to the monolithic foundations [7].

3.5 CONNECTIONS DESIGN

The connections were carefully designed according to the requirements of UBC [8] and Bulgarian Codes [6,7]. The connections of the upgrading elements must insure the full load transfer from the connected elements to the adjacent supporting structure, Figure 9. At the same time, the main upgrading elements must insure the bearing capacity of the main structure under combined loads. The ties between new and existing elements were made by strips, anchor bolts passing through reinforced concrete elements and anchor bolts with epoxy clay. Application of Bulgarian welding consumables and increased quality control of the welds were proposed.

4 CONCLUSIONS

The experience in analysing and upgrading of such a complex industrial structure, comprised of elements with substantial differences in material, rigidity, construction and general behaviour, showed some interesting conclusions, as follows:

- It is always worth considering several alternative concepts for seismic upgrading and strengthening of complex structures. Some of the conceptual upgrades may not work as expected in the stage of detailed design and others may even come out to be applicable.

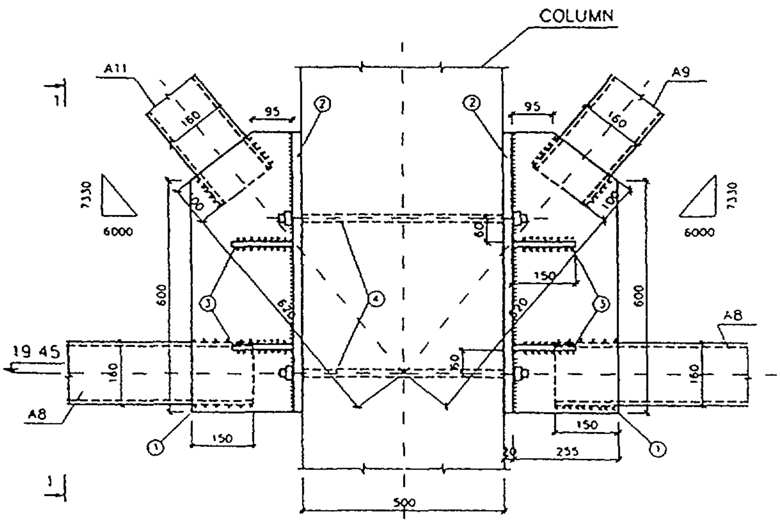

Figure 9 Typical Connection Detail

- It is important to take into account the structure of heavy bridge travelling cranes when making a 3D model of an industrial structure for seismic analysis, because in this way:
  - more realistic assessment of the structural behaviour is achieved;
  - the sub-crane structures can be analysed precisely and upgraded adequately;
  - assessment of the crane-building structure interaction during seismic event can be done and conclusions for the seismic qualification of the crane structure can be drawn;
- The development of detailed models of complex building structures should be done very carefully. Complex models may come out to be very sensitive when conducting spectral seismic analysis. Extremely important is the mass distribution which should be made close to reality and, at the same time, reasonable to be processed.
- The detailed design of connections may sometimes result in changing of the whole upgrading concept when there is lack of space to construct the connection or the adjacent structural member has no sufficient capacity or adequate configuration.

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CAPACITY ASSESSMENT OF THE CONTAINMENT STRUCTURE OF UNITS 5 AND 6 AT KOZLODUY NPP

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XA0100509

Abstract

The containment structure of the reactor building is made of a post-stressed concrete shell with a steel liner. The post tension cables (tendons) are anchored in a stiff ring girder at the junction of the cylinder shell with spherical dome. The wall thickness is variable for different zones of the cylinder and the dome.

Two finite element models are developed to study the structural behaviour of the containment. The first one is composed as a stick model and is used for seismic response analysis of the containment structure including the effects of soil-structure interaction. The second one is detailed finite element shell model of the containment including inclined arrangement of the prestressing cables. It is used for the study of linear and non-linear static and dynamic responses of the containment under loads due to normal operation, additional loads due to the anticipated operational occurrences and some additional loads due to accident conditions.

Assessment of the bearing capacity of the structure is done along with a study of failure modes in critical load combinations.

The evaluation of the prestressing of the containment is made by investigating the prestressing technology, as well as the on-line scanning of the prestressing using embedded sensors and annual verification of prestressing by control of tendon stresses during operation of the unit. Comparison of this evaluations with the finite element model analyses results will help to tune the model and assess the reliability of the non-destructive control / monitoring system of the containment.

1. INTRODUCTION

The objective of the study is to provide assessment of the structural behaviour and safety capacity of the WWER-1000 MW Reactor Building Containment at Kozloduy NPP under critical combination of loads according to the current international requirements. The analysis is focused on a realistic assessment of the Containment taking into account the non-linear shell behaviour of the pre-stressed reinforced concrete structure. Previous assessments of the status of pre-stressing cables pointed out that the efficiency of the Containment as a final defence barrier for internal and external events depends on their reliability. Due to this, the experimental data obtained from embedded sensors (gauges) at pre-stressed shell structure is to be compared with the results from analytical investigations. The reliability of the WWER-1000 MW accident prevention system is under evaluation in the project.

The Soviet standard design WWER-1000 MW type units installed at Kozloduy NPP were originally designed for a Safe Shutdown Earthquake (SSE) with a peak ground acceleration (PGA) of 0.1g. The new site seismicity studies revealed that the seismic hazard for the site significantly exceeds the originally estimated and a Review Level Earthquake (RLE) anchored to PGA=0.20g was proposed for re-assessment of the structures and equipment at Kozloduy NPP [1].

Comparison between the Russian design requirements and the international regulations was performed. Additionally, an investigation of the pre-stressing technology and the annual control of the cables' pre-stressing of the Containment is carried out. The crane influence on the dynamic behaviour of the Containment will be done as well as a study of the integrity of the Containment as a final defence barrier.

2. DESCRIPTION OF THE STUDY AND THE EXPERIMENTAL METHODS USED

EQE-Bulgaria undertook the following activities to fulfil the scope of the study

- Finite element modelling of the WWER-1000 MW Reactor Containment of Kozloduy NPP,
- Static and dynamic analyses including soil-structure interaction assessment,
- Strength and failure mode analyses of the Reactor Containment including reliability assessment of cable pre-stressing technology

These activities were aimed at efficient analyses of the structural behaviour and integrity of the Containment as well as at assessment of the reliability of the cables pre-stressing The analyses give a deeper insight of the Reactor Containment safety

2.1 Description of the Containment Structure

The Containment structure of the Reactor Building is a pre-stressed concrete shell with a steel liner. The post tension cables (tendons) are anchored in a stiff ring girder at the junction of the cylinder shell with the spherical dome The wall thickness is variable for the different zones of the cylinder and the dome

The heavy polar crane is located close to the upper part of the shell The general view of the cross-section and plan of the WWER-1000 MW Reactor Building are shown in Figures 2 1 and 2 2 , respectively

2.2 Development of the Containment Structure 3-D Models

Two finite element models were developed to study the structural behaviour of the Containment·

- A preliminary 3-D shell model of the Containment using SAP90 program for linear analyses,
- A detailed 3-D coupled model of the Reactor Building using COSMOS/M program for non-linear analyses

The first model was used for preliminary static loads analyses of the Containment for an assessment of the loading conditions effects on the structure integrity The second model is a detailed finite element coupled shell and stick model of the Reactor Building It is used for studying of linear and non-linear static and dynamic responses of the Reactor Building from the loads due to normal operation, additional loads due to the anticipated operational occurrences and some additional loads due to accident conditions

2.3 Static and Dynamic Analyses Including the Soil-Structure Interaction

The dynamic analysis was carried out in two phases

1) A stick model was developed in order to assess the seismic response of the Reactor building including soil-structure interaction

The main objective was to evaluate the seismic response of the Reactor Building, based on data available for soil properties, site seismicity, free field ground motion, and structure design State-of-the-art Soil-Structure Interaction (SSI) analysis of the Reactor Building complex was conducted to assess the effect of greater than designed for seismic event

2) A detailed 3-D coupled shell and stick model of the Reactor building was developed

The model was used for static and dynamic analyses of the Containment The internal forces due to seismic response and the static load conditions were defined

2.4 Strength and Failure Mode Analyses of the Reactor Containment

The capacity check of the Containment shell was done in accordance with the current Bulgarian design codes and the IAEA recommendations and guidelines Pre-stressed reinforced concrete Containment structures have large seismic margins above the SSE level because they have been designed for a combined SSE and loss of coolant accident. Shear, flexural and bond failure modes were postulated and analysed. This study concentrated on concrete cracks development and flexural failure modes analyses

3. WORK CARRIED OUT

3.1 Data Collection

The following information was collected for the modelling and the analyses of the Containment

- Soil profile,
- Detailed drawings of the Containment structure;
- Embedded gauges records since 1986;
- Data from the annual checks of the cables pre-stressing

3.2 General Assumptions

In order to obtain a preliminary picture of the Containment dynamic behaviour the following assumptions were accepted for the linear response analysis study:

- The Containment is a separate structure, lying on the thick plate at Elevation +13 20m, and it has independent dynamic behaviour for seismic loading

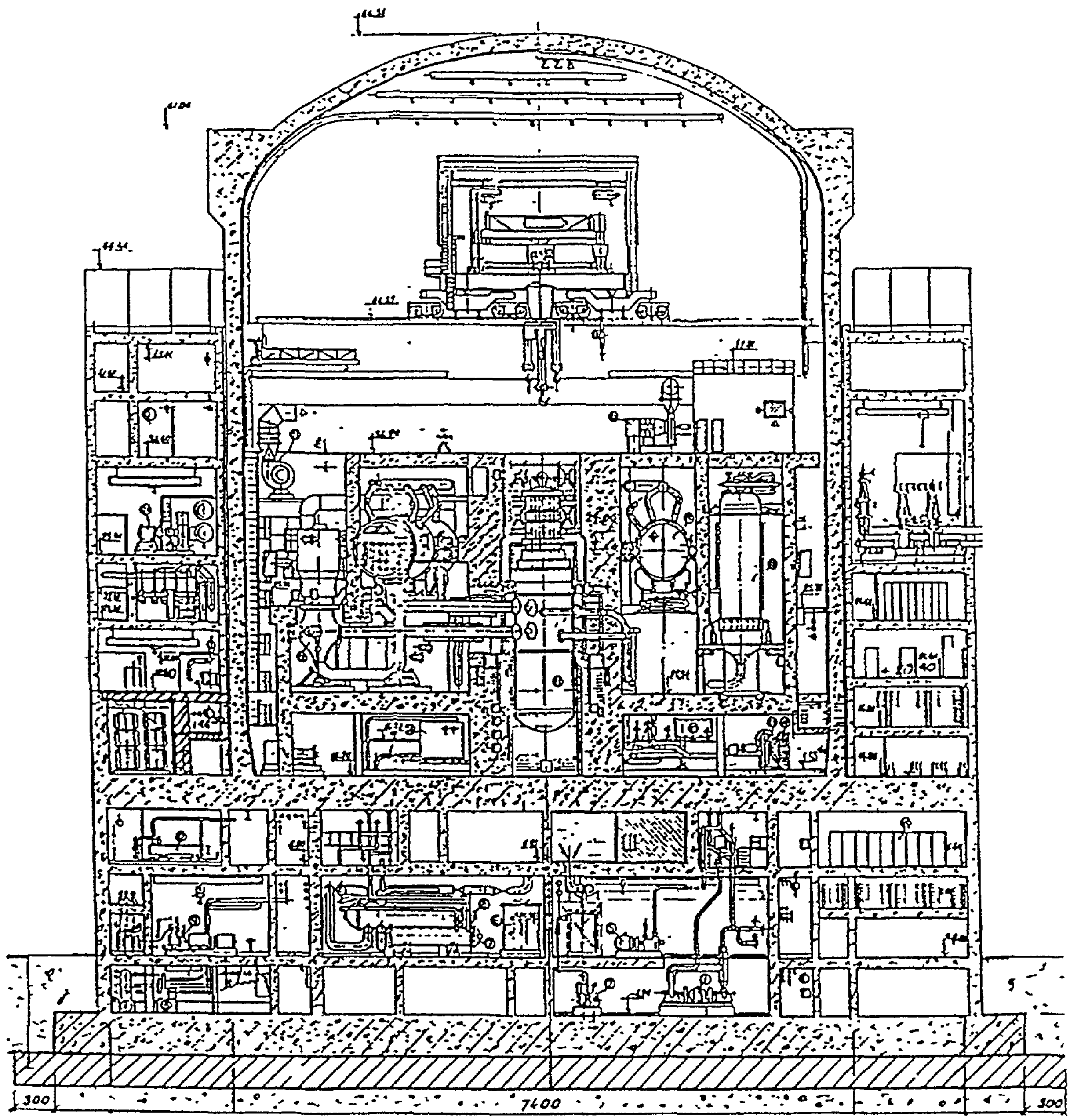


Fig 2 1 WWER-1000 MW Reactor Building Cross Section

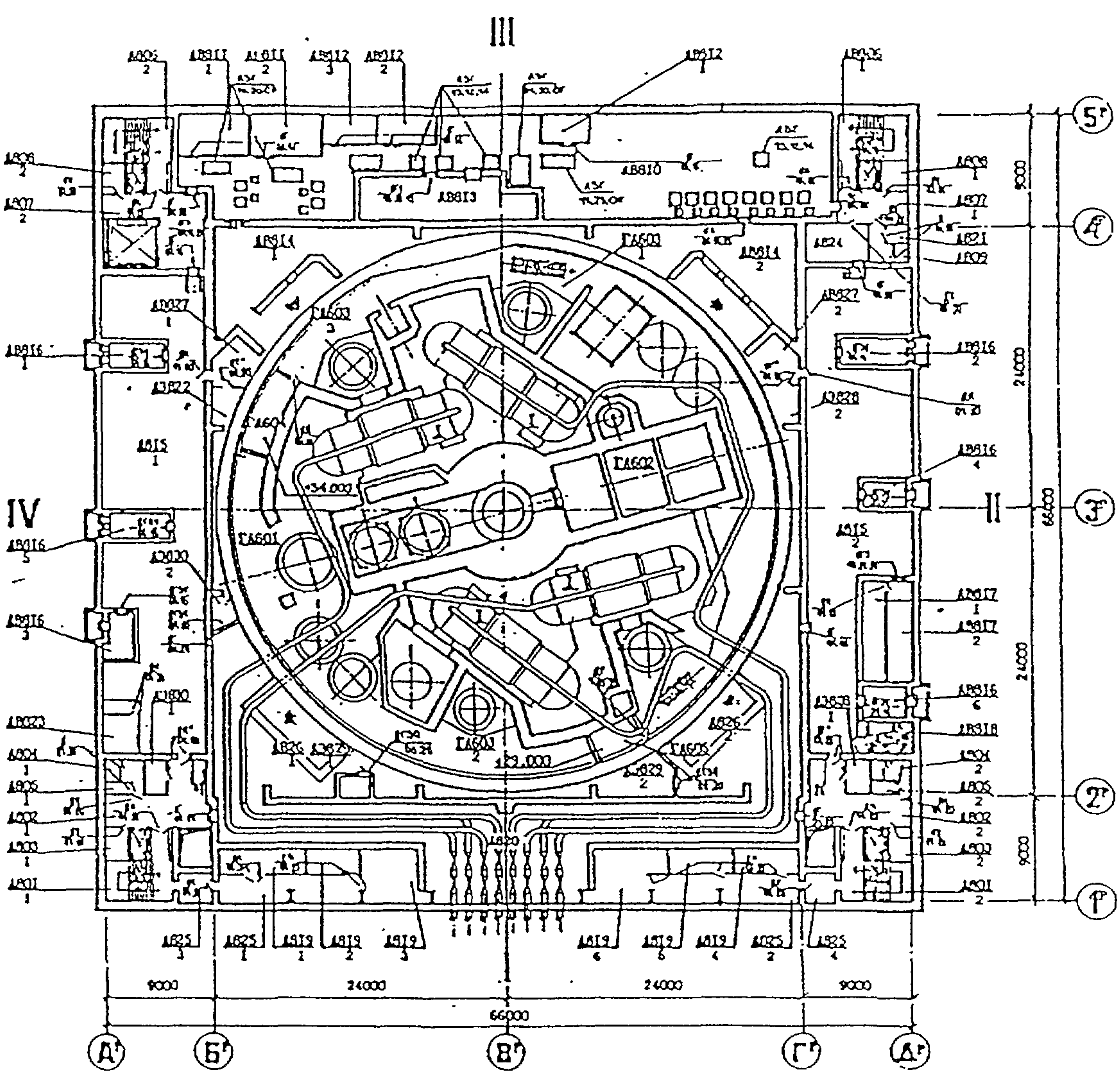


Fig. 2.2 Plan view of a WWER-1000 MW reactor building.

- The study of the Containment as a separate structure is accurate enough for the other loading conditions (dead load, temperature, pre-stressing, etc.). LOCA
- The soil-structure interaction effects are taken into account in the input floor response spectra.

3.3 Reactor Building Modelling

First phase

The model is composed of 3-D beam elements. It comprises equivalent stick models of the Reactor Building Containment, Substructure, Internal Structure and Outer Building Structure.

General view of the model is presented in Fig. 3.1.

Second phase

Detailed 3-D model of the Containment structure was developed using COSMOS program. A general view of the model is shown in Fig. 3.2. Due to the axial symmetry of the structure and the loading conditions, and the symmetry of seismic loads about vertical plane, only half of the Containment was modelled. 4-node thick shell elements and beam elements with 6 DOF per node were used. The elements have both membrane and bending characteristics. The model consists of 784 shell elements and 813 nodes. The elements are assumed to be isotropic with constant thickness.

They are arranged on the middle surface of the cylindrical shell and the spherical dome. The Containment model is constrained at the upper thick mat of the Substructure. The appropriate boundary conditions (restraints) were applied on the symmetry plane nodes. The horizontal thick plate at Elevation +13.20m was modelled by means of appropriate elastic springs. The influence of the reinforcement was taken into account by means of increasing the thickness of the cylindrical and spherical shell from 1.20m and 1.10m to 1.28m and 1.20m respectively.

Material non-linearity is considered for static loading condition. The characteristic curves are approximated by bilinear curves. The non-linear behaviour of the model is controlled by displacements. The initial modulus of elasticity of the pre-stressed reinforced concrete is assumed to be E=30000 MPa and Poisson's coefficient is ν=0.2.

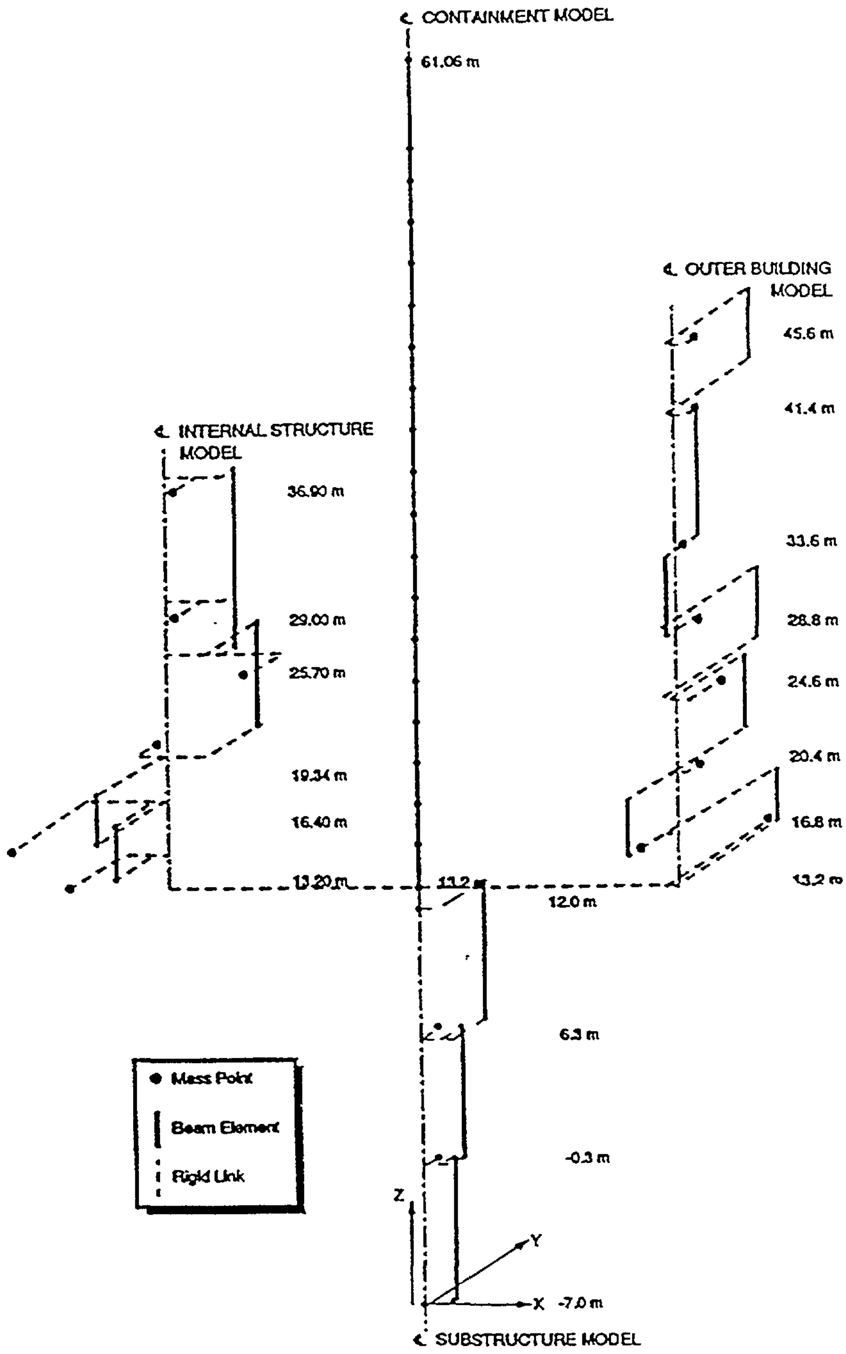


Fig. 3.1 Kozloduy 1000 MW Unit Beam Element Model

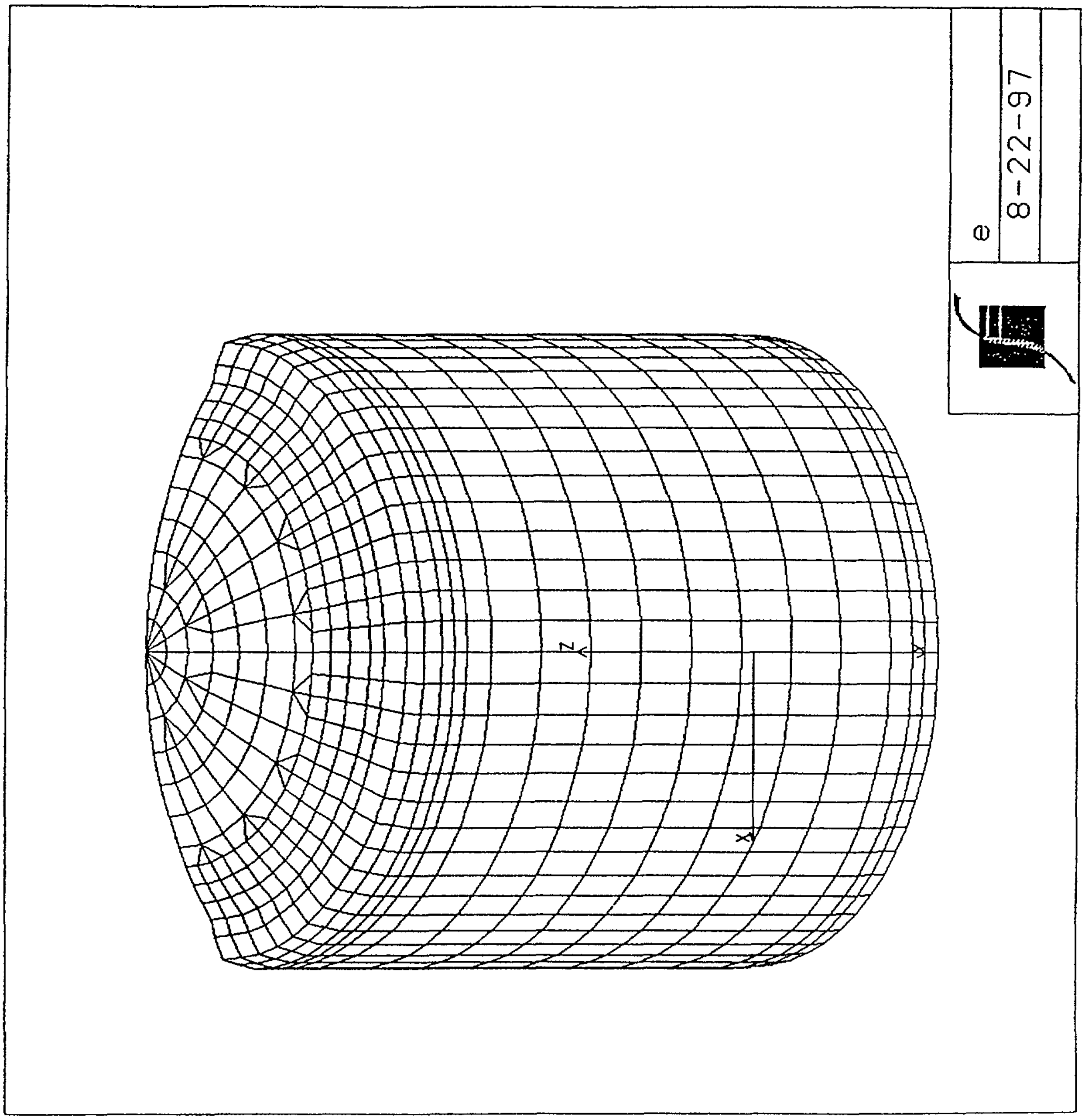


Fig. 3.2 3-D Shell Model of the Containment Structure

3.4 Loading Conditions

The loads considered in the study are:

- loads due to normal operation - dead loads, pre-stressing, crane loads, snow, live loads;
- additional loads due to the anticipated operational occurrences - internal pressure and temperature due to design bases accident, loads due to asymmetrical pre-stressing of the structures, sunshine influence;
- additional loads due to accident conditions - loss of coolant accident (LOCA), impact by pre-stressing tendon break.

The pre-stressing of the Containment is modelled by equivalent uniform pressure on the middle surface of the shell model varying in different locations. The values of the pressure vary in broad range, see Fig. 3.3. The pre-stressing of the cables can not cover the design requirements. The

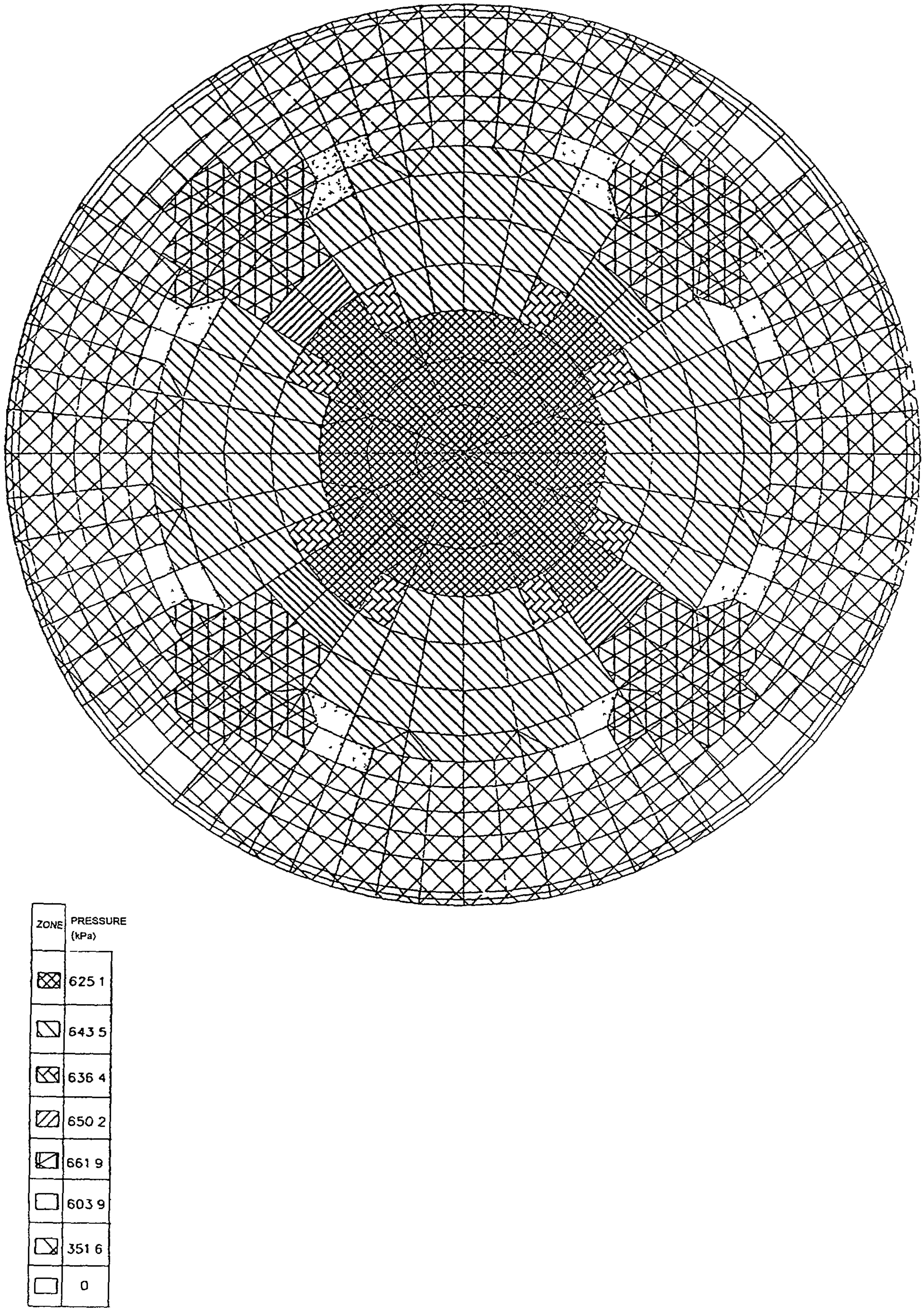


Fig 3 3 Pressure at the Spherical Shell Due to Cables Prestressing recommended value of 10000 kN pre-stressing force can not be reached during the annual pre-stressing checks. The mean values of the pre-stressing forces vary from 8500 kN to 9000 kN. A new system of pre-stressing cables is under development for changing of the existing cables.

An anticipated occurrence is a cable break during service period or during annual cables checks. This event may cause a shock impact on the Containment structure. Static loading of a cable breakage on the Containment was considered.

The LOCA is modelled by means of internal pressure with intensity of 474 kN/m^2.

3.5 Static and Dynamic Analyses

First phase

The seismic response analyses are based on both response spectrum method and time history response analyses. The seismic response analyses include the soil-structure interaction effects, obtained by the substructure method incorporated in the CLASSI chain of computer programs [2].

This part of the study was covered by splitting the study into following steps:

- Generation of Single Earthquake Matching Kozloduy Site Specific Response Spectrum
- Development of High Strain Soil Properties
- Reactor Building Impedance and Scattering Functions
- SSI Seismic Response Analyses
- Floor Response Spectra Generation

A fixed-base linear-elastic modal extraction analysis was performed to determine the eigenvalues and eigenvectors of the combined model. The model was analysed for seismic loading using the available envelop response spectra for the Foundation Reference Point (FRP) or corresponding time histories for different soil conditions. The time histories at FRP were obtained by SSI analysis of the Reactor Building for the site specific seismic input motion. The envelope response spectra were generated by enveloping and broadening of FRP response spectra for three different soil conditions: best estimate, lower and upper boundaries.

Time history and in-structure response seismic inputs at elevations -7.00 and +13.20 were obtained. Free field response spectra and in-structure response spectra at elevation +13.20 are given in Fig. 3.4 and Fig. 3.5.

Second phase

The eigenvalue and mode shapes extraction analysis was carried out for the 3-D shell model of the Containment. The first 30 eigen modes were obtained. A selection of them is given in Fig. 3.6 and Fig. 3.7. Mode shapes frequencies and mass participation coefficients are presented in Table 3.1.

The loading conditions were combined according to the current requirements. The bearing capacity of the Containment was assessed. Finally, the locations of the critical points and the possible failure modes were defined.

3.6 Experimental Data

The evaluation of the Containment pre-stressing was made by

- investigating the pre-stressing technology;
- on-line scanning of the pre-stressing based on embedded sensors data during unit operation;
- annual verification of cables pre-stressing by the control of tendon stresses.

Comparison of this evaluations with the finite element model analyses results helps to tune the model and assess the reliability of the non-destructive control and monitoring system of the Containment. value of the cable force should be 10000 kN. According to the Soviet design regulations a maximum decrease of 15% of this value is allowed. The pre-stressing forces are re-evaluated each 2-3 years. Meanwhile, a monitoring system is arranged using embedded gauges for prevention of unexpected occurrences. The reliability of this system is under discussion. The appropriate way to tune the gauges data is to compare the Containment stresses obtained from gauges with those obtained from annual cables checks. For this purpose it is needed to figure out analytically the stress-strain status of the Containment due to pre-stressing forces which are read

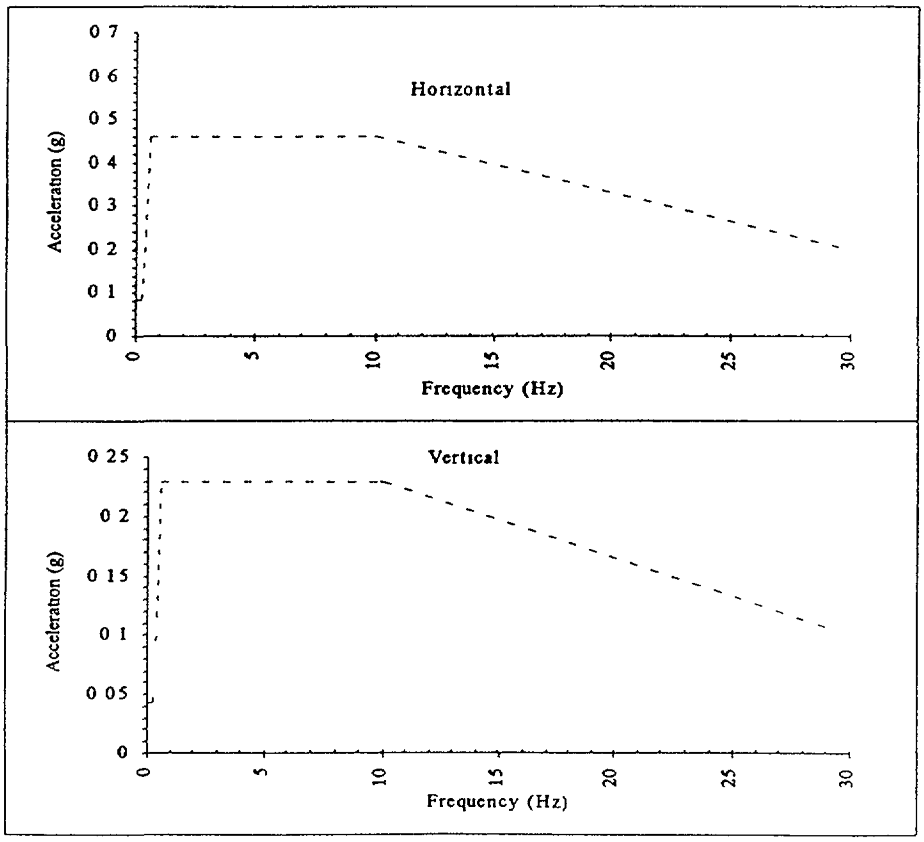


Fig 3 4 Free Field Response Spectra

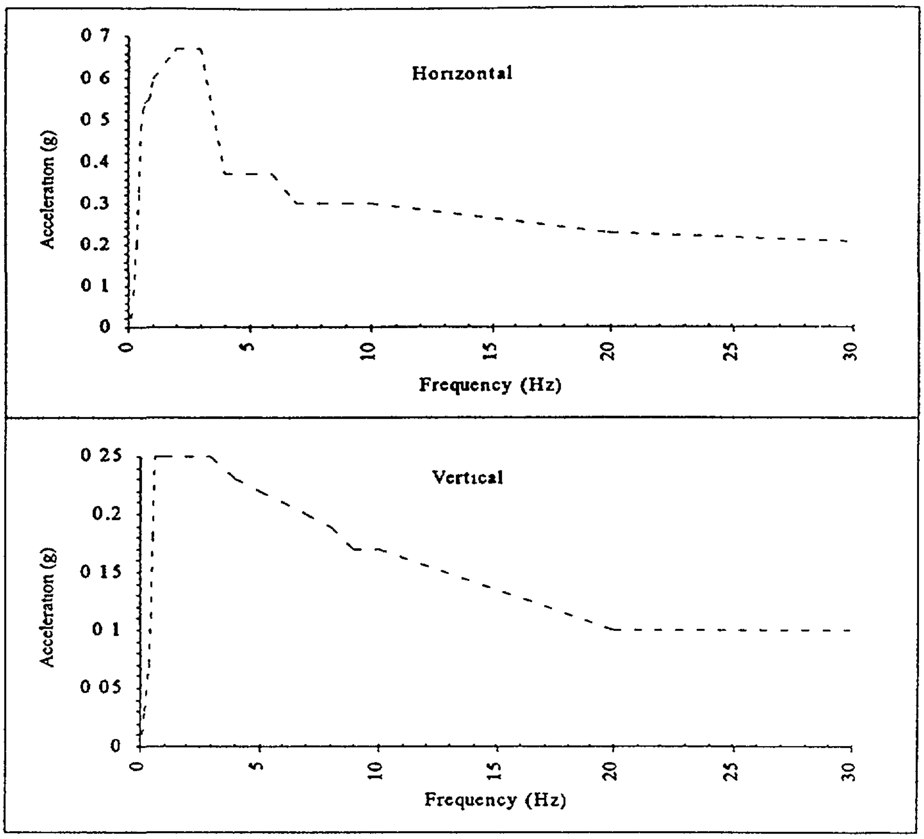
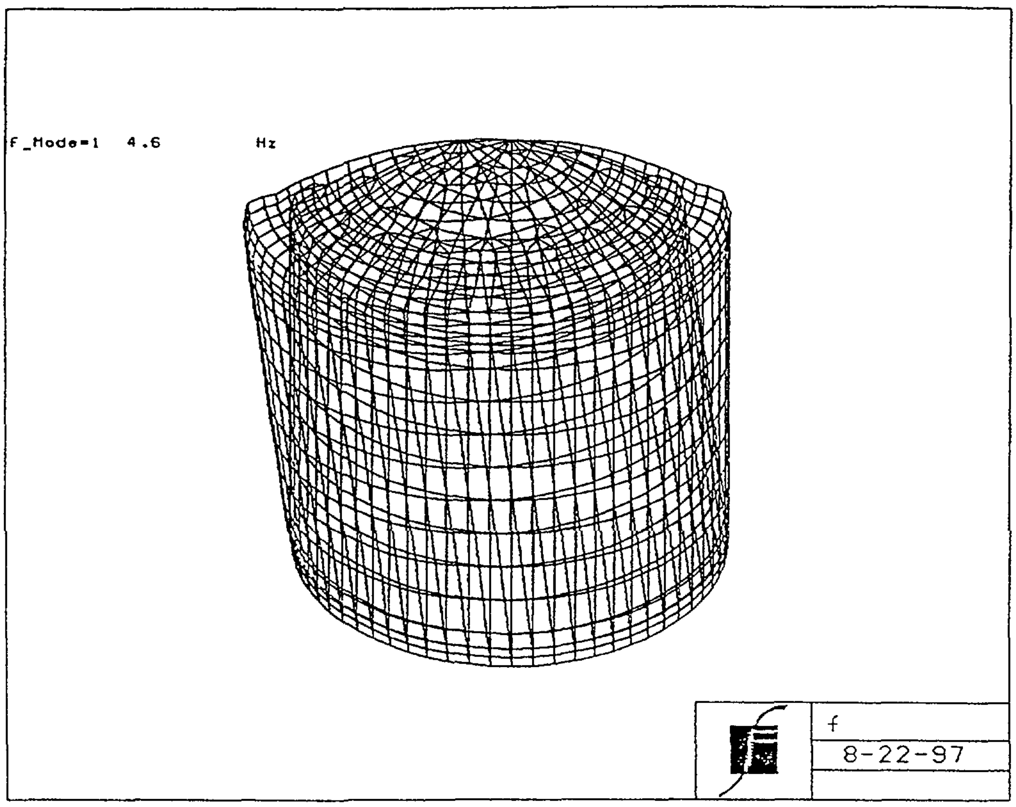


Fig 3 5 In-structure Response Spectra at Elevation +13 20



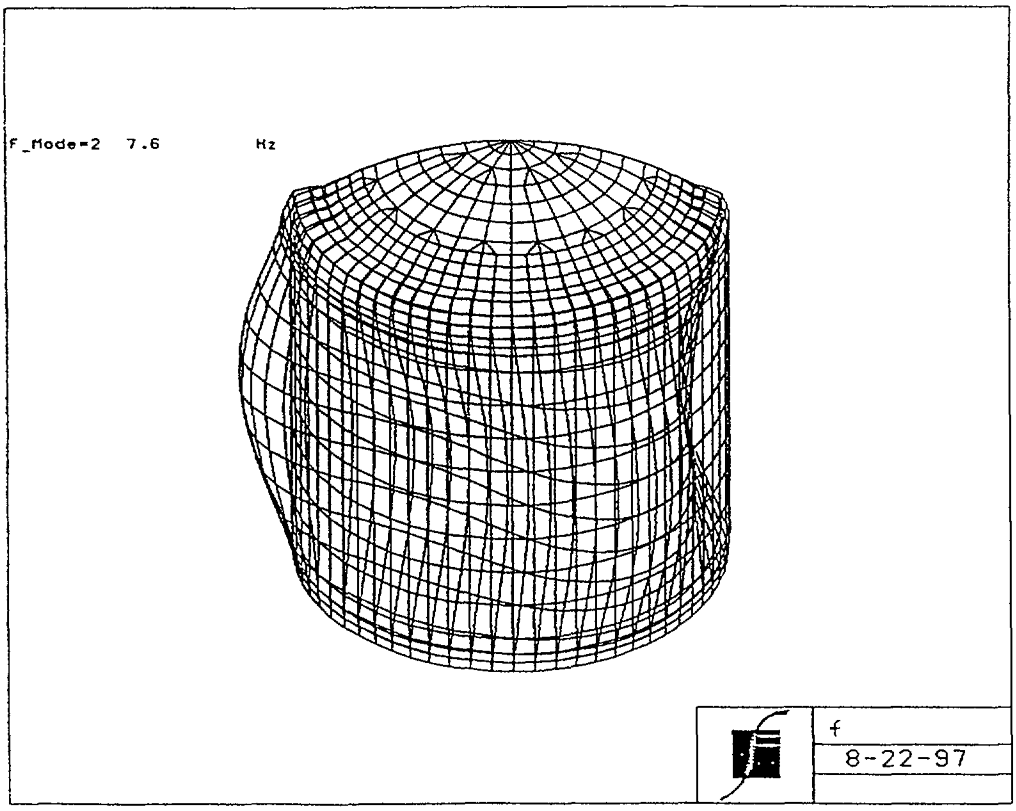
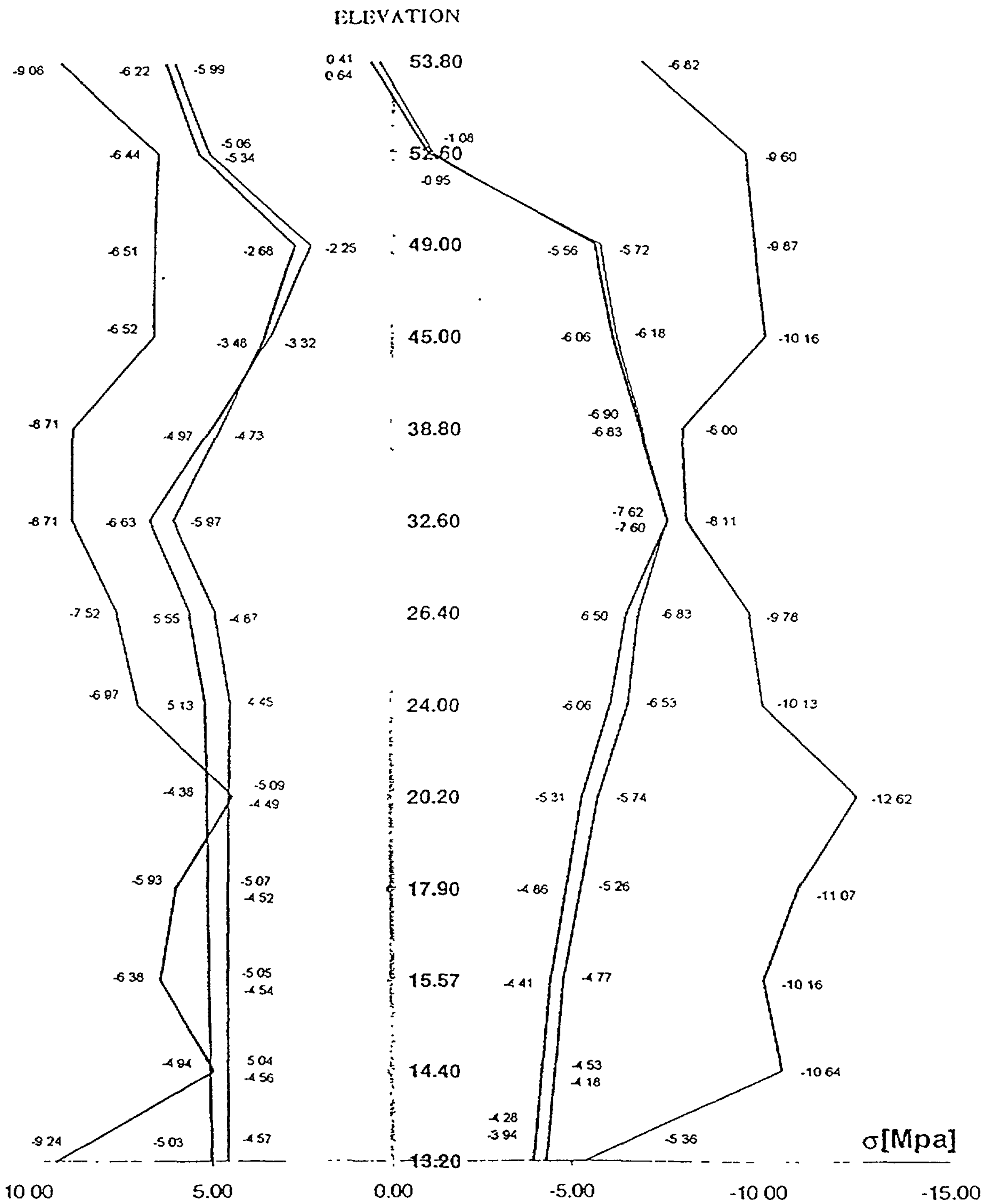
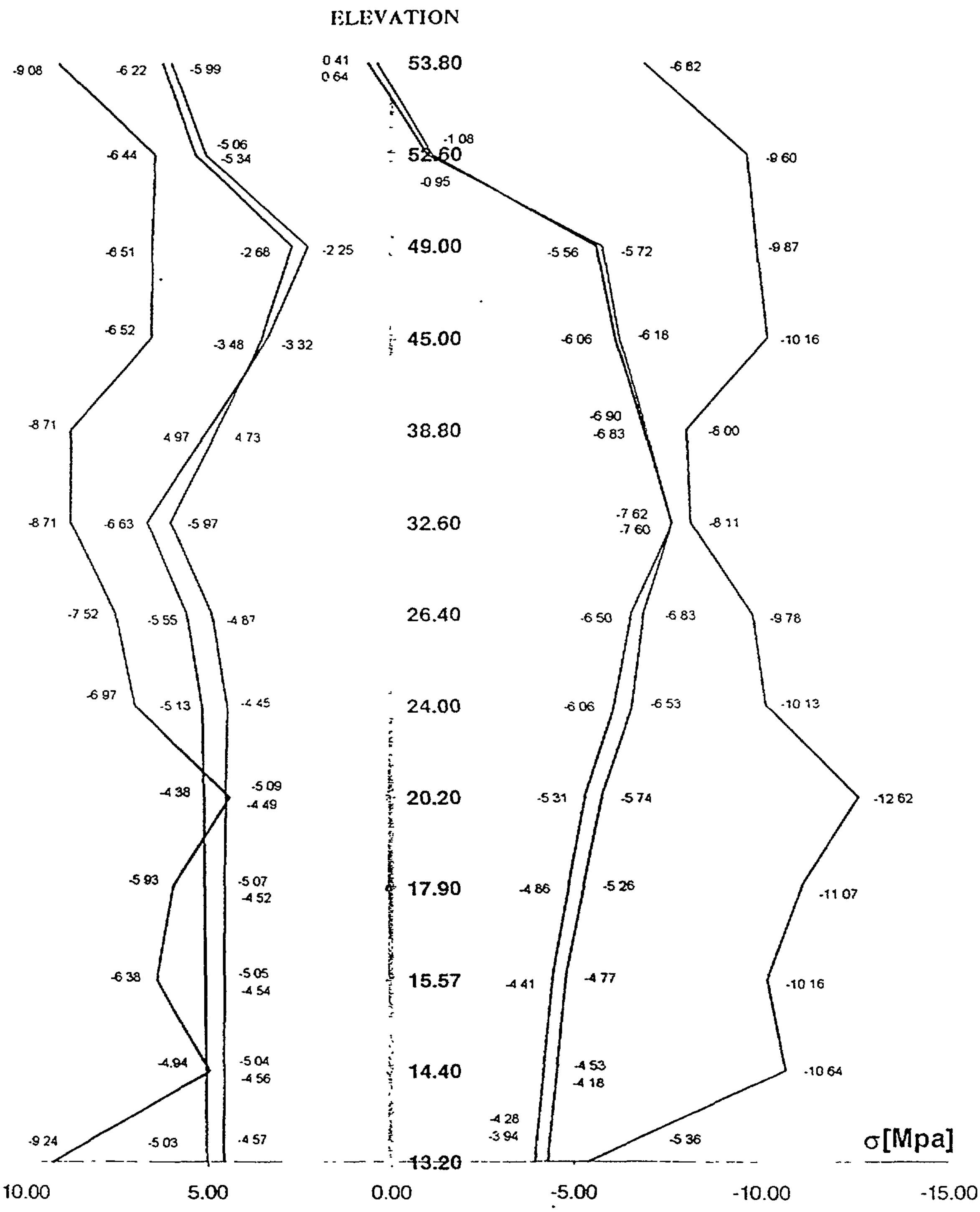


Fig. 3.6 Mode Shape 1, F1=4.6 Hz, Mode Shape 2, F2=7.6 Hz



- analytically calculated stresses at 100% pre-stressing
- stresses, obtained from gauges at initial pre-stressing
- stresses, obtained from gauges a year after initial pre-stressing

Fig. 3.7 Comparison of Stresses in Concrete Obtained from the Pre-stressing Monitoring and by the Analytical Study



- analytically calculated stresses at 100% pre-stressing
- stresses, obtained from gauges at initial pre-stressing
- stresses, obtained from gauges a year after initial pre-stressing

Fig. 3 8 Comparison of Stresses in Concrete Obtained from the Pre-stressing Monitoring and by the Analytical Study directly from monitoring system. Comparison of stresses in concrete obtained from the pre-stressing monitoring and by the analytical study are given in Fig. 3.7.

The reliability of the embedded monitoring gauges is controlled. Special procedures were developed and implemented. Unfortunately, an increasing number (more than 50%) of the gauges have failed in meeting the reliability criteria. An alternative monitoring method for prestressing assessment should be developed.

4. RESULTS AND CONCLUSIONS

Stresses on the inner and outer surface of the Containment were figured out.

The results of the presented study show the following:

- The critical load combination is dead load + prestessing + seismic forces. Forces in hoop and longitudinal direction due to dead load and prestessing are shown in Fig. 4.2 and Fig. 4.3.
- The compression stresses prevail. They reach values of 14.9 MPa at the spherical shell and 21.0 MPa at the cylindrical shell concrete sections. The allowable stress prescribed by the applicable code for this case is 17.0 MPa. It is exceeded by more than 23%.
- The maximum stress resulting from the combination dead load + prestessing is 17.3 MPa. In this case the allowable stress is exceeded by 1.7%.
- The maximum concrete tensile stresses occur above the stiffening ring and at the bottom of the cylindrical shell. These stresses have resulted from the non-uniform pre-stressing of the spherical shell. They reach the value of 4.7 MPa. The allowable stress in this case is 1.2 MPa. The calculated cracks width is 0.11 mm.
- The cracks depth is expected to be small and not to affect the pre-stressing cables.
- The breakage of one pre-stressing cable is not critical for the Confinement. The resulting forces in longitudinal and hoop direction due to cable breakage are presented in Fig. 4.1.
- The cables prestressing specified by the Russian designers is not reached.
- The load combination containing LOCA results in occurrence of tensile stresses throughout the concrete shell. Forces in longitudinal and hoop direction are shown in Fig. 4.4 and Fig. 4.5. Fortunately, the reinforced concrete has adequate resistance.

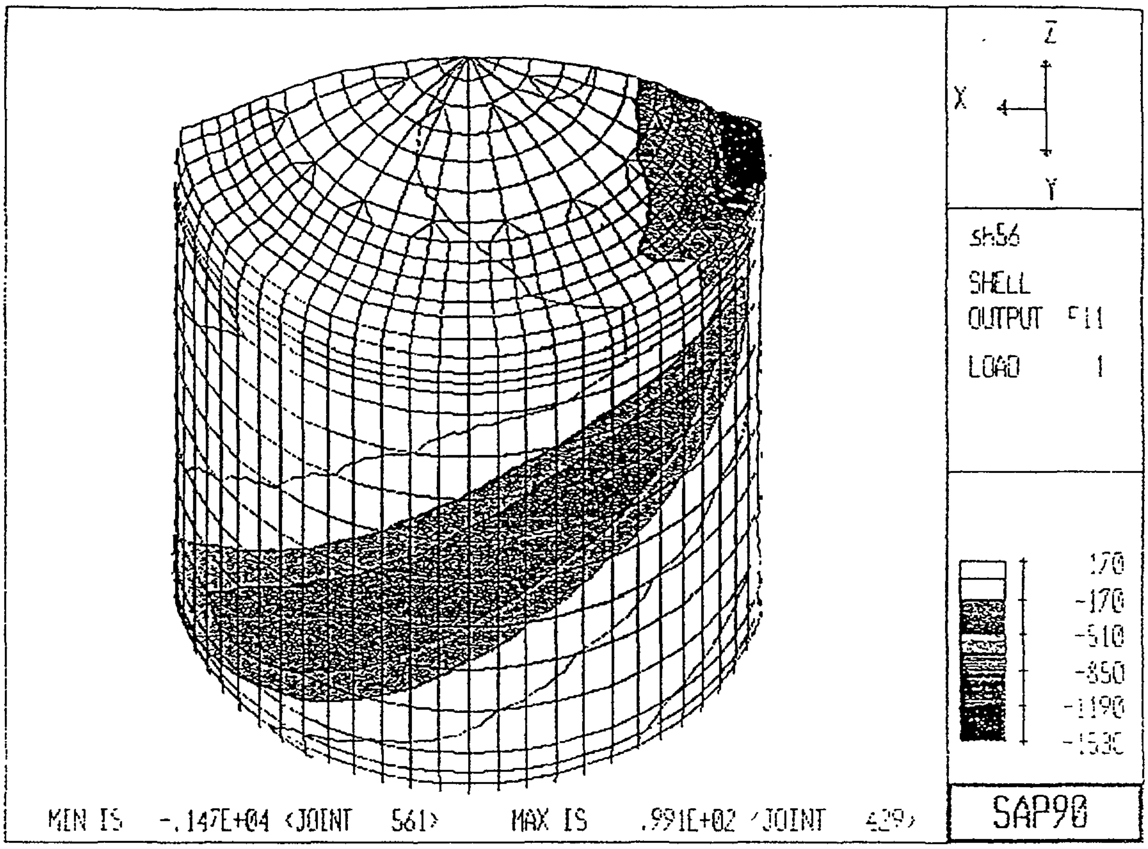
The static and dynamic analyses resulted in finding the margins of the Containment behaviour. Its preserving capabilities as a final defence barrier of the environment were assessed as satisfactory. The current requirements and regulations were met.

Additional prestressing of the cables is not recommended. It might lead to exceeding of the concrete allowable compressive strength.

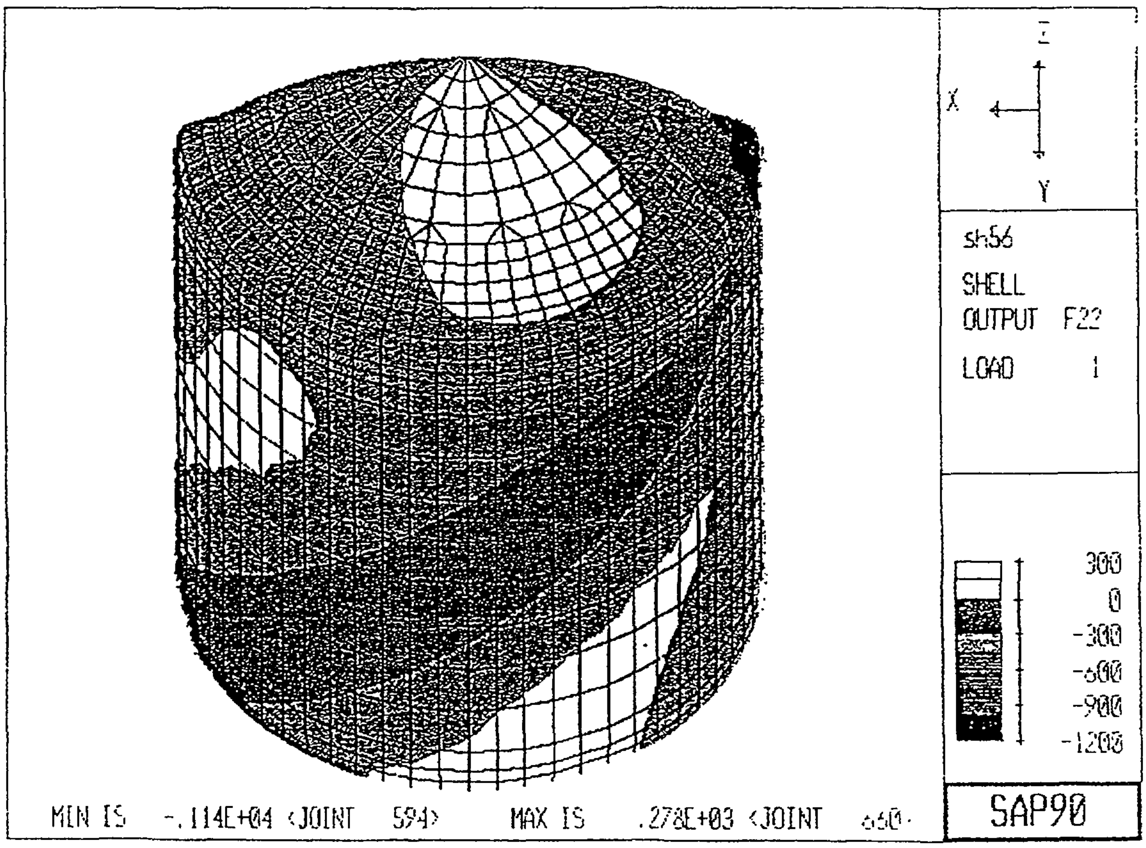
LOCA does not lead to significant increase of the stresses in the pre-stressing cables.

The critical points of the Containment are located above the stiffening ring and above the plate at elevation +13.20.

The reliability of the monitoring system is decreasing. Alternative monitoring procedures should be developed.

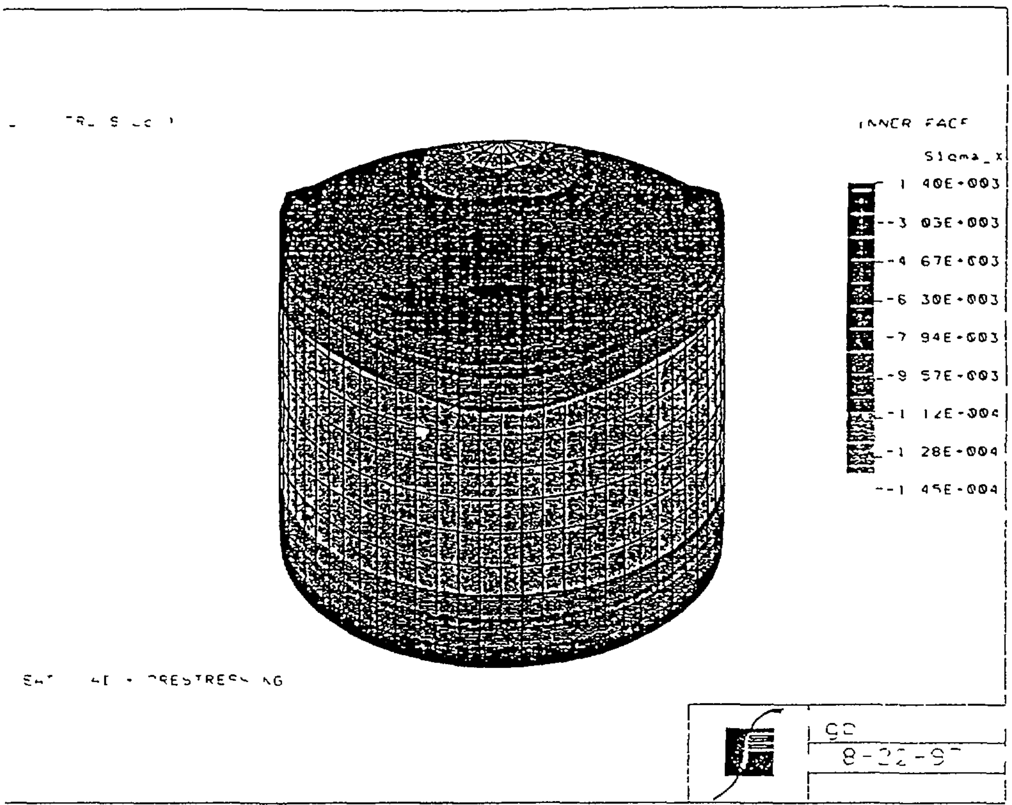


Hoop Direction



Longitudinal Direction

Fig. 4.1 Forces in Longitudinal and Hoop Direction Due to Cable Breakage



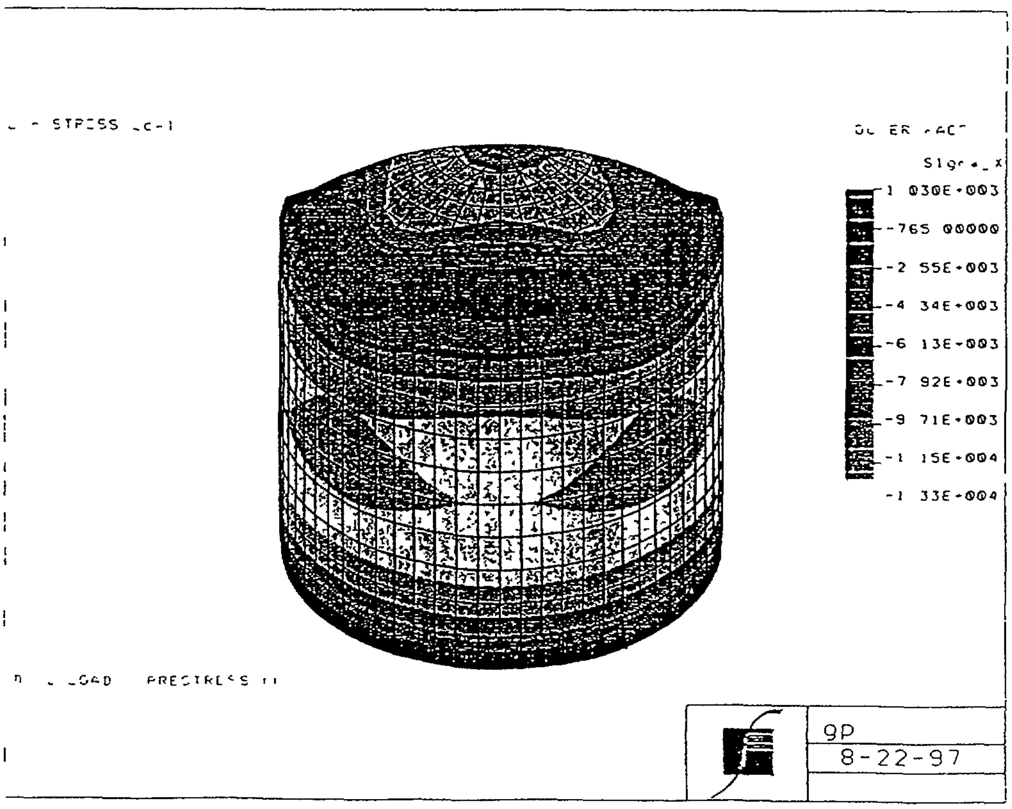


Fig 4 2 Hoop Stresses at the Containment Due to Dead Load and Pre-stressing

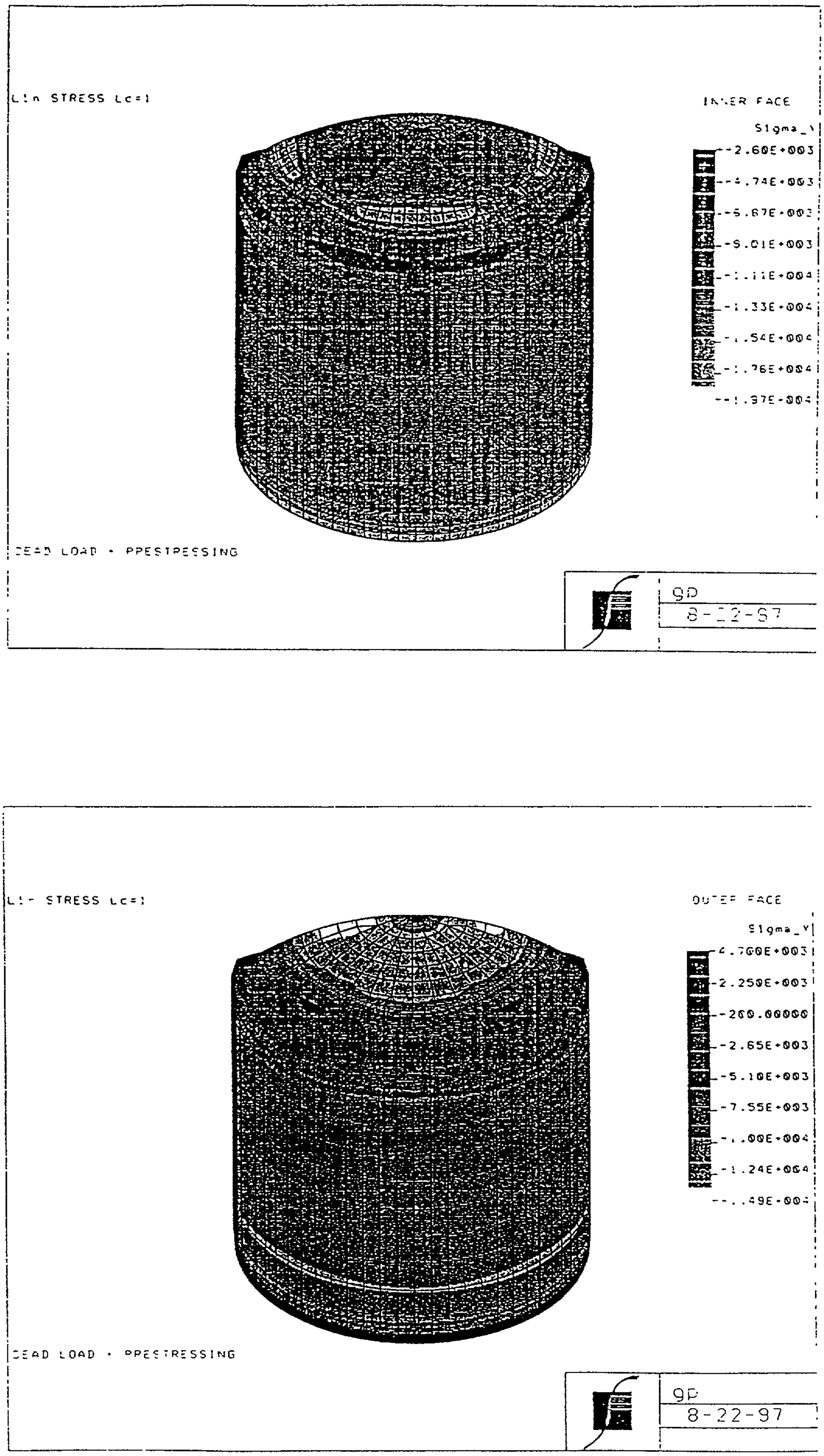
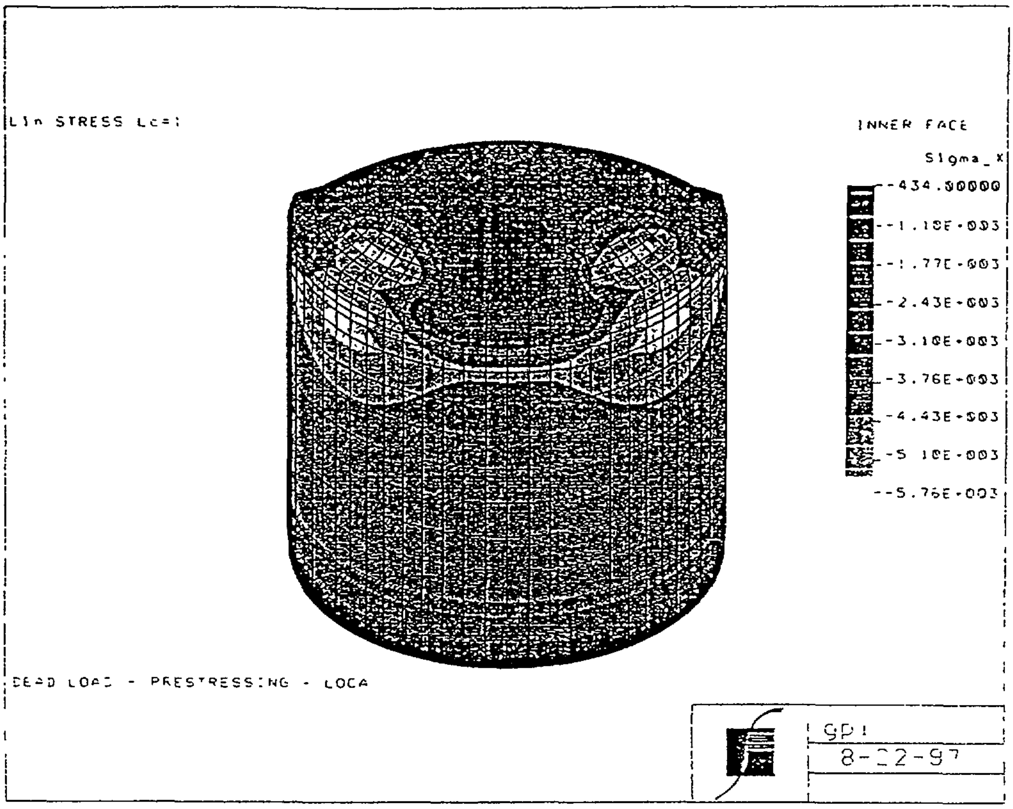


Fig. 4.3 Longitudinal Stresses at the Containment Due to Dead Load and Pre-stressing



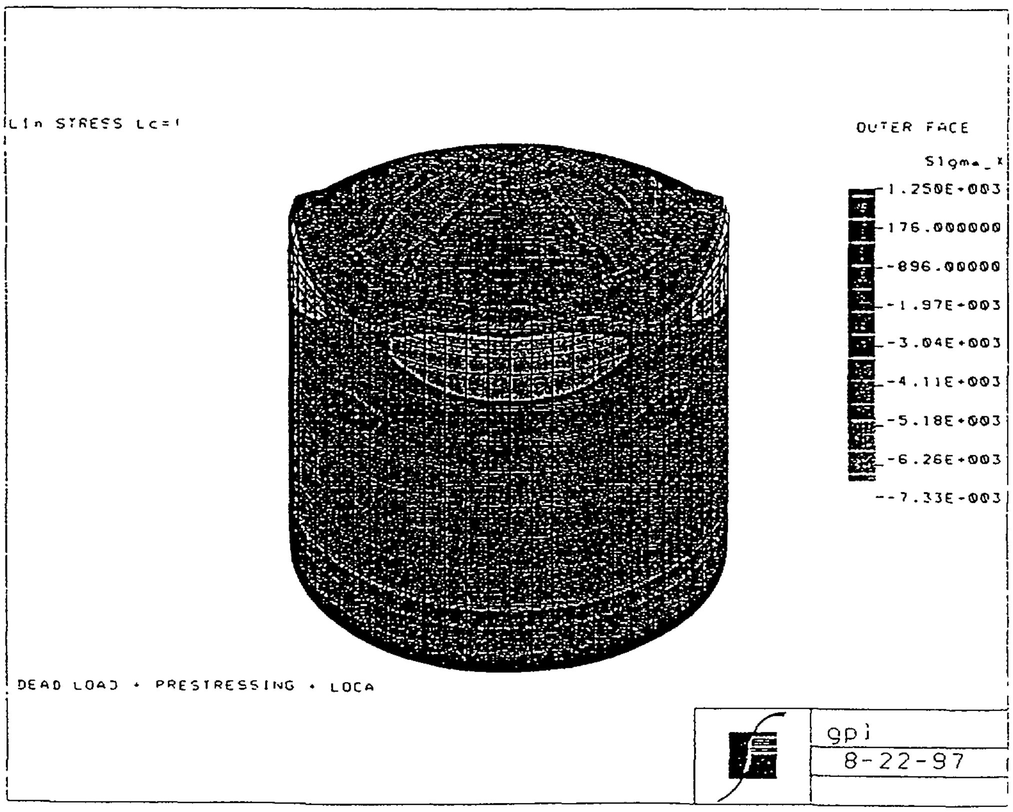
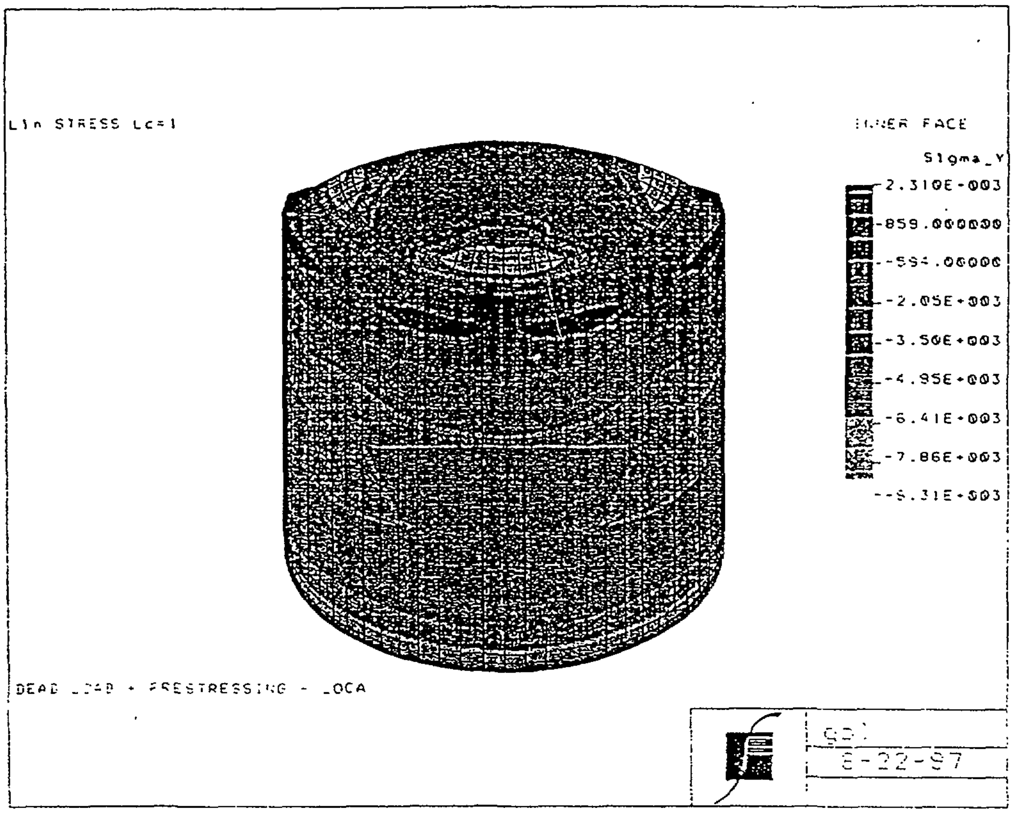


Fig. 4.4 Hoop Stresses at the Containment Due to Dead Load, Pre-stressing and LOCA



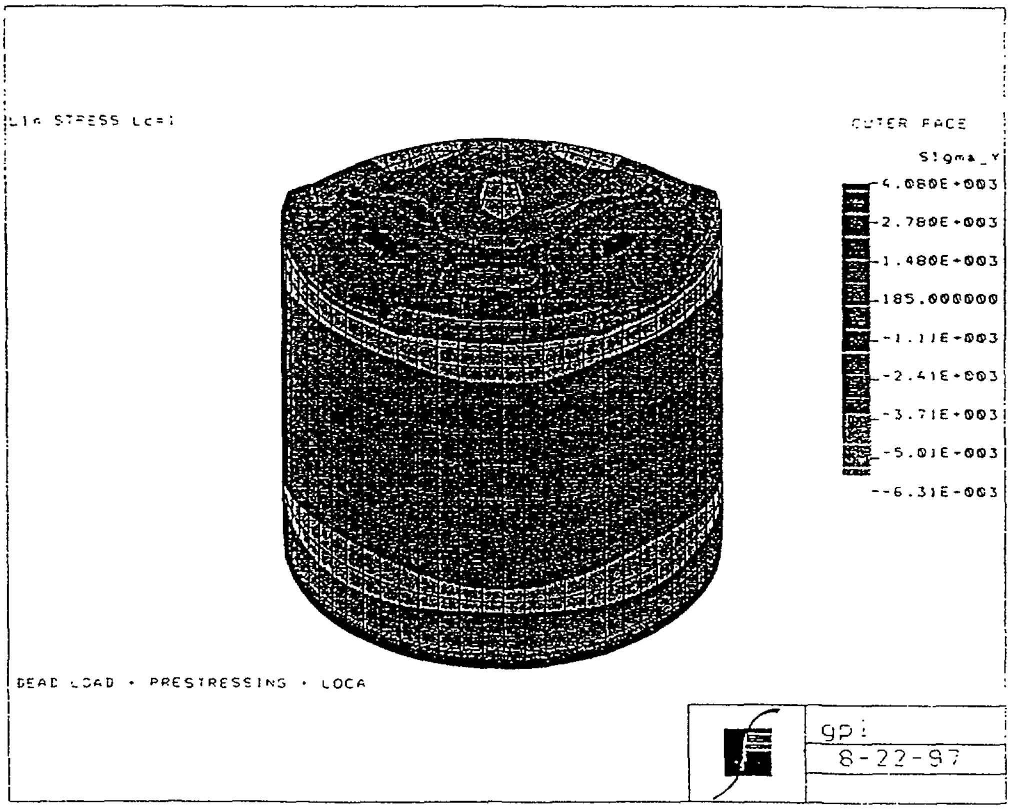


Fig. 4.5 Longitudinal Stresses at the Containment Due to Dead Load, Pre-stressing and LOCA

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COMMENTS ON THE SEISMIC SAFETY OF NUCLEAR POWER PLANTS IN EASTERN EUROPE

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Abstract

After the break-up of the Soviet Union, ten countries in Eastern Europe inherited Soviet-designed nuclear power plants which were constructed without adequate provisions to resist earthquake-generated lateral forces. An earthquake at their locations could seriously damage these plants and could result in Chernobyl-like consequences on the environment. There is an ongoing program to reinforce these plants using conventional piecemeal methods. A newly developed seismic protection strategy called "base isolation" or "seismic isolation", widely used in the United States to retrofit existing buildings, is recommended as an economical, technically superior, and more effective solution — where applicable — to make these nuclear power plants capable of resisting seismic forces.

Introduction

The planned construction of permanent cover for the Chernobyl Nuclear Power Plant (Civil Engineering Feb. 1997, Paper 11133) will add another thousand million U.S. dollars to the immense material loss and human suffering caused by the 1986 explosion. It can be reasonably expected that improved operating procedures and better trained personnel can guard against another such "meltdown" in the future.

There is another possibility of a disaster of monumental proportions at Eastern European nuclear power plants which can be caused by earthquakes. What is being done and what can be done to prevent this from happening?

Ten Eastern European countries, Armenia, Bulgaria, Czech Republics, Hungary, Poland, Romania, the Russian Federation, Slovakia, Slovenia, and Ukraine inherited Soviet-designed nuclear power plants of poor seismic design and construction compared to acceptable standards, because the seismicity of the different sites were grossly under-estimated by the designers.

At the International Symposium on Seismic Safety Relating to Nuclear Power Plants held in Kobe, Japan, between March 3 and 6, 1997, it was reported by Aybars Gürpinar and Antonio Godoy of the International Atomic Energy Agency and T. Katona (Hungary) and K. Kostov (Bulgaria) that some site-related external events (earthquakes) were not properly considered in the original plant design of eleven nuclear power plants in those countries.

For example at the Paks nuclear power plant in Hungary, seismic loads on the buildings and their contents had not been included at all in the original design. Unexpectedly on August 15, 1985, an earthquake occurred at Berhida-Peremarton villages close to the north shore of Lake Balaton in Hungary. The intensity on the Modified Mercalli Scale was VII, with corresponding damage to poorly built structures. It was sheer luck that the magnitude of the earthquake registered only 4.7 on the Richter scale and that the epicenter was about 120 km from the Paks nuclear power plant where no damage was observed and a potential disaster was avoided.

A recent study of the seismicity of the site at Paks, carried out by British consulting engineers Ove Arup & Partners, established that ground accelerations of 25% of gravity can be expected in a seismic event. Experience in recent earthquakes is telling us that ground motions of such magnitude can result in serious damage to buildings and their contents unless they are designed and constructed to resist seismic forces. The seriousness of the problem has been recognized and certain remedial measures are being taken by some countries, in particular at Paks, Hungary and Kosloduy, Bulgaria where earthquakes occurred in 1977, 1986, and twice in 1990. The available conventional technology and the piecemeal application controlled by available funds, however, has limitations in its effectiveness. The analysis of equipment and piping for seismic loading and the necessary construction associated with it is complicated by the fact that different levels of the plant have different seismic response, and it is generally necessary to use multiple-support response spectrum analysis. The question is this: *can newly developed technologies improve the quality and simplify the design and construction of earthquake rehabilitation of existing nuclear power plants*? The answer to this question is of utmost importance because much of the work to make all Soviet-designed nuclear power plants in Eastern Europe capable of resisting earthquake forces is yet to be done.

Base Isolation: a new strategy for earthquake protection

During the past twenty-five years a new design technology called "base isolation" or "seismic isolation" for earthquake protection of buildings and other structures such as bridges and highway overpasses received worldwide acceptance and is being used in the design and construction of *new* nuclear power plants as well (Koeberg, South Africa, and Cruas-Meysse, France) (see Table). Base-isolated buildings are mounted on rubber-steel combination shock-absorbing pedestals (like the ones proposed for the permanent cover at Chernobyl) that prevent most of the horizontal ground movement from being transmitted to the structures during an earthquake. The base isolation system works by intercepting and absorbing much of the destructive earthquake energy, which is turned into harmless heat and never reaches the building.

TABLE 1. SUMMARY OF ISOLATION APPLICATIONS AT NUCLEAR FACILITIES[1]

Facility/Location	Seismic Design Basis	Applications	Isolation System
Cruas Nuclear Power Plant, France (four-900 MWe units)	0.3g PGA	2 nuclear islands, each having 2 reactor units, containment buildings and auxiliary buildings	approx. 1800 laminated neoprene/steel bearings per nuclear island
Koeberg Nuclear Power Plant, South Africa (two-900 MWe units)	0.3g PGA	1 nuclear island with 2 reactor units, containment and auxiliary buildings	approx. 1800 laminated neoprene/steel bearing with slip surface
Torillon Radioactive Waste Facility France	0.3g PGA	3-story, reinforced-concrete building	52 laminated neoprene/steel bearings
La Hague Reprocessing Plant, France	0.3g PGA	spent-fuel pools	laminated neoprene/steel bearings
Sellafield Nuclear Reprocessing Facility of British Nuclear Fuels, Ltd., England	0.3g PGA	20 pipe bridges	approx. 10 laminated natural rubber/steel bearings per bridge
Diablo Canyon Nuclear Power Plant, USA	0.75g PGA	2 exciter units, turbine building	4 high-damped laminated natural rubber/steel bearings per exciter

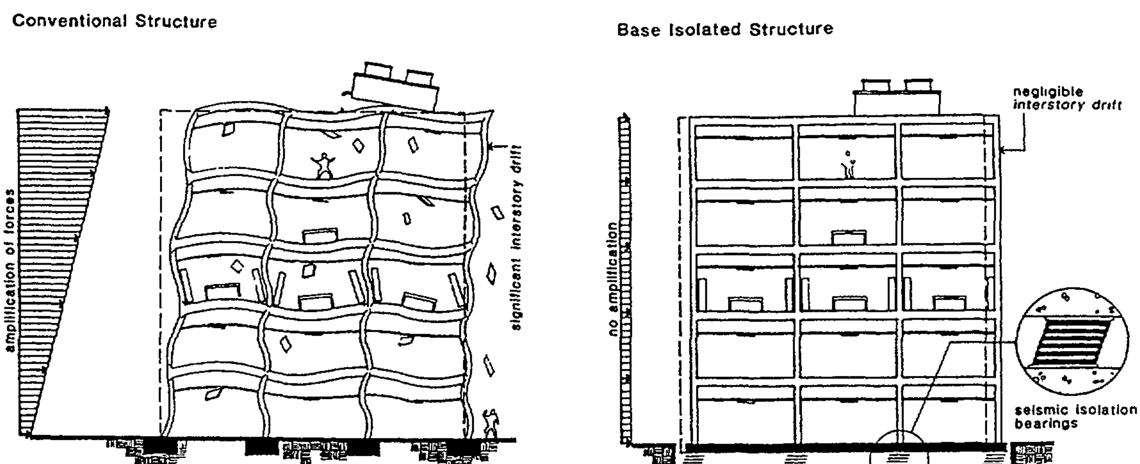
1. Excluding spent-fuel storage rack applications.

Base isolation also changes the behavior of the buildings during earthquakes (Fig. 1). The natural vibration frequency of buildings is changed to the vibration frequency of the base isolators, far away from the dominant earthquake frequency. Unlike conventional design the buildings don't "resonate" to earthquakes; they are "de-tuned". Rather than shaking violently with the ground, they "float" gently on their foundation and the damage causing "whipping" action on the higher floors is eliminated. Consequently, the nonstructural building components that are attached to the structure and the loose contents of the buildings as well are safe and buildings can remain functional even after a major earthquake. In the design of nuclear power plants the reduction of response of internal equipment and of the unattached contents of the buildings to earthquake forces is a primary goal.

Base isolation can be used not just for protection of new buildings, but to rehabilitate existing buildings as well. The buildings are cut away from their foundations after base isolators are placed under load carrying walls and columns. Additional construction provides a connecting diaphragm on the top of the base isolators. Most of the construction work is done under the building at the foundation level, therefore, interference with everyday use of the building is minimal or can even be avoided with careful planning. In the United States there are several major buildings completed or under construction using this technology. *Consideration should be given if base isolation technology could be used for retrofitting Soviet-designed nuclear power plants in Eastern Europe with base isolators to make them capable of resisting earthquake generated forces. Base isolation technology can also be used to retrofit individual items of safety related equipment in nuclear power plants which are attached to the building structure and require seismic qualification.*

Research Programs

There are several large research programs directed toward the use of base isolation strategy for nuclear facilities. In the United States a program funded by the Department of Energy and conducted by Argonne National Laboratories was carried out over a period of 1988 to 1992. This program included shake table tests and testing of a variety of elastomeric base isolators to determine their dynamic characteristics, failure modes, and fatigue resistance. The testing program was carried out at the Earthquake Engineering Research Center (EERC) of the University of California at Berkeley in Richmond, California, and at the Energy Technology Engineering Center near Los Angeles, California.



Buildings having conventional structural systems bend and deform during earthquakes. Accelerations of the ground are amplified on the higher floors and the loose contents are damaged. Building deformations could be permanent.

In a base-isolated structure, movement during an earthquake takes place at the level of the isolators. The buildings do not deform. Floor accelerations are low; the building, its occupants, and its loose contents are safe.

FIG. 1.

Recently, a series of tests were conducted at EERC on high-damping natural rubber base isolators for the U.S. Advanced Liquid Metal Reactor Program (ALMR). Base isolation is now an integral part of the structural design for this reactor.

Other seismic-resistant design technologies such as energy-dissipating devices are emerging and while not as widely implemented as base isolation, they could play a vital role in the seismic retrofit of existing nuclear power plants. To explore the use of these new technologies for seismic protection of existing Eastern European nuclear power plants should be made part of the Coordinated Research Program conducted by the International Atomic Energy Agency, and/or the European Commission's Nuclear Safety Programme in Central and Eastern Europe (PHARE) and in the CIS countries (TACIS).

Earthquakes: how big, when, and where?

Seismologists are telling us that on global perspective an earthquake of 8.0 magnitude on the Richter scale happens once a year, a magnitude of 7 happens every week, and a magnitude of 6 is a daily occurrence. We also know that most earthquakes happen along well-known earthquake faults, but damaging earthquakes also occur where there are no known faults. The long-range prediction of the time of occurrence is based on some facts but mostly on probabilistic data and it is hardly more than speculation. The fact is that earthquakes can strike *at any time, on any day, anywhere*, and we must prepare for their occurrence, without procrastination. Time is the essence. The responsible governments should be forcefully advised again about the existing condition of their nuclear power plants, they should be asked to re-arrange their priorities, and they should provide adequate funds for a vigorous and accelerated rehabilitation program.

Modern technology is fully developed and it is available to contribute to the on-going program of how to make these Eastern European nuclear power plants capable of resisting earthquakes and avoid a pending disaster of monumental proportions.

Base Isolation Works

In addition to theoretical considerations, sophisticated computer dynamic analysis, and hundreds of laboratory shake table tests in the United States, Japan, New Zealand, France, Italy, etc., two *recent earthquakes* provided valuable proof that base isolation, where applicable, provides earthquake protection to a degree not possible to achieve with conventional methods.

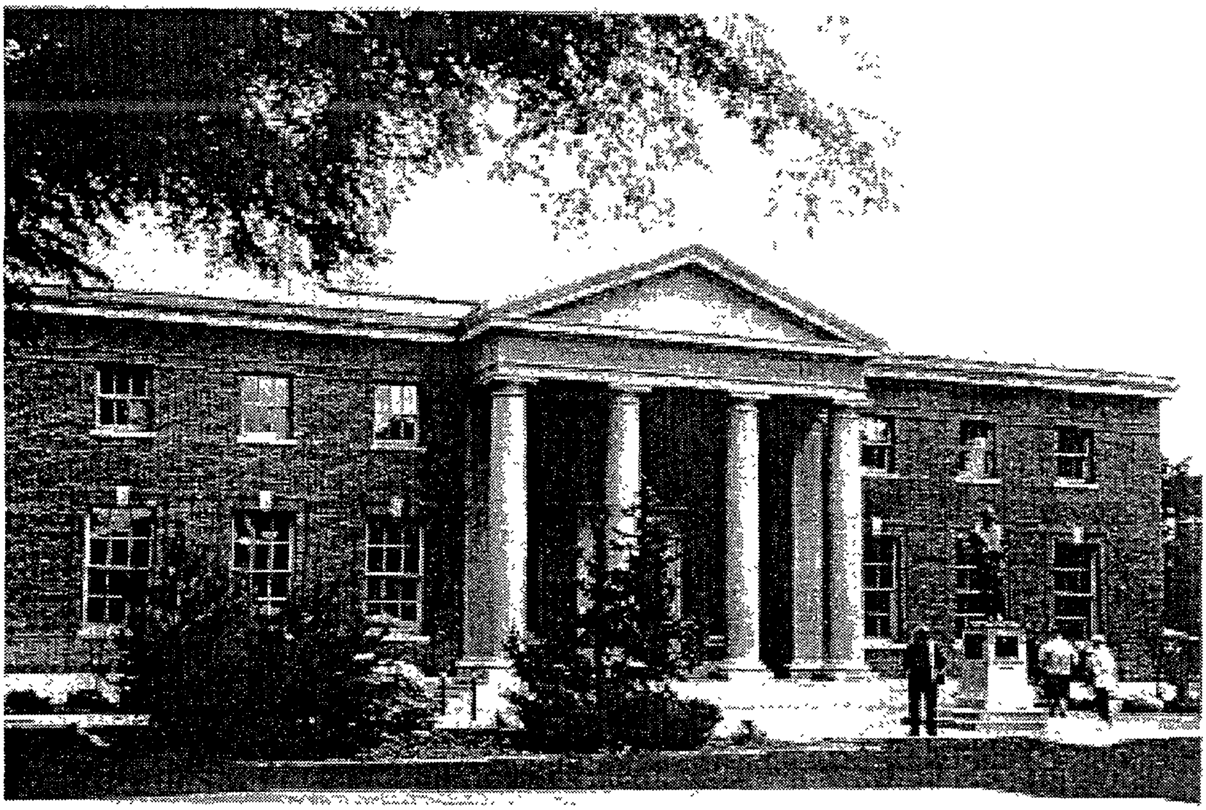
The base-isolated University of Southern California hospital building equipped with seismic sensors as part of the California Strong Motion Instrumentation Program, experienced the 1994 Northridge earthquake without any damage to the building or its contents — "not even a medicine bottle turned over" — and remained fully functional during and after the earthquake. The contents of an adjacent building were left in disarray, rendering that building non-functional immediately after the earthquake. The instruments at the hospital recorded a 65% reduction of the 0.37 g ground acceleration to 0.13 g across the base isolators, without any significant amplification on the higher floors.

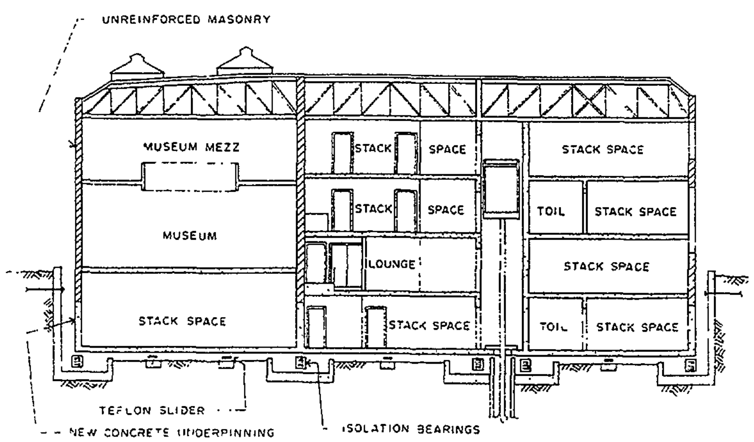
The world's largest base-isolated building, the six-story 500,000 sq.ft. West Japan Postal Savings and Computer Center, was hit by the 1995 6.9 magnitude earthquake at Kobe, Japan. The earthquake activated the base isolation system and the building moved laterally back and forth during the earthquake; the maximum horizontal displacement was 22 cm. Seismic sensors recorded a 63% reduction of the ground acceleration from 0.30 g to 0.11 g without any significant amplification on the higher floors. A conventionally designed three-story building located about 2 km from the Postal building experienced a ground acceleration of 0.37 g, which was amplified at the roof to 1.17 g. This is about *10 times higher* than what was measured at the same time at the base-isolated West Japan Postal Savings and Computer Center

Examples of Retrofitting Existing Buildings with Base Isolation

In the United States there are at this time 20 major existing buildings completed, under construction, or under design for retrofitting with base isolators. The same technology is also used for reinforcing existing bridges and highway overpasses for earthquake protection. About 80 such projects have been completed so far (Figs. 2-8).

MACKAY SCHOOL OF MINES, RENO, NEVADA



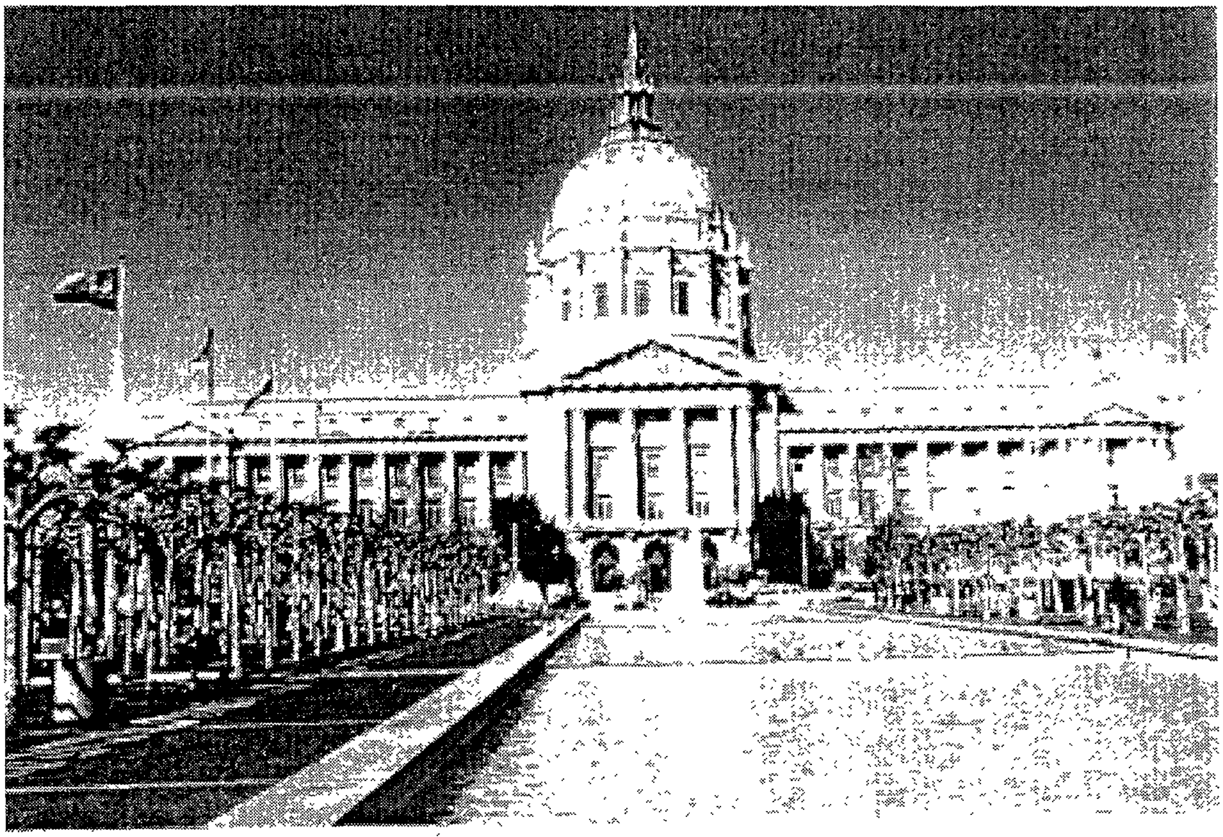


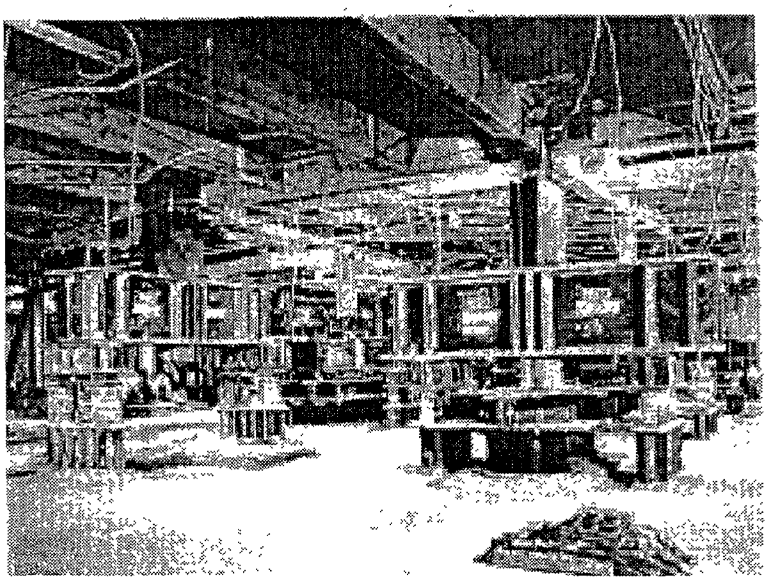
Base Isolation:
Alexander G. Tarics (BIC)
Douglas Way (BIC)
James M. Kelly
FURON, Inc.
Structure:
Jack Howard & Associates
Architecture:
Casazza-Peetz and Hancock

Total Project Cost: $7 million

The Mackay School of Mines building was built in 1908 without any capability of resisting earthquake-generated lateral forces. It had a small basement, which was enlarged and extended to the exterior walls. Base isolators were installed under the basement floor.

FIG. 2.





Base Isolation:
Forell/Elsesser Engineers
Dynamic Isolation Systems, Inc.
Structure:
Forell/Elsesser Engineers , OLMM
Architecture:
San Francisco Department of Public Works
Bureau of Architecture
Total Project Cost: $105 million
Base Isolation Portion: $40 million

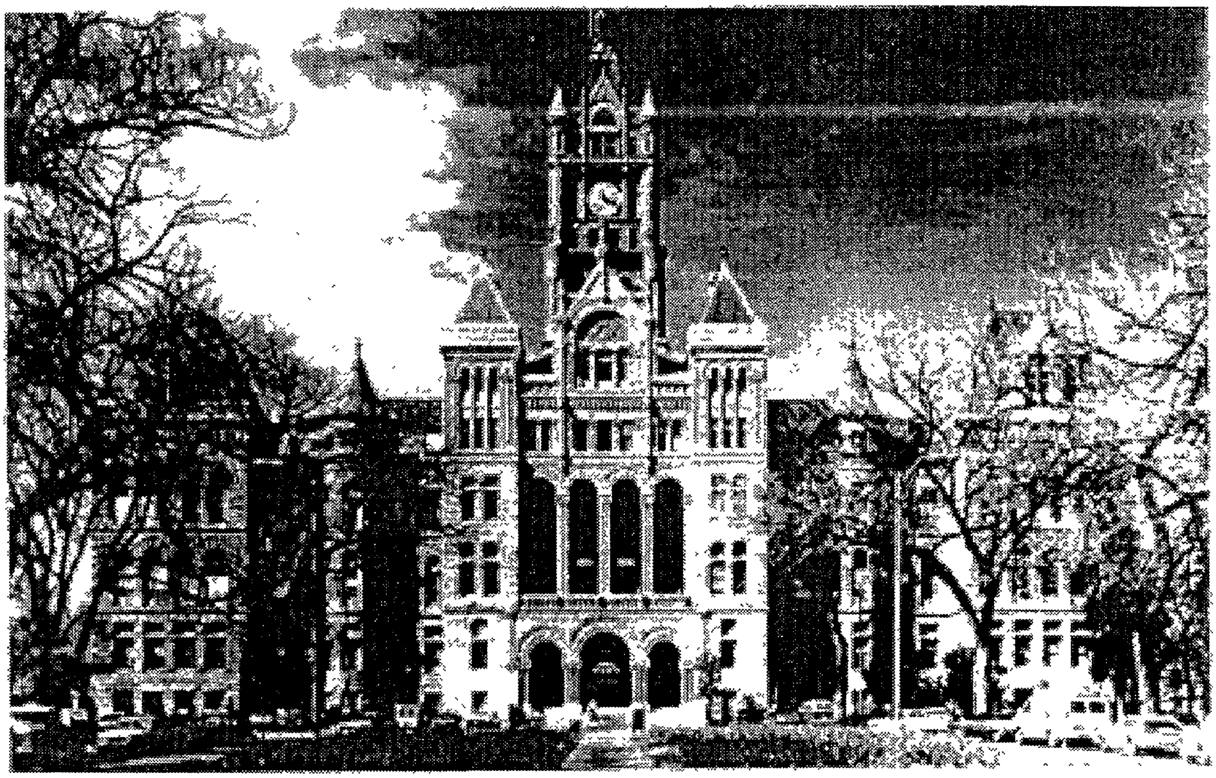
The San Francisco City Hall was heavily damaged in the 8.1 magnitude 1906 earthquake. The 1989 Loma Prieta earthquake damaged the building again, and it is now being retrofitted with base isolators to prepare for the next earthquake. The work is in progress and will be completed in the fall of 1997

FIG. 3.

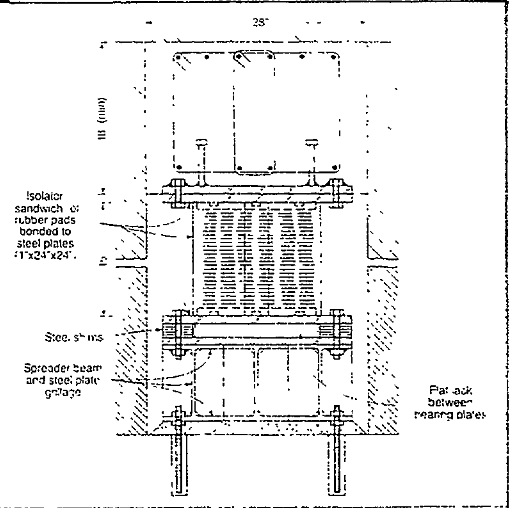
Conclusion

Base isolation reduces earthquake-generated forces and accelerations in existing buildings by a factor of approximately 3 to 10 as compared to conventional retrofitting. The actual reduction depends on the size, shape, height, and flexibility of the building. Base isolation is less sensitive to the uncertainties associated with the prediction of the magnitude of the design earthquake than conventional methods. If a larger earthquake than predicted strikes, the building simply experiences larger lateral displacements with corresponding increase in the horizontal force on the base isolators. This force in turn is significantly reduced in the system before it reaches the building. Soil conditions determine the vibration characteristics of the design earthquake. Careful geotechnical study must be made on any building site for which base isolation is considered.

SALT LAKE CITY AND COUNTY BUILDING, UTAH



BASE ISOLATOR



Typical isolator consists of assembly of rubber pads bonded to steel plates, either with or without lead dampening core

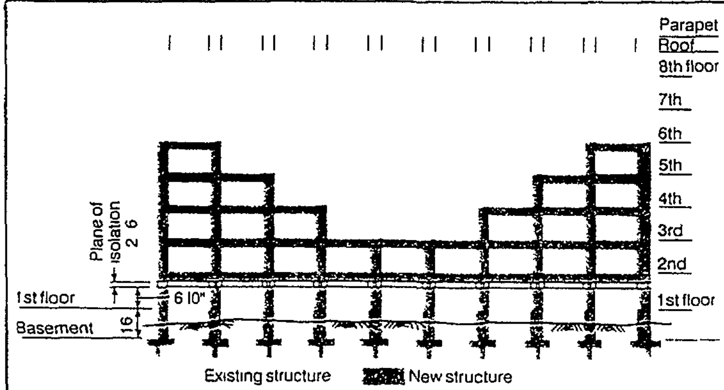
E.W. Allen

Base Isolation:
Forell/Elsesser Engineers
Dynamic Isolation Systems, Inc.
Structure:
E.W. Allen & Associates
Forell/Elsesser Engineers
Architecture:
The Ehrenkrantz Group
Burch Beall, Associate Architect
Total Project Cost: $30 million
Base Isolation Portion: $6 million

This building — a historical landmark — was designed and built between 1890 and 1894 without any provision to resist lateral earthquake forces. By inserting base isolators under the building, it is now capable of resisting earthquakes forces which may occur at this location.

FIG. 4.

SOUTH ELEVATION



Ductile concrete frame added to original structure carries two isolators at each column North wall is similar to south, but east and west walls have frame to sixth floor at all columns

EHI

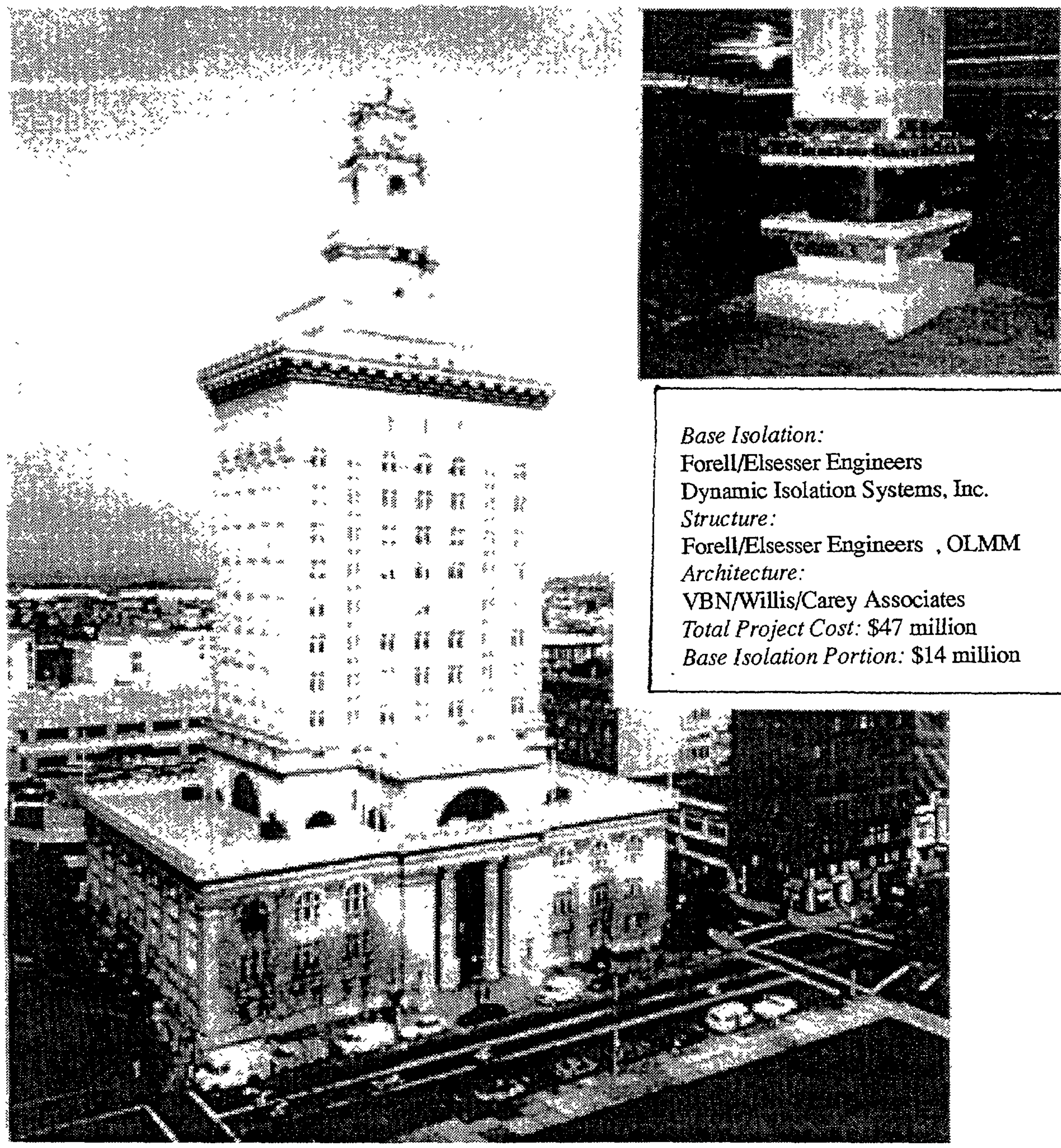
Base Isolation
Englekirk & Hart, Inc.
Dynamic Isolation Systems, Inc

Structure
Englekirk & Hart, Inc
Total Project Cost $14 million
Base Isolation Portion $10 million

In this building at Seal Beach, California, employees are monitoring all NASA space launches and flights Base isolators were installed without any interruption of the activities of this building Located only 1 km from the Newport Inglewood fault, the building is now capable of withstanding a 7 0 magnitude earthquake

FIG. 5.

A comparison of accelerations in a fixed-base conventionally designed building versus a base-isolated building is shown in Fig. 9. The maximum acceleration on the top of the buildings is 0.75 g and 0.1 g, respectively, a reduction by a factor of 7 5 in the base-isolated building, assuming 0.25 g ground accelerations.



Base Isolation:
Forell/Elsesser Engineers
Dynamic Isolation Systems, Inc.
Structure:
Forell/Elsesser Engineers , OLMM
Architecture:
VBN/Willis/Carey Associates
Total Project Cost: $47 million
Base Isolation Portion: $14 million

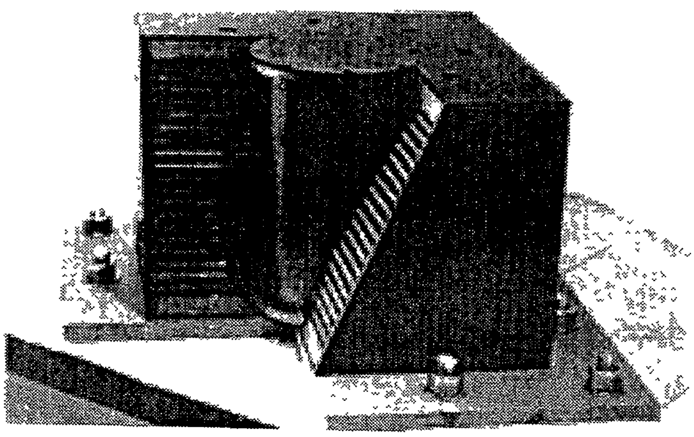
The Oakland City Hall, originally built in 1914, is the world's tallest building retrofitted with base isolators for earthquake protection. The building was badly damaged in the 1989 Loma Prieta earthquake, and the repair of this damage was part of the work, which was completed in 1995.

FIG. 6.

Figure 10 indicates that increasing the ground acceleration by 0.1 g results in an increase of 0.3 g on the top of the fixed-based building, while only 0.04 g on the top of the base-isolated building. This example demonstrates that *a base-isolated building is significantly less sensitive than a fixed-base building to the uncertainties associated with the prediction of the design earthquake.*

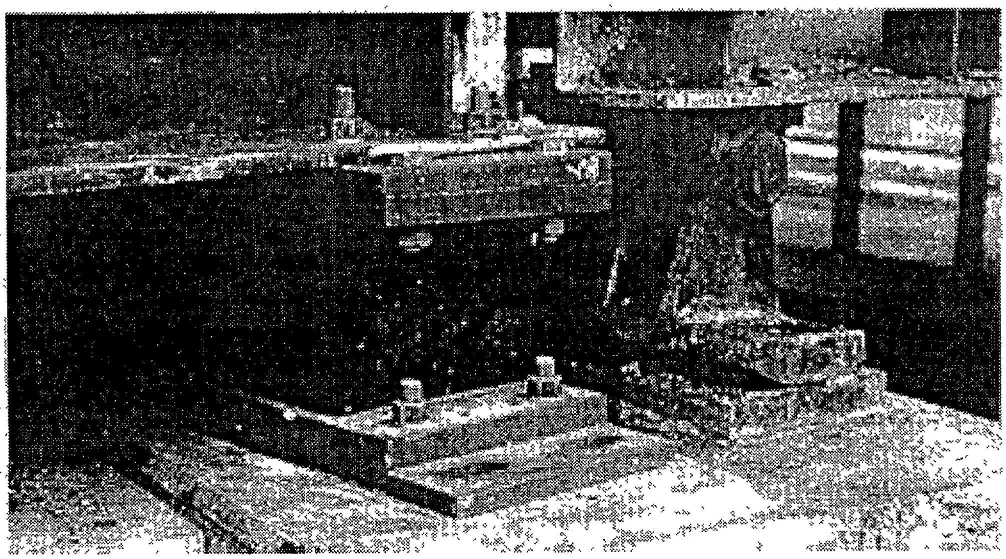
Base isolation strategy merits serious consideration for retrofitting the Soviet-designed nuclear power plants in ten East European countries, thus making them capable of resisting earthquakes that have been predicted to occur at their location.

VETERANS ADMINISTRATION HOSPITAL, LONG BEACH, CALIFORNIA



Base Isolation:
N. Youssef & Associates
Dynamic Isolation Systems, Inc.
Structure:
N. Youssef & Associates
Architecture:
A.C. Martin Associates
Total Project Cost: $18 million
Base Isolation Portion: $12 million

FIG. 7.



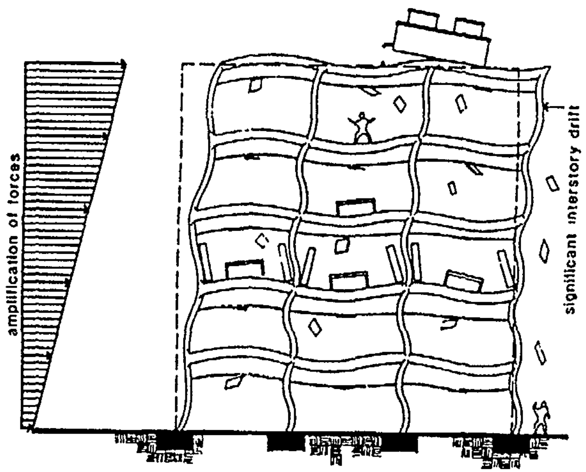
Base Isolation: Dynamic Isolation Systems, Inc. - *Structure*: Sverdrup Corp. and Hsiong Associates

Base isolators are installed to replace the old supports of this bridge and make it capable of resisting earthquakes. Base isolators are used to retrofit highway overpasses as well. The north and south approach viaducts to the Golden Gate Bridge in San Francisco are also being retrofitted with base isolators.

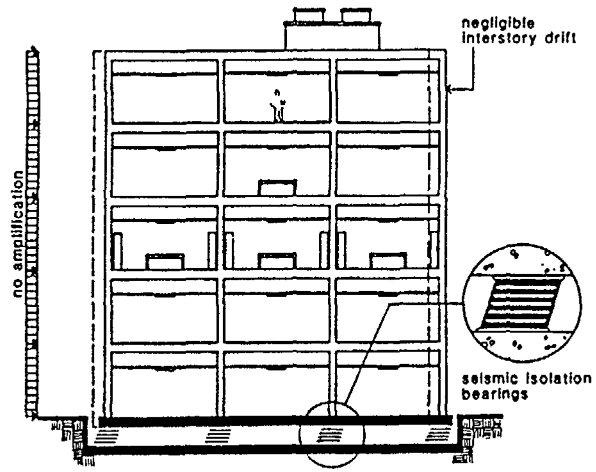
FIG. 8.

COMPARISON

Conventional Structure



Base Isolated Structure



GROUND ACCELERATION
0.25g

REDUCTION
NO REDUCTION

ACCELERATION AT BASE
0.25g

AMPLIFICATION
BY A FACTOR OF 3

ACCELERATION AT THE TOP
3 x 0.25g = 0.75g

GROUND ACCELERATION
0.25g

REDUCTION
60%

ACCELERATION AT BASE
0.25g - 0.6 x 0.25g = 0.10g

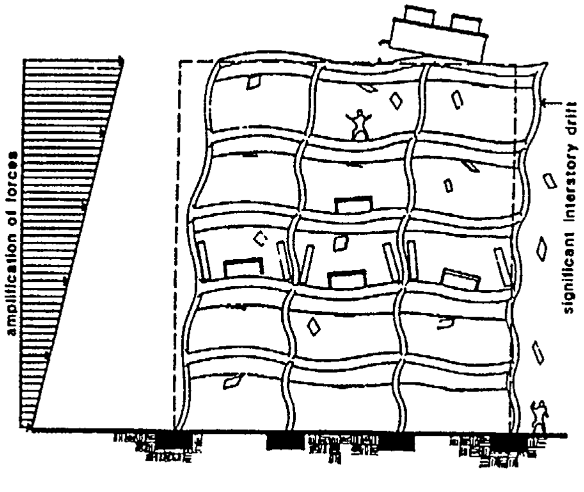
AMPLIFICATION
NO SIGNIFICANT AMPLIFICATION

ACCELERATION AT THE TOP
0.10g

FIG. 9.

COMPARISON

Conventional Structure



GROUND ACCELERATION

0.25g
0.35g

REDUCTION

NO REDUCTION

ACCELERATION AT BASE

0.25g
0.35g

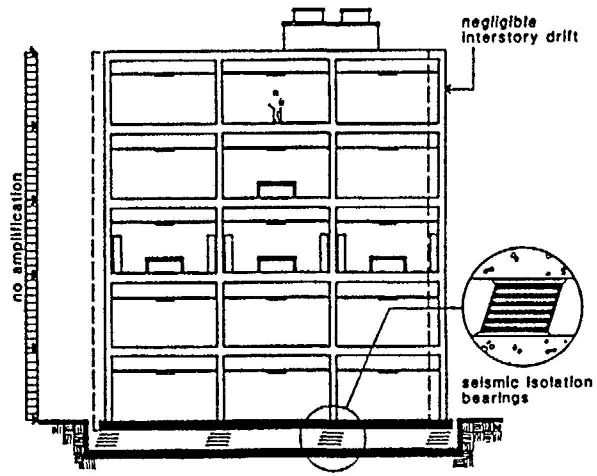
AMPLIFICATION

BY A FACTOR OF 3

ACCELERATION AT THE TOP

3 x 0.25g = 0.75g
3 x 0.35g = 1.05g
INCREASE = 1.05g - 0.75g = 0.3g

Base Isolated Structure



GROUND ACCELERATION

0.25g
0.35g

REDUCTION

60%

ACCELERATION AT BASE

0.25g - 0.6 x 0.25g = 0.10g
0.35g - 0.6 x 0.35g = 0.14g

AMPLIFICATION

NO SIGNIFICANT AMPLIFICATION

ACCELERATION AT THE TOP

0.10g
0.14g
INCREASE = 0.14g - 0.10g = 0.04g

FIG. 10.

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RESULTS OF BENCHMARK STUDY FOR THE SEISMIC ANALYSIS AND TESTING OF WWER TYPE NPPS — FULL SCALE TESTING RESULTS

(Session II)

RESULTS OF A BENCHMARK STUDY FOR THE SEISMIC ANALYSIS AND TESTING OF WWER TYPE NPPs: OVERVIEW AND GENERAL COMPARISION FOR PAKS NPP

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Abstract

Within the framework of the IAEA coordinated *"Benchmark Study for the seismic analysis and testing of WWER-type NPP's"*, in-situ dynamic structural testing activities have been performed at the Paks Nuclear Power Plant in Hungary. The specific objective of the investigation was to obtain experimental data on the actual dynamic structural behaviour of the plant's major constructions and equipment under normal operating conditions, for enabling a valid seismic safety review to be made.

This paper refers on the comparison of the results obtained from the experimental activities performed by ISMES with those coming from analytical studies performed for the Coordinated Research Programme (CRP) by Siemens (Germany), EQE (Bulgaria), Central Laboratory (Bulgaria), M. David Consulting (Czech Republic), IVO (Finland).

This paper gives a synthetic description of the conducted experiments and presents some results, regarding in particular the free-field excitations produced during the earthquake-simulation experiments and an experiment of the dynamic soil-structure interaction global effects at the base of the reactor containment structure. The specific objective of the experimental investigation was to obtain valid data on the dynamic behaviour of the plant's major constructions, under normal operating conditions, to support the analytical assessment of their actual seismic safety. The full-scale dynamic structural testing activities have been performed in December 1994 at the Paks (H) Nuclear Power Plant. The Paks NPP site has been subjected to low level earthquake-like ground shaking, through appropriately devised underground explosions, and the dynamic response of the plant's 1st reactor unit important structures was appropriately measured and digitally recorded, with the whole nuclear power plant under normal operating conditions. In-situ free field response was measured concurrently and, moreover, site-specific geophysical and seismological data were simultaneously recorded too.

For the benchmark purposes it was decided to make reference to the instrumentation lay-out of the three blasts experiment. This instrumentation lay-out covered the more directly safety related structures, i.e., the reactor containment building itself, the above-located reactor hall and one of the nearby coupled chimneys (the southern ones). In order to compare homogeneous data, all the data were converted into acceleration data; thus the recorded velocity signals were derivated in order to obtain acceleration signals. Starting from these acceleration data the acceleration response spectra were calculated. In order to simply compare the experimental results with the analytical results the same positions were reported on the same tables.

The following general considerations can be drawn: the amplitudes of the calculated response spectra are higher than those obtained experimentally, there is a high influence of the frequency energy content due to the nature of the explosion excitation, it has to be expected that during a seismic event a higher excitation at the soil level will involve dissipating mechanism leading to higher values of the damping, the experimental tests should be envisaged when dealing with the seismic verification of an existing NPP in order to have a more refined estimation of the damping: this estimate is valuable in order to provide a lower bound to the damping values.

1. INTRODUCTION

This paper refers to the comparison of the results obtained from the experiments performed by ISMES on the Paks NPP with those coming from analytical studies performed for the Co-ordinated Research Programme (CRP) on "Benchmark study for the Seismic Analysis and Testing of WWER Type Nuclear Power Plants" sponsored by the International Atomic Energy Agency (IAEA) of Vienna. There are twenty five participants from fifteen countries in this CRP (Ref. 1). The participants to the above cited benchmark study for the full scale testing comparison of Paks NPP structures were:

- Siemens (Germany)
- EQE (Bulgaria)
- Central Laboratory (Bulgaria)
- M. David Consulting (Czech Republic)
- IVO (Finland)

The specific objective of the experimental investigation was to obtain valid data on the dynamic behaviour of the plant's major structures under normal operating conditions, to support the analytical assessment of their actual seismic safety.

The full-scale dynamic structural testing activities were performed in December 1994 at the Paks (H) Nuclear Power Plant. The Paks NPP site was subjected to low level earthquake-like ground shaking, through appropriately devised underground explosions, and the dynamic response of major structures of the first unit of the NPP was measured and digitally recorded. In-situ free field response was measured concurrently in order to obtain site-specific geophysical characteristics. The general layout of the experiment is given in Figure 1.

The experimental data were collected to obtain basic information on the geophysical characteristics of the Paks NPP site, together with reference information on the true dynamic characteristics of its main structures and give some indication on the actual dynamic soil-structure interaction effects for the case of low level excitation.

The free field response (Fig. 2), was then distributed among the five participants in order to enable them to calculate the response of the first unit reactor building and to compare the calculated response with the experimentally measured data. More detailed information on the tests can be found in References 2 to 4.

2. MEASUREMENT POSITIONS AND REFERENCE DIRECTIONS

The four reactors of the Paks Nuclear Power Plant are arranged as two twins. The main building of each twin, houses two reactor units in a symmetrical layout and is composed of a stiff 72 m long, 52 m wide and 18,9 high reinforced concrete bearing structure. The latter is supported, together with an adjacent 42 x 24 m condensation tower, on a ~ 2 m thick continuous reinforced concrete direct foundation slab. At level + 18,9 m above the monolithic concrete reactor containment structure, there is a steel frame supported reactor maintenance and reloading hall, fitted with two large overhead rolling cranes.

A large number of seismometers[1] and accelerometers were mounted at appropriate locations in the nuclear power plant major buildings, for recording their structural response to the artificially produced ground motion. Attention was focused on the first reactor unit structures, as these were situated closer to the ground excitation sources.

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[1] Velocity-type transducers, characterized by a higher sensitivity with respect to the accelerometers.

GENERAL LAYOUT OF THE EARTHQUAKE SIMULATION EXPERIMENTS AT THE PAKS NPP SITE

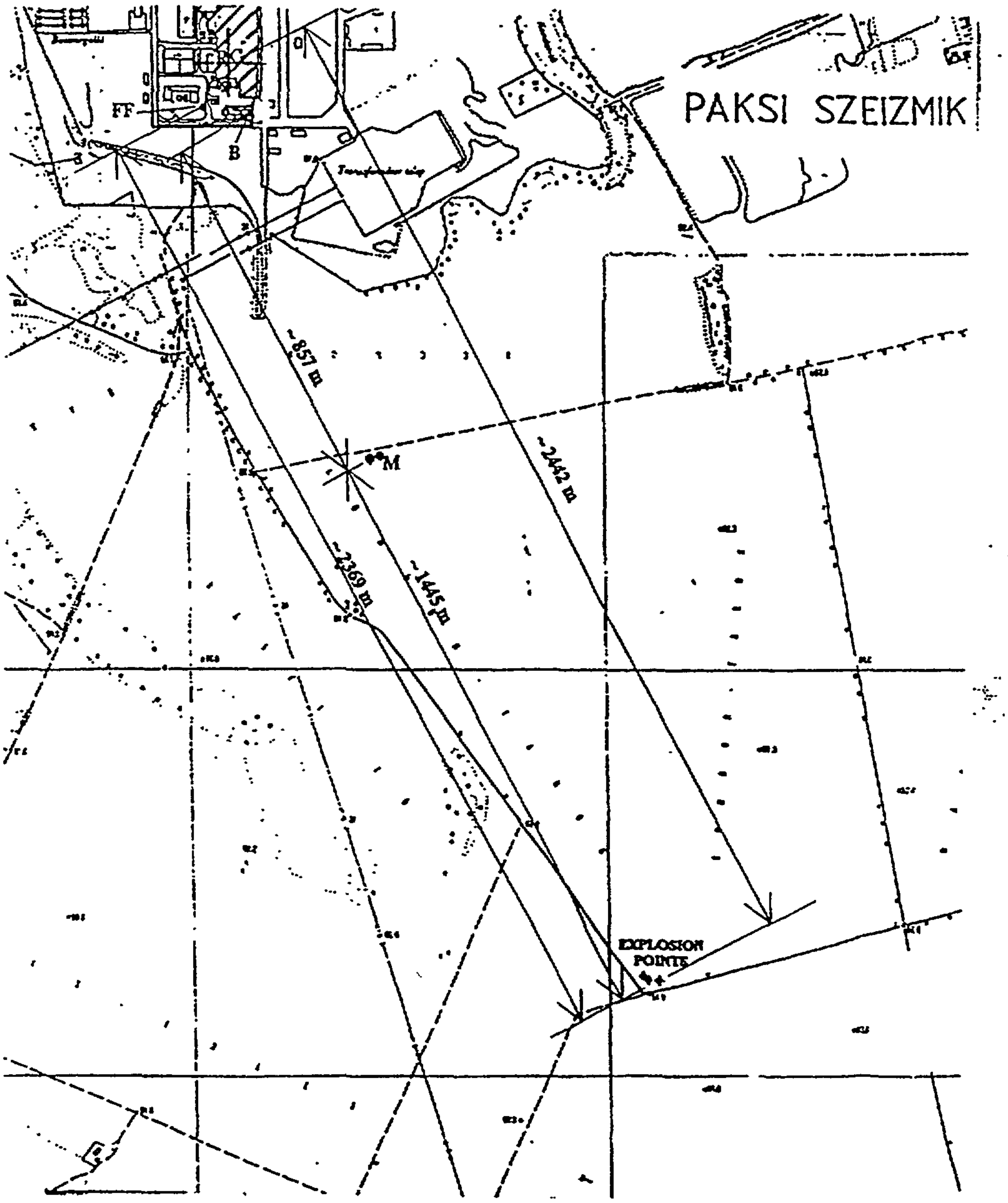


Fig. 1 Explosion points position

First of all, a three orthogonal axes seismometric station **FF** (n. 1-3), was buried 110 cm deep in the natural soil, at a distance of 119 m from the reactor base centre, as shown in Figure 1.

For the benchmark study purposes it was decided to make a reference to the instrumentation lay-out of the three blast experiment. This instrumentation lay-out covered mainly *the safety related structures, i.e., the reactor containment building itself, the reactor hall and one of* the nearby coupled chimneys (the southern ones). The location and progressive numbering of the transducers that were selected for the benchmark are indicated in Figure 3.

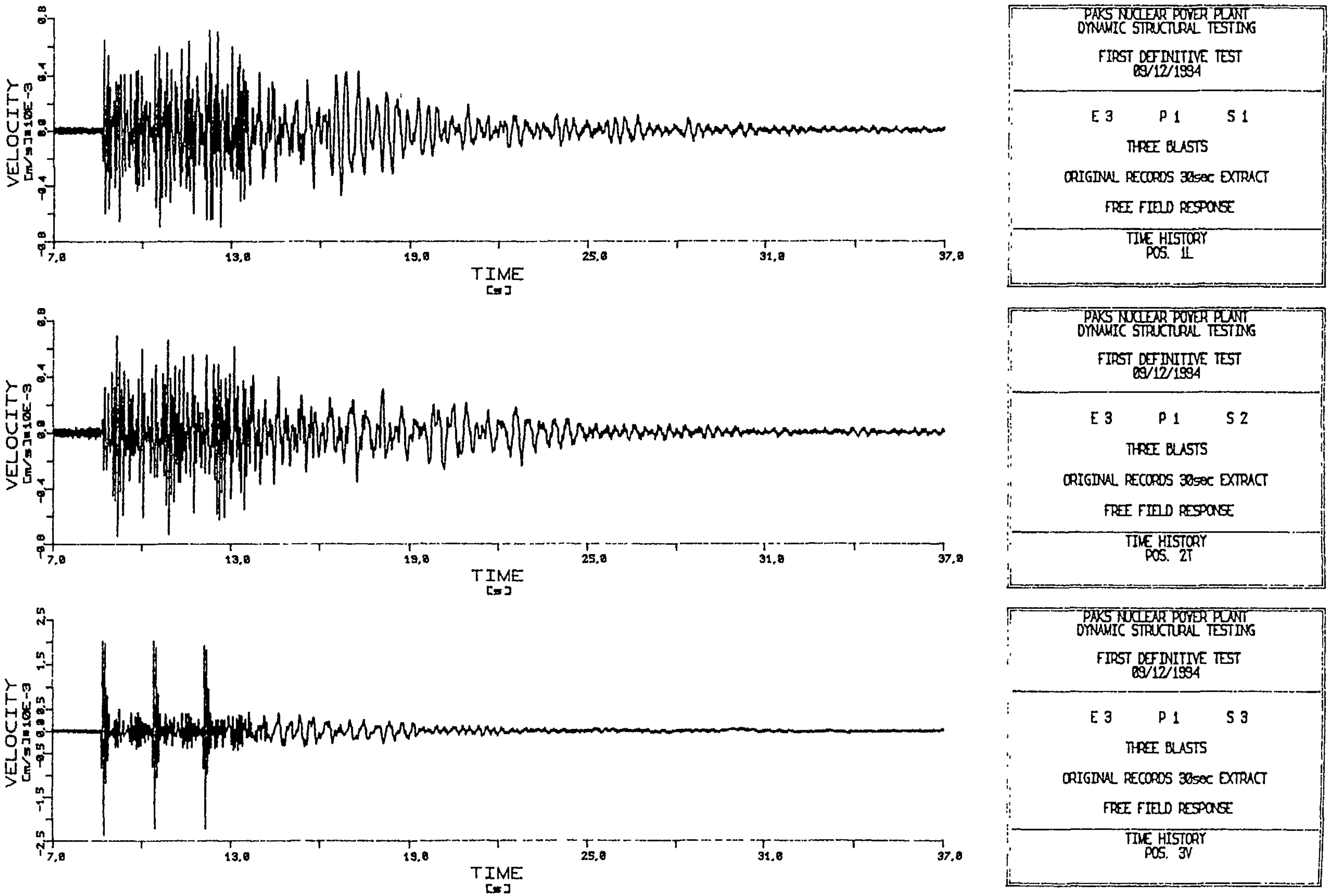


Fig. 2 Free-field excitation

Reactor building	ISMES	SIEMENS	EQE	CL	MD	IVO
Locations	Number/Sensitivity direction					
Free-field (N-S)	1/L	1/X	1/X	1/Y	1/Y	1/X
Free-field (E-W)	2/T	2/Y	2/Y	2/X	2/X	2/Y
Free-field (Vertical)	3/V	3/Z	3/Z	3/Z	3/Z	3/Z
Base-mat (-6,5 m)	4/L	4/X	4/X	491/Y	65/Y	4/X
Base-mat (-6,5 m)	5/T	5/Y	5/Y	491/X	65/X	5/Y
Base-mat (-6,5 m)	6/V	6/Z	6/Z	491/Z	65/Z	6/Z
Base-mat (-6,5 m)	14/L	14/X	14/X	1525/Y	656/Y	14/X
Base-mat (-6,5 m)	15/T	15/Y	15/Y	1525/X	656/X	15/Y
Base-mat (-6,5 m)	16/V	16/Z	16/Z	1525/Z	656/Z	16/Z
Base-mat (-6,5 m)	21/L	21/X	21/X	1493/Y	601/Y	21/X
Reactor hall (+18,9 m)	33/T	33/Y	33/Y	....	753/Y	33/Y
Reactor hall (+18,9 m)	35/L	35/X	35/X	342/Y	773/Y	35/X
Reactor hall (+18,9 m)	36/T	36/Y	36/Y	342/X	751/X	36/Y
Reactor hall (+18,9 m)	37/V	37/Z	37/Z	342/Z	751/Z	37/Z
Over head crane railway	46/T	46/Y	46/Y	....	2387/X	46/Y
Over head crane railway	47/V	47/Z	47/Z	....	2387/Z	47/Z

Fig. 3 Location and numbering of the response positions on the reactor building

In this table, each measuring position is identified with an arrow indicating the sensitivity direction. The following convention will be adopted throughout this document:

- L (stands for Longitudinal) in direction North-South (the direction of the line connecting the two twin reactor units);
- T (stands for Transversall) in direction East-West;
- V (stands for Vertical) in vertical direction;

3. SIMULATED EARTHQUAKE EXCITATION TESTS

For carrying out the experimental investigation, in December 1994, the Paks NPP site was subjected to the effects of appropriately designed buried explosions, with the object of inducing an earthquake-type excitation to the plant's structures. Actually, by transmitting the vibratory energy to the structures through their foundation soil - as in real earthquakes - the full-scale soil-structure dynamic effects are activated and can thus be investigated for the case of low strain excitation. A set of different successive experiments was performed at the Paks site, with the whole nuclear power plant under normal operating conditions.

The experiments were performed by detonating explosive charges (TNT), previously installed in deep boreholes, at an overall mean horizontal distance of 2442 m in the South/South-East direction from the first unit reactor base centre.

After a preliminary test, the first definitive experiment was performed by detonating three TNT charges with two delays, at a mean horizontal distance of 2434 m from the first reactor base centre.

4. CHARACTERISTICS OF THE EXPERIMENTAL AND ANALYTICAL DATA

4.1. Experimental data

For the synchronous recording of the free-field excitation data, together with the related structural response signals during the low strain earthquake-type excitation experiments, use was made of advanced multichannel data acquisition and analysis system, developed by ISMES, the hardware of which was set up in a mobile laboratory parked beside the first reactor unit building.

This computerized data acquisition and analysis system is capable of recording simultaneously up to 52 signals at a 200 kHz sampling frequency, with real-time analog to digital conversion. It is a sub-module of AIACE (The Advanced ISMES Acquisition, Analysis and Control Environment), a hardware and software environment that has been specifically developed for the performance of static or dynamic experiments, while providing wide data analysis capabilities.

Once the first instrumentation layout was installed, the related shielded cablings connected and the data acquisition set up, a series of measurements were made during plant normal operating conditions, for examining the ambient of vibration intensity levels and frequency content. As significant noise levels were noted to be present at higher frequencies, it was decided to make use of low-pass analog filters in the recordings to be made, for eliminating the high frequency noise prior to digitizing. Acquisitions were hence made with 20 Hz low-pass filters inserted in all the measurement channels. These filters also performed the anti-aliasing function.

A sampling rate of 200 Hz was chosen in order to ensure a satisfactory definition of the blast-induced vibration time histories. An example of the original data is given in Fig. 2.

4.2. Analytical data

4.2.1. Siemens data

The original Siemens data were acceleration response spectra with the following characteristics:

- 76 frequency values from 0,1 Hz up to 34 Hz;
- acceleration in m/s^2

4.2.2. EQE data

The original EQE data were acceleration time histories with the following characteristics:

- 5000 points in the time domain with a sampling interval of 0,005 s;
- acceleration in m/s^2;

4.2.3. CL data

The original CL data were acceleration response spectra with the following characteristics:

- 100 frequency values from 0,1 Hz to 25 Hz;
- acceleration in cm/s^2;
- case 1: original free field experimental acceleration time histories, as delivered by ISMES;
- case 2: deconvoluted free field experimental acceleration time histories (for more information reference is made to the paper by Mr. Kostov et al. presented during the RCM in Bergamo, Italy).

4.2.4. MD data

The original MD data were acceleration response spectra with the following characteristics:

- 96 frequency values from 0,2 Hz up to 33 Hz;
- acceleration in m/s^2;

4.2.5. IVO data

The original IVO data were acceleration time histories with the following characteristics:

- 4000 points in the time domain with a sampling interval of 0,005 s;
- acceleration in m/s^2;

5. DATA PROCESSING DESCRIPTION

5.1 Experimental Data

In order to compare homogeneous information, all data were converted into acceleration; thus derivatives of the recorded velocity signals were generated in order to obtain acceleration signals. Starting from these acceleration data, the acceleration response spectra were calculated on a time window lasting 19,995 seconds with the following parameters:

- 12 frequency values per octave starting from 0,05 Hz up to 100 Hz;
- two damping ratio values (2% and 5%);

The experimentally obtained response (ISMES) are then overplotted on the analytical spectra with a solid line in the frequency range from 0 Hz up to 35 Hz.

5.2. Analytical data

In order to simply compare the experimental results with the analytical results, the same positions were reported on the same tables in correspondence to two different damping ratio values. In Figure 3, a synoptic table of the positions is given to identify the original marking of each participant to the benchmark.

Original spectra were simply plotted and original time histories were processed with the same parameters used for the experimental data.

Figure 4 - 18 present the comparison of the experimental data with analytical results of five participants in terms of response spectra with 2 and 5% damping.

Text cont. on pg. 219.

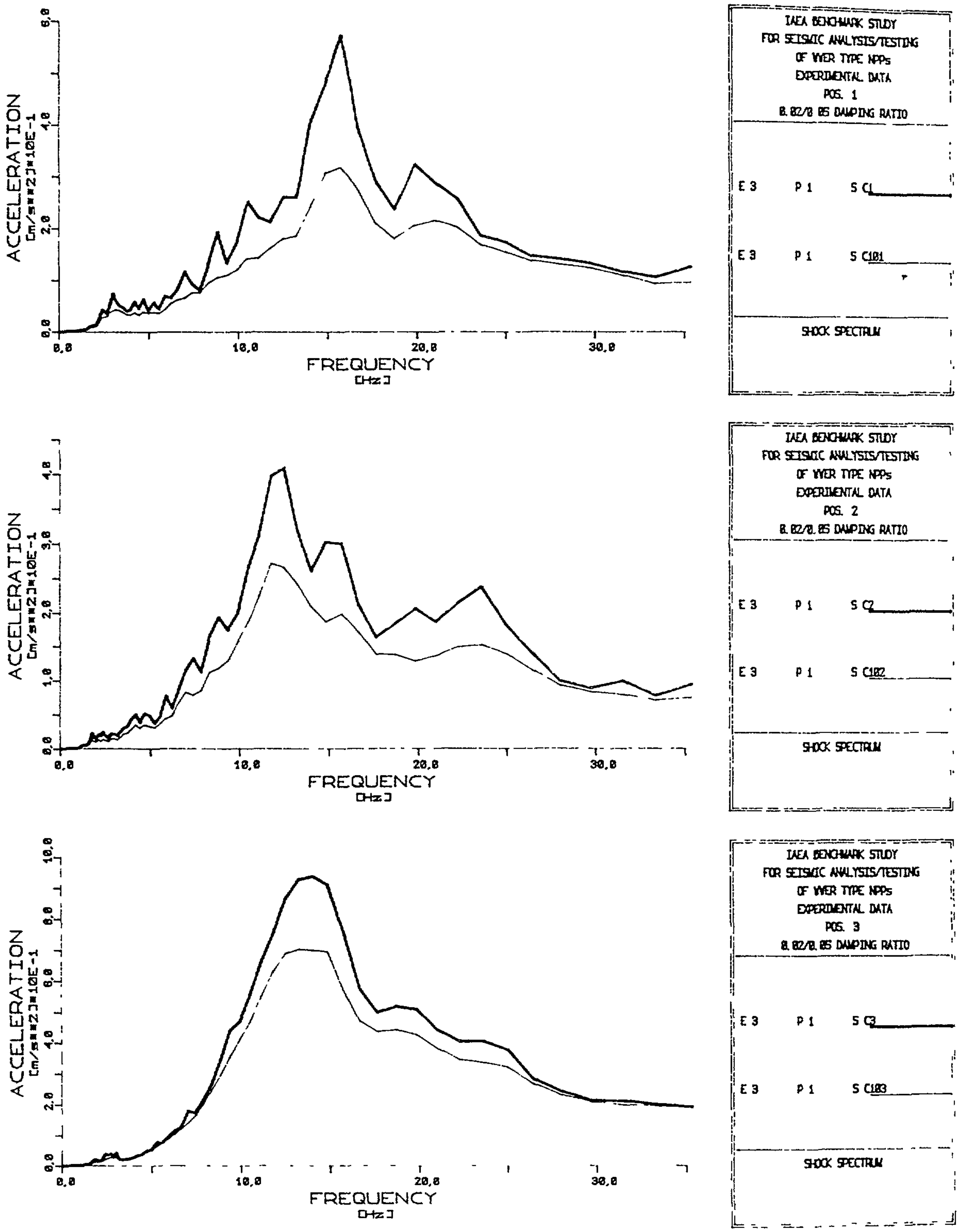


Fig. 4 Free-field response spectra

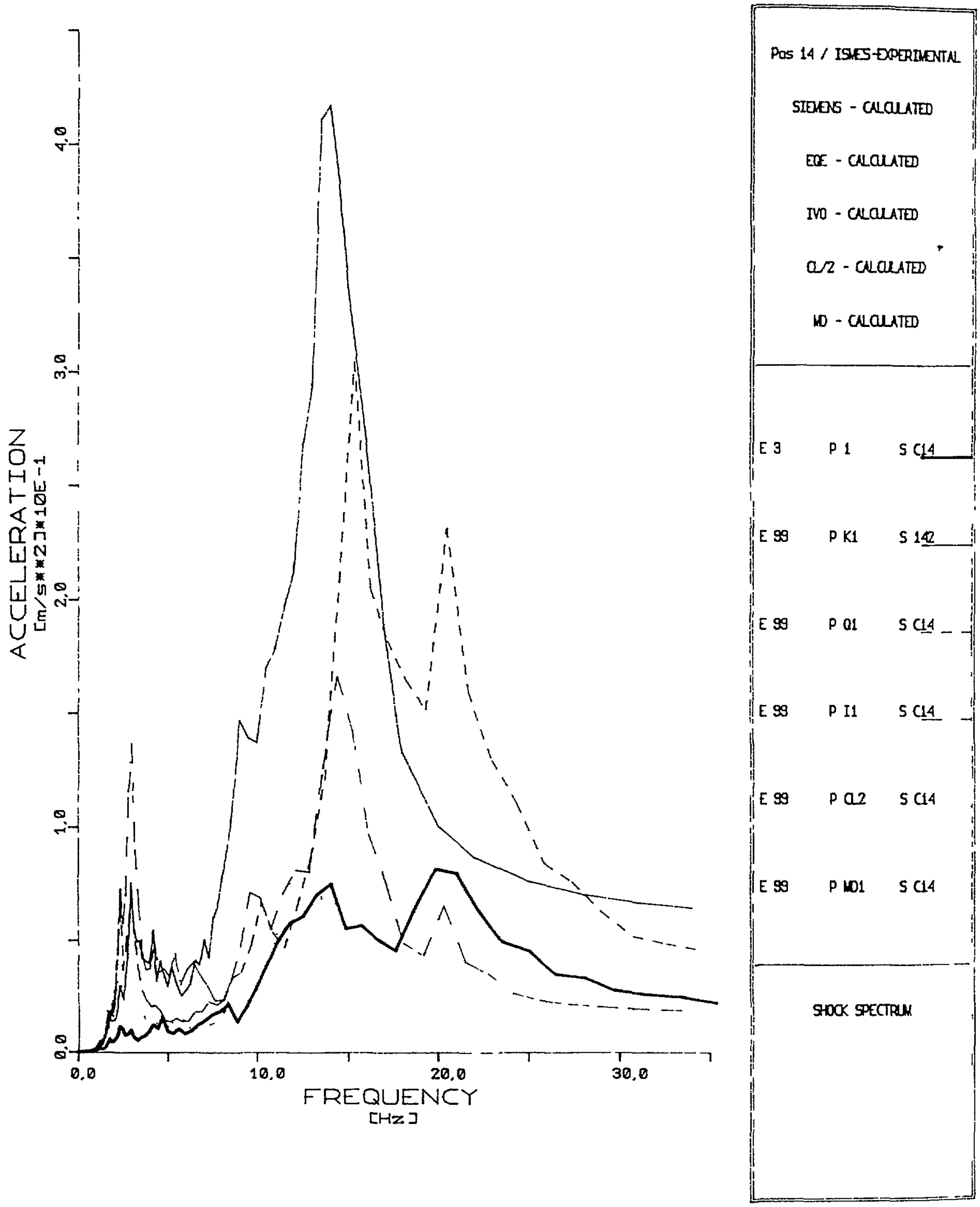


Fig. 5 Comparison between experimental and calculated response spectra at 2% damping ratio in position 14 (frequency range from 1 to 35 Hz)

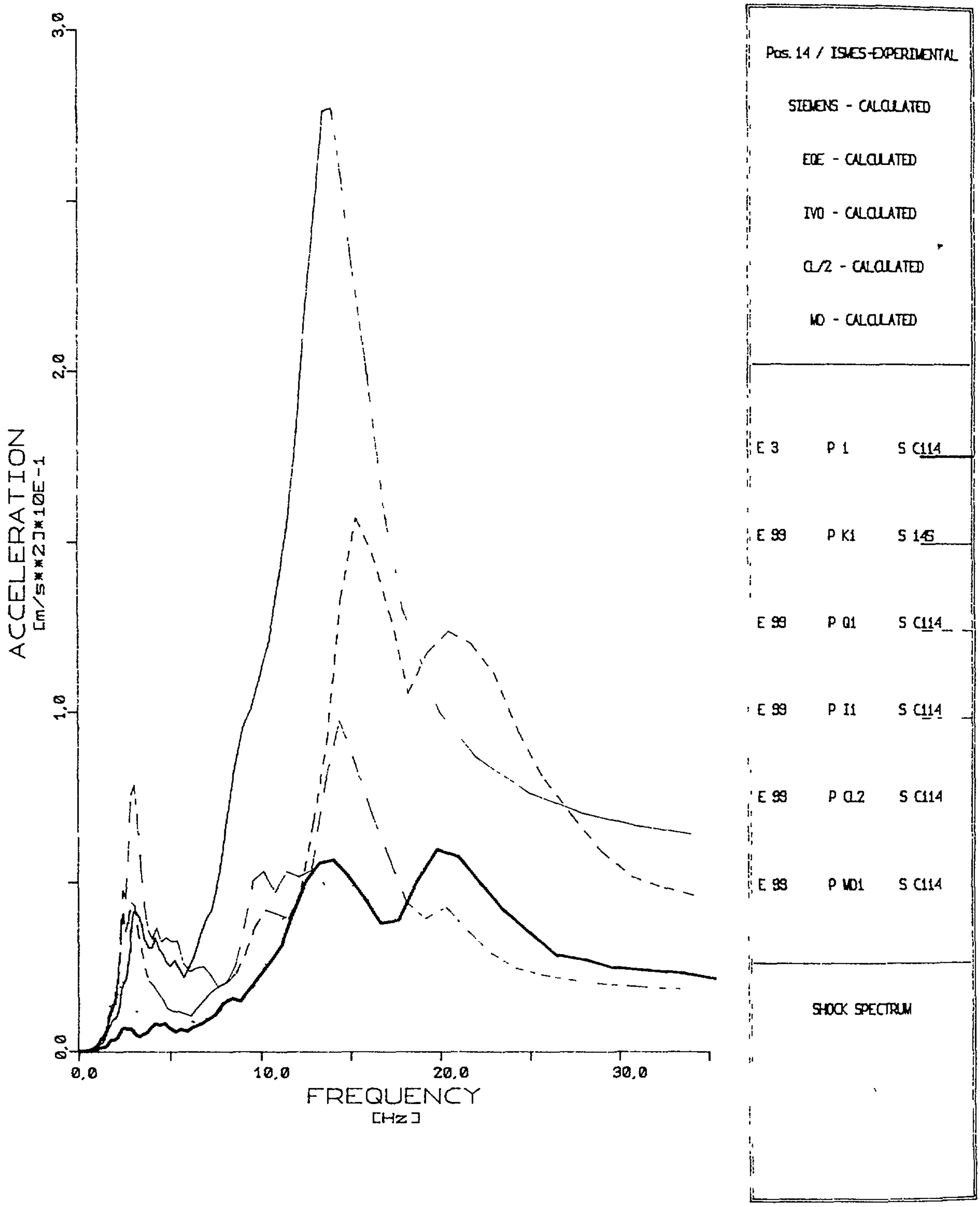


Fig. 6 Comparison between experimental and calculated response spectra at 5% damping in position 14 (frequency range from 1 to 35 H)

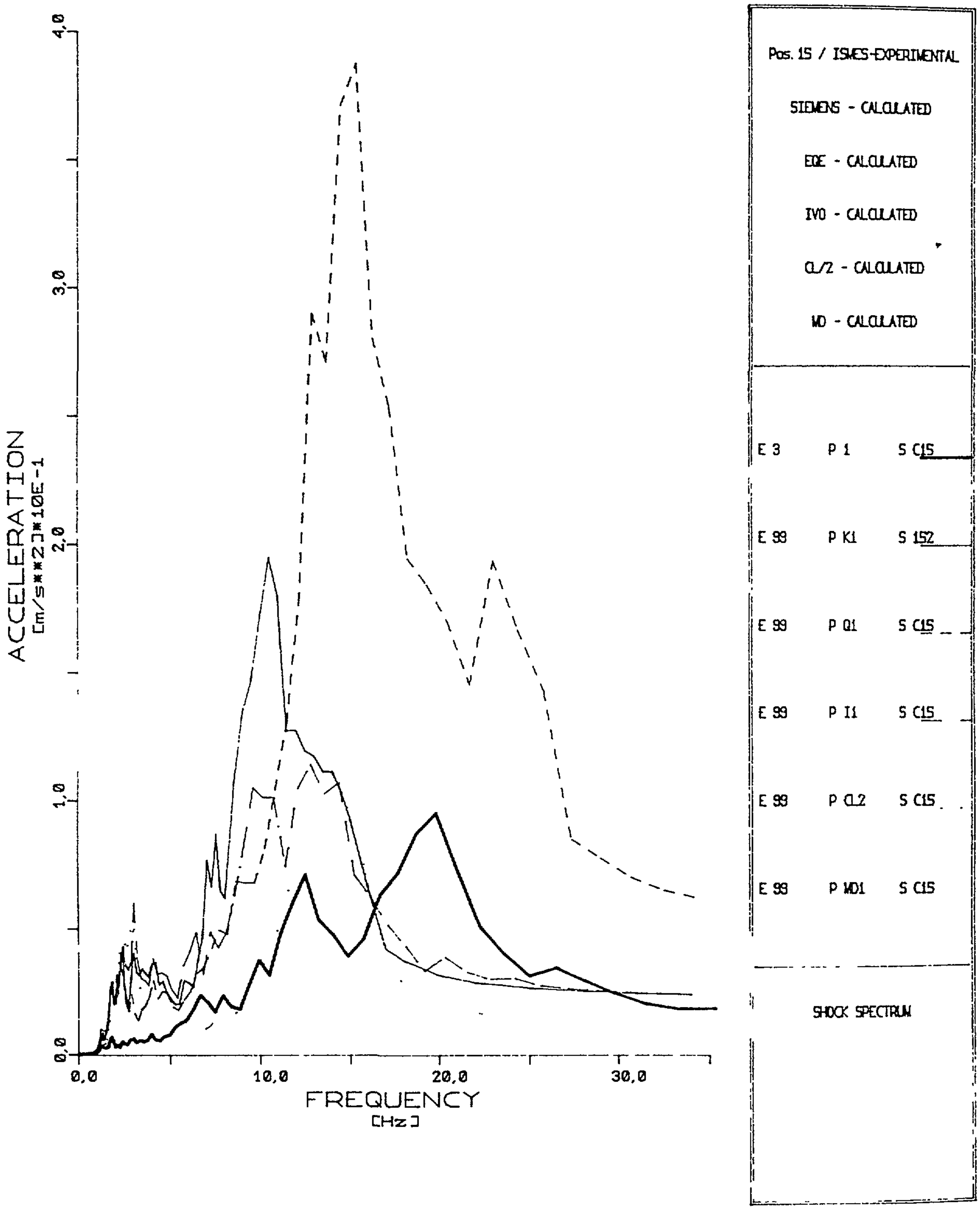


Fig. 7 Comparison between experimental and calculated response spectra at 2% damping in position 15 (frequency range from 1 to 35 Hz)

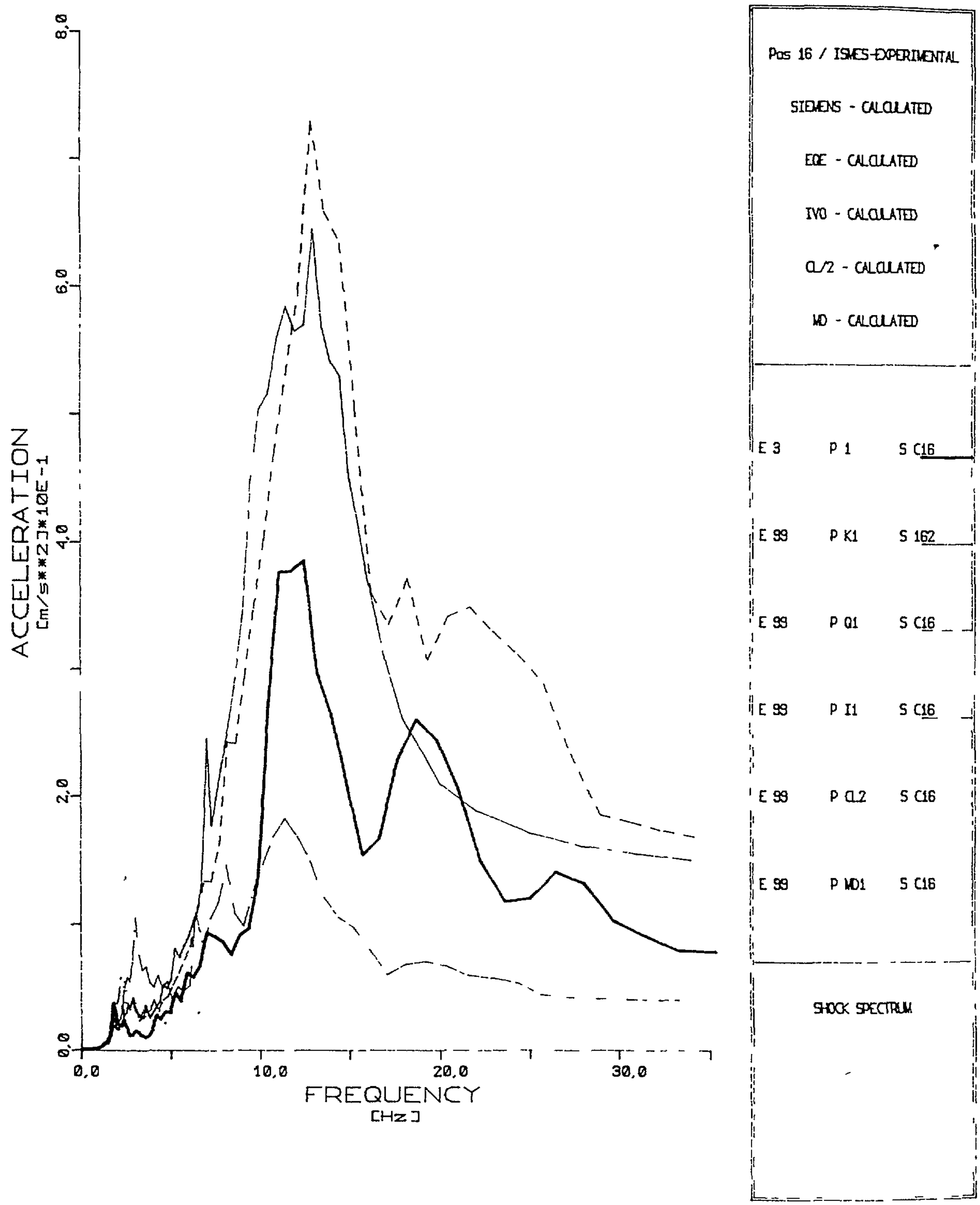


Fig. 8 Comparison between experimental and calculated response spectra at 2% damping in position 16 (frequency range from 1 to 35 Hz)

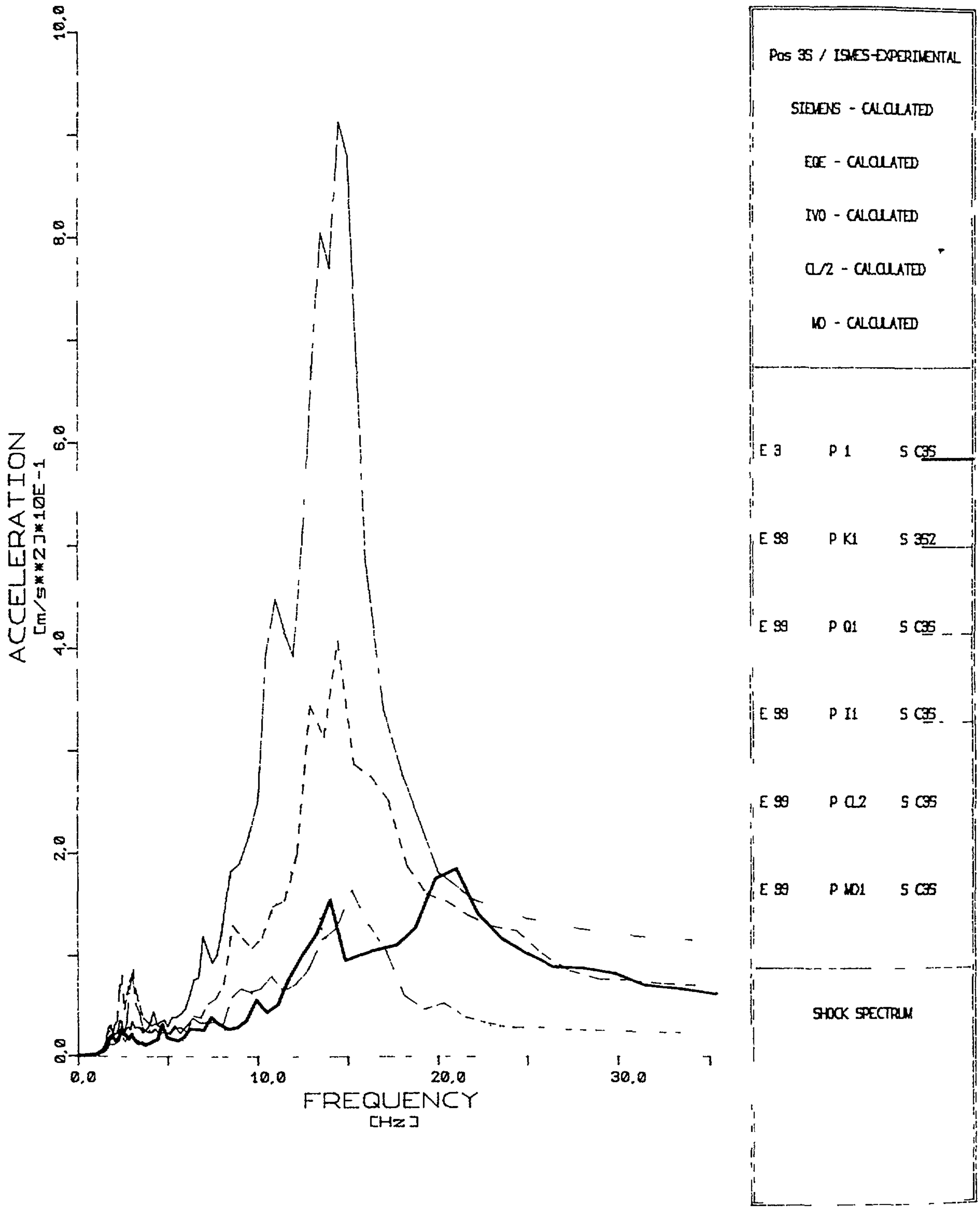


Fig. 9 Comparison between experimental and calculated response spectra at 2% damping in position 35 (frequency range from 1 to 35 Hz)

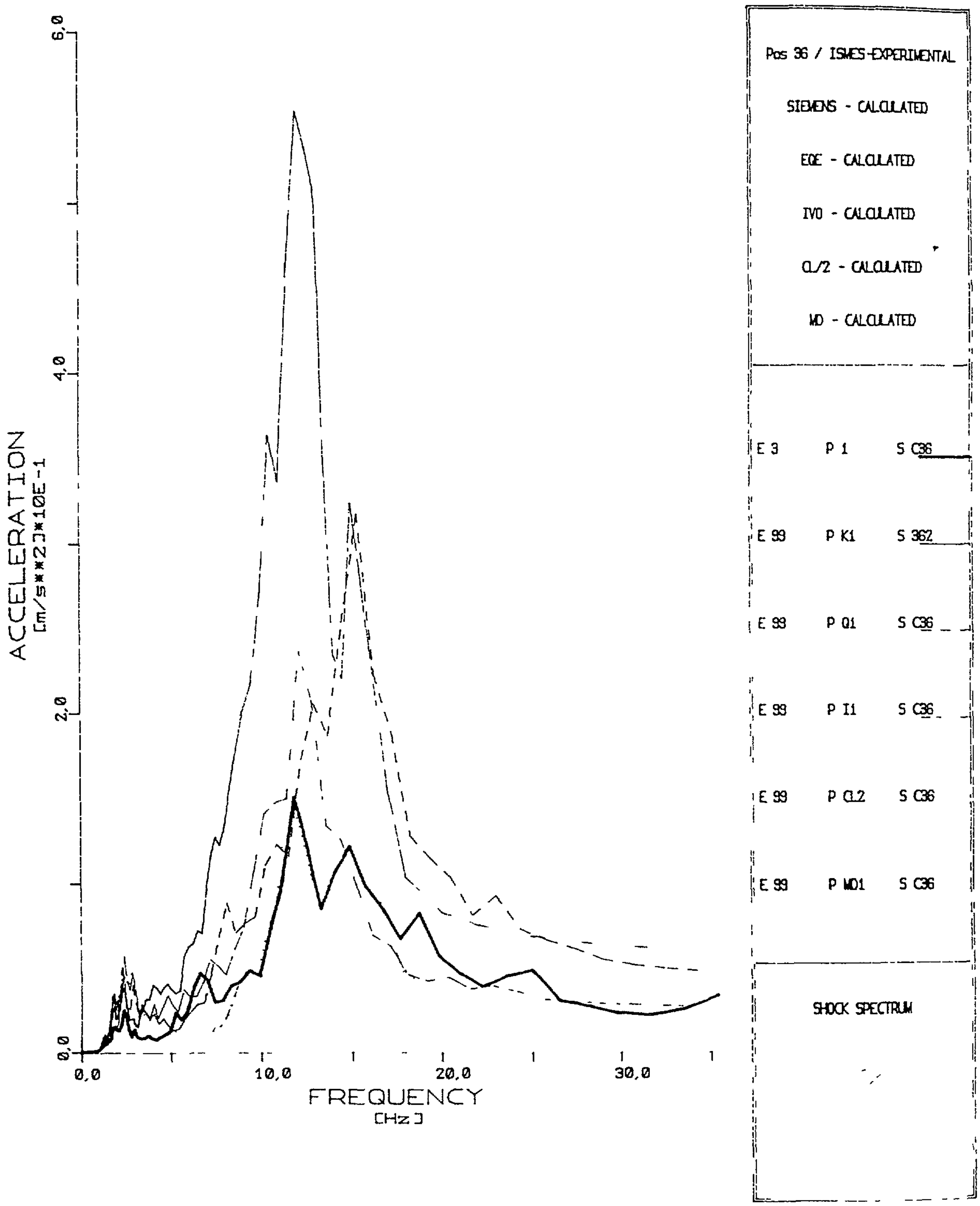


Fig. 10 Comparison between experimental and calculated response spectra at 2% damping ratio in position 36 (frequency range from 1 to 35 Hz)

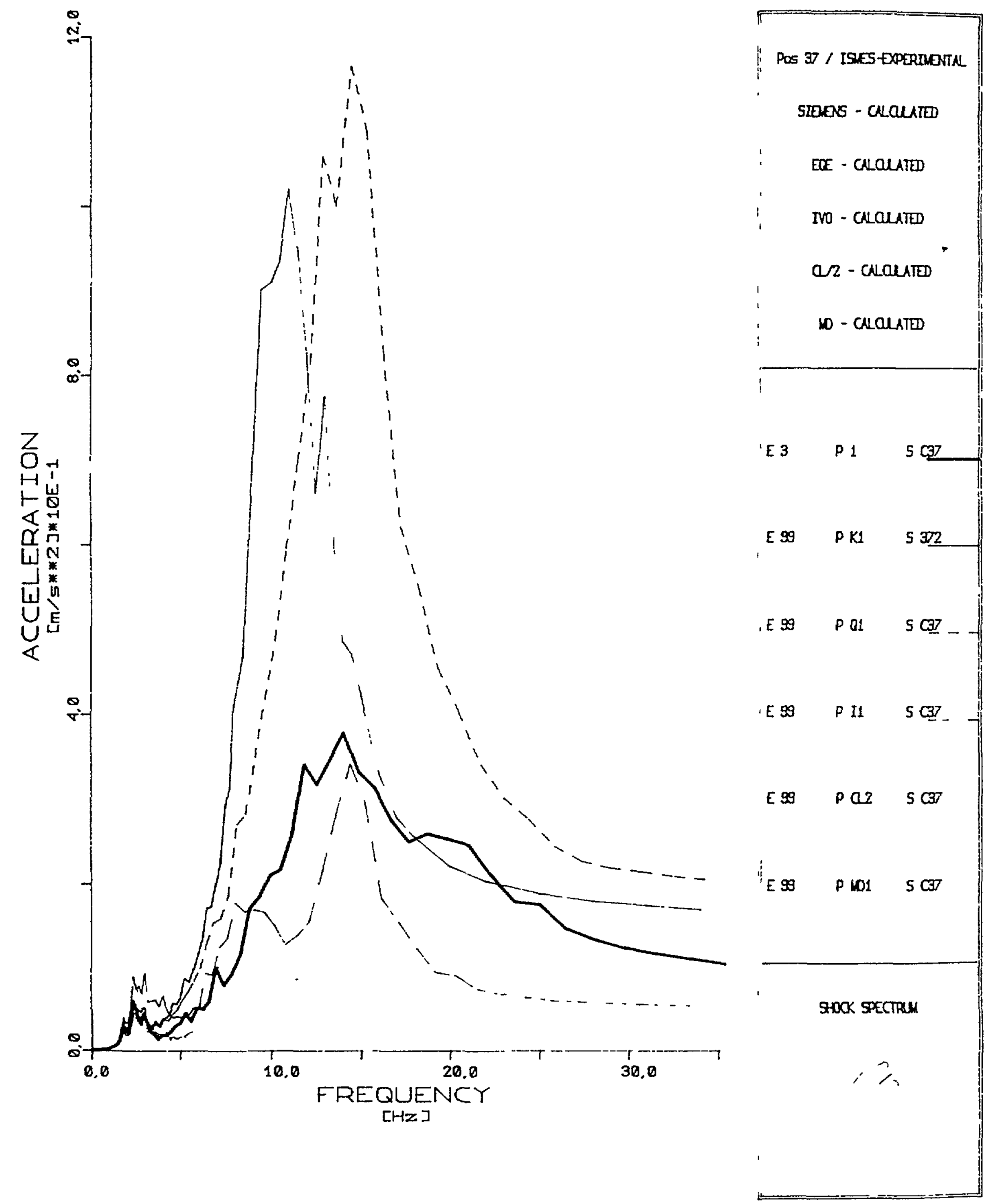


Fig. 11 Comparison between experimental and calculated response spectra at 2% damping ratio in position 37 (frequency range from 1 to 35 Hz)

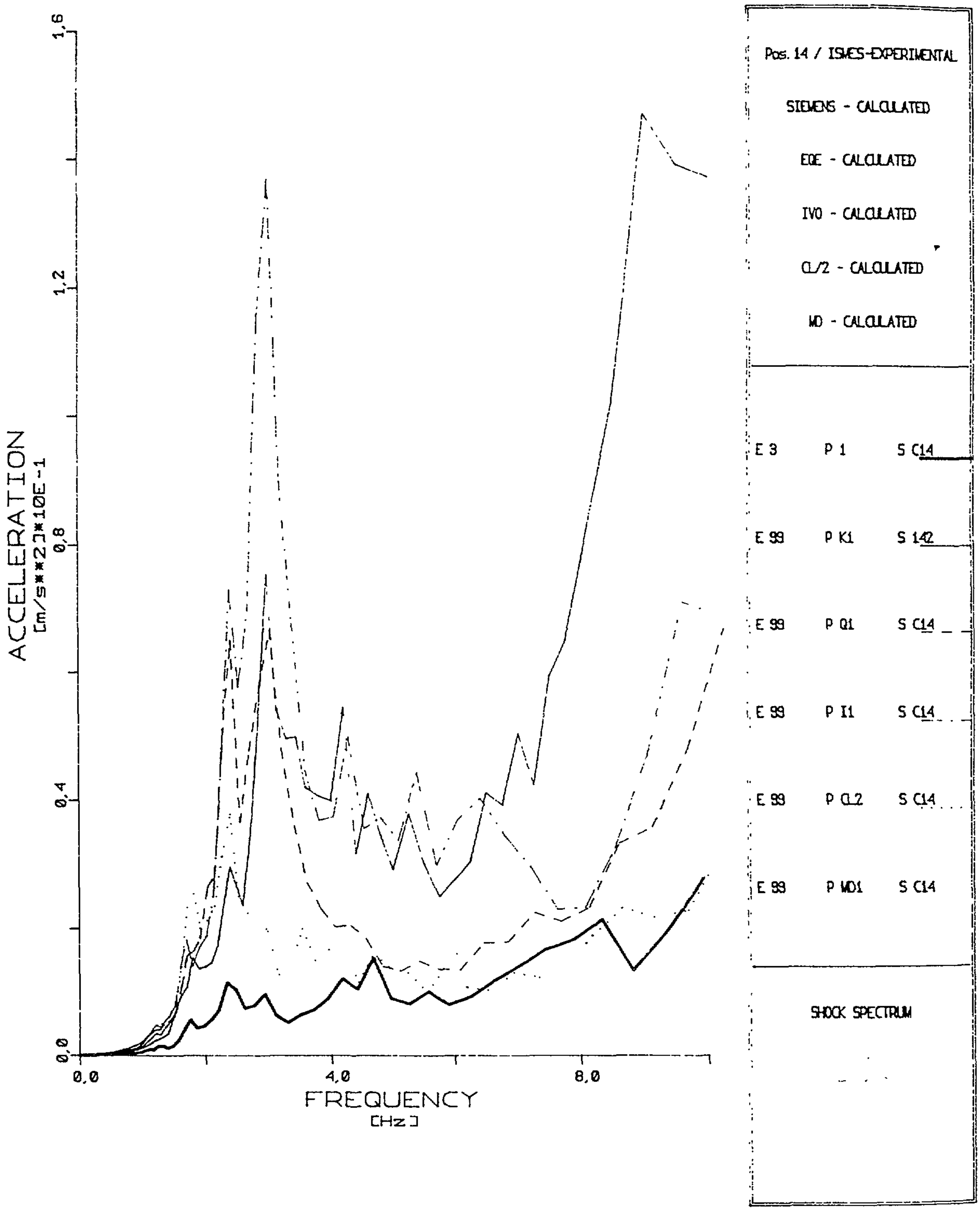


Fig. 12 Comparison between experimental and calculated response spectra at 2% damping ratio in position 14 (frequency range from 1 to 10 Hz)

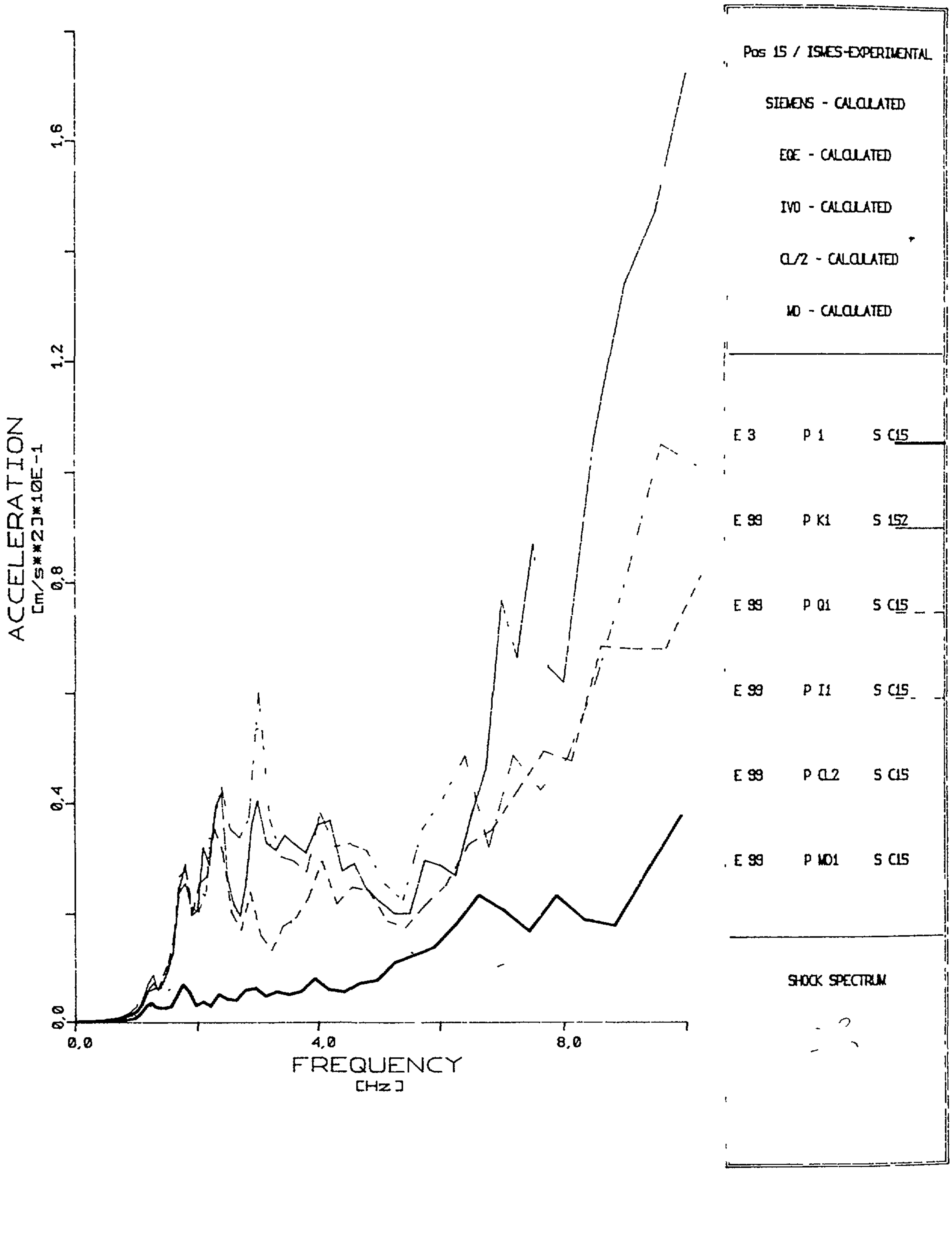


Fig. 13 Comparison between experimental and calculated response spectra at 2% damping ratio in position 15 (frequency range from 1 to 10 Hz)

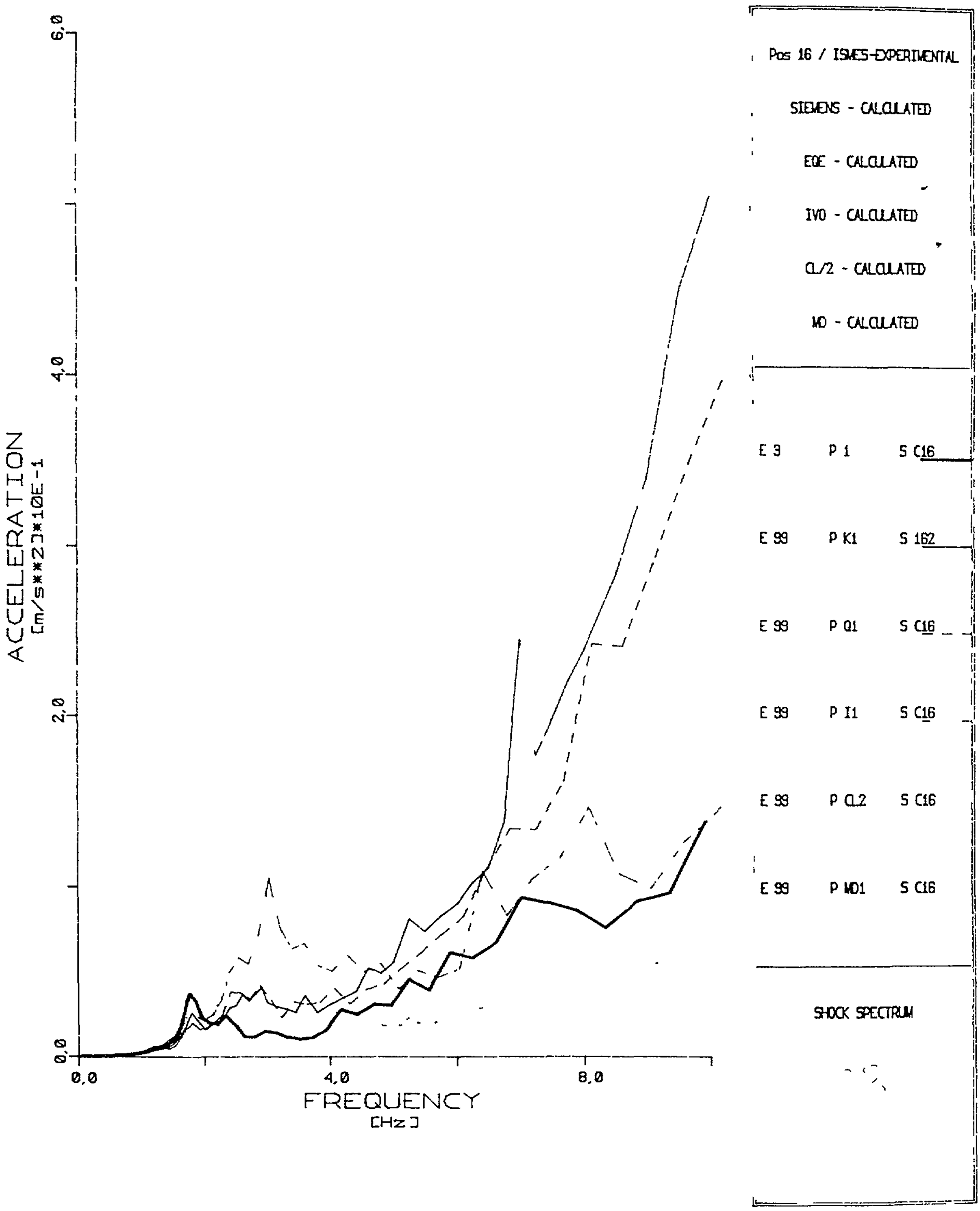


Fig. 14 Comparison between experimental and calculated response spectra at 2% damping ratio in position 16 (frequency range from 1 to 10 Hz)

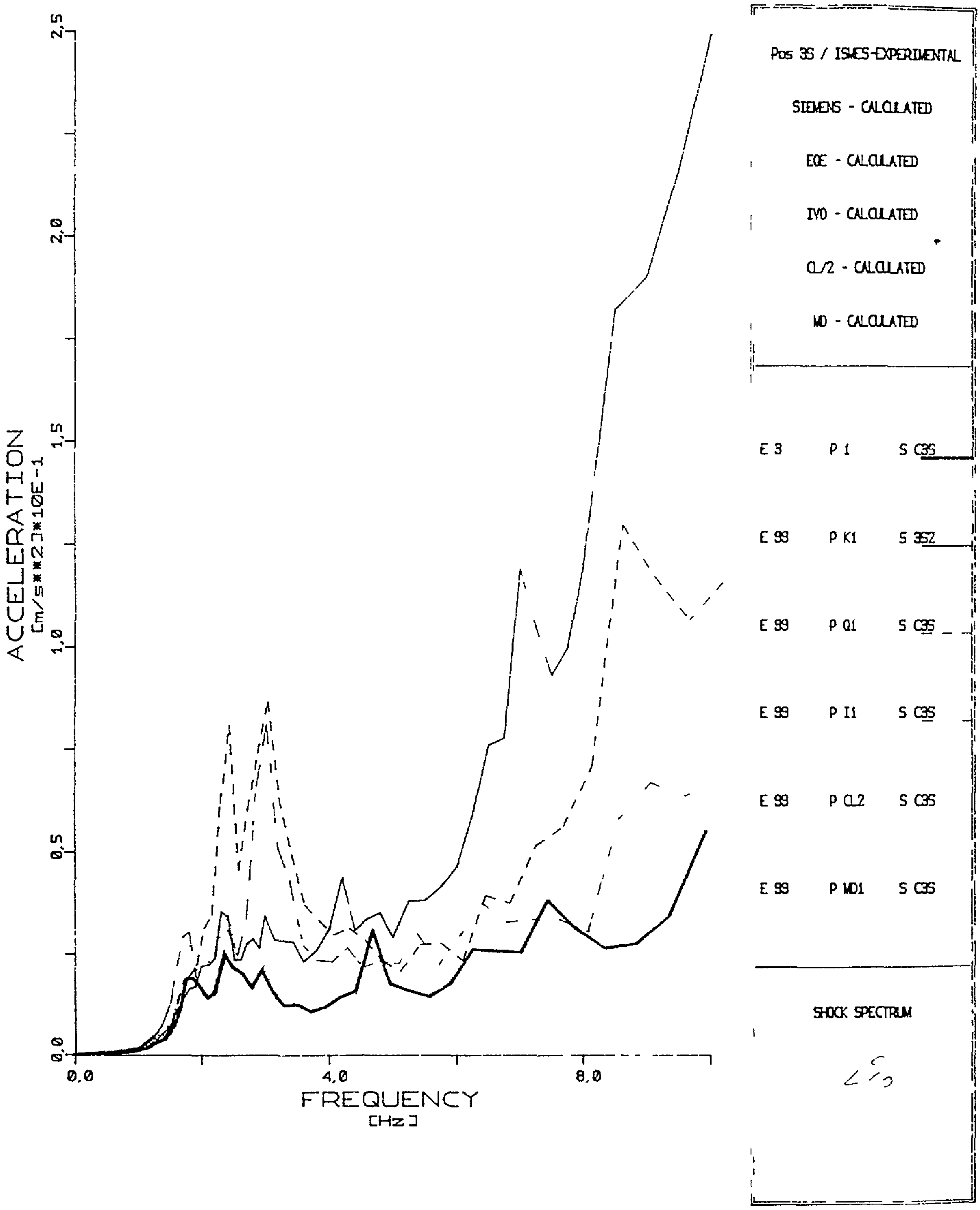


Fig. 15 Comparison between experimental and calculated response spectra at 2% damping ratio in position 35 (frequency range from 1 to 10 Hz)

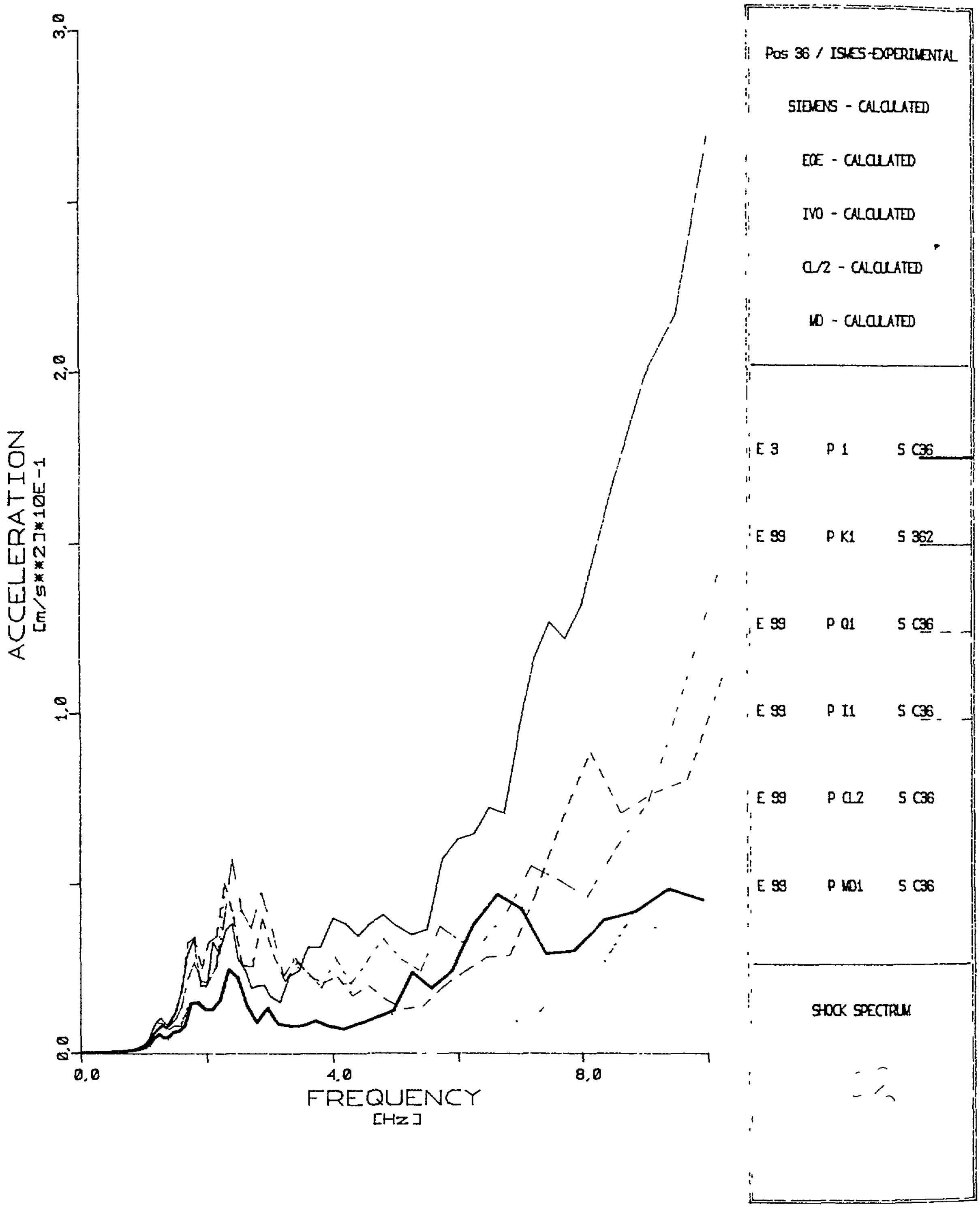


Fig. 16 Comparison between experimental and calculated response spectra at 2% damping ratio in position 36 (frequency range from 1 to 10 Hz)

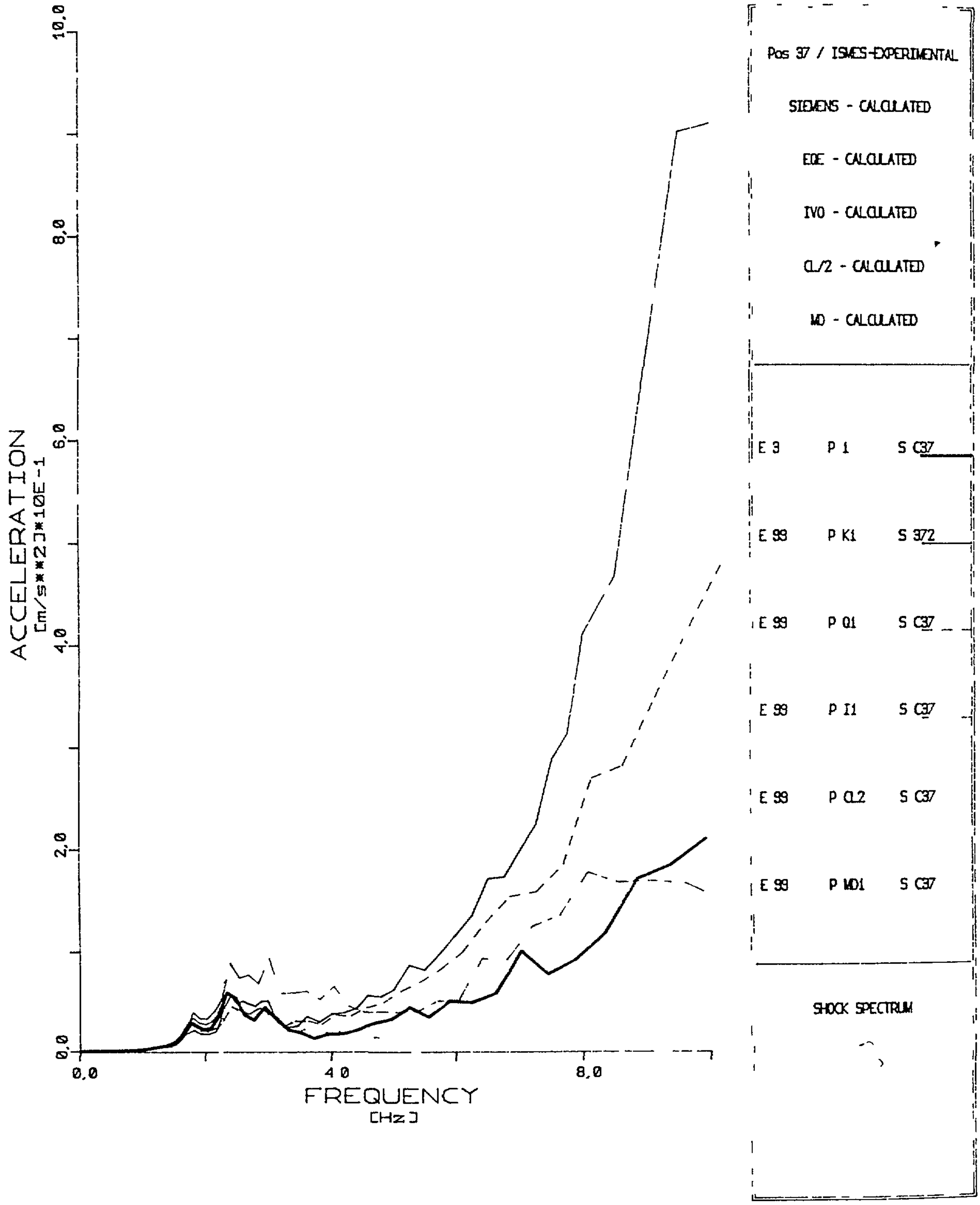
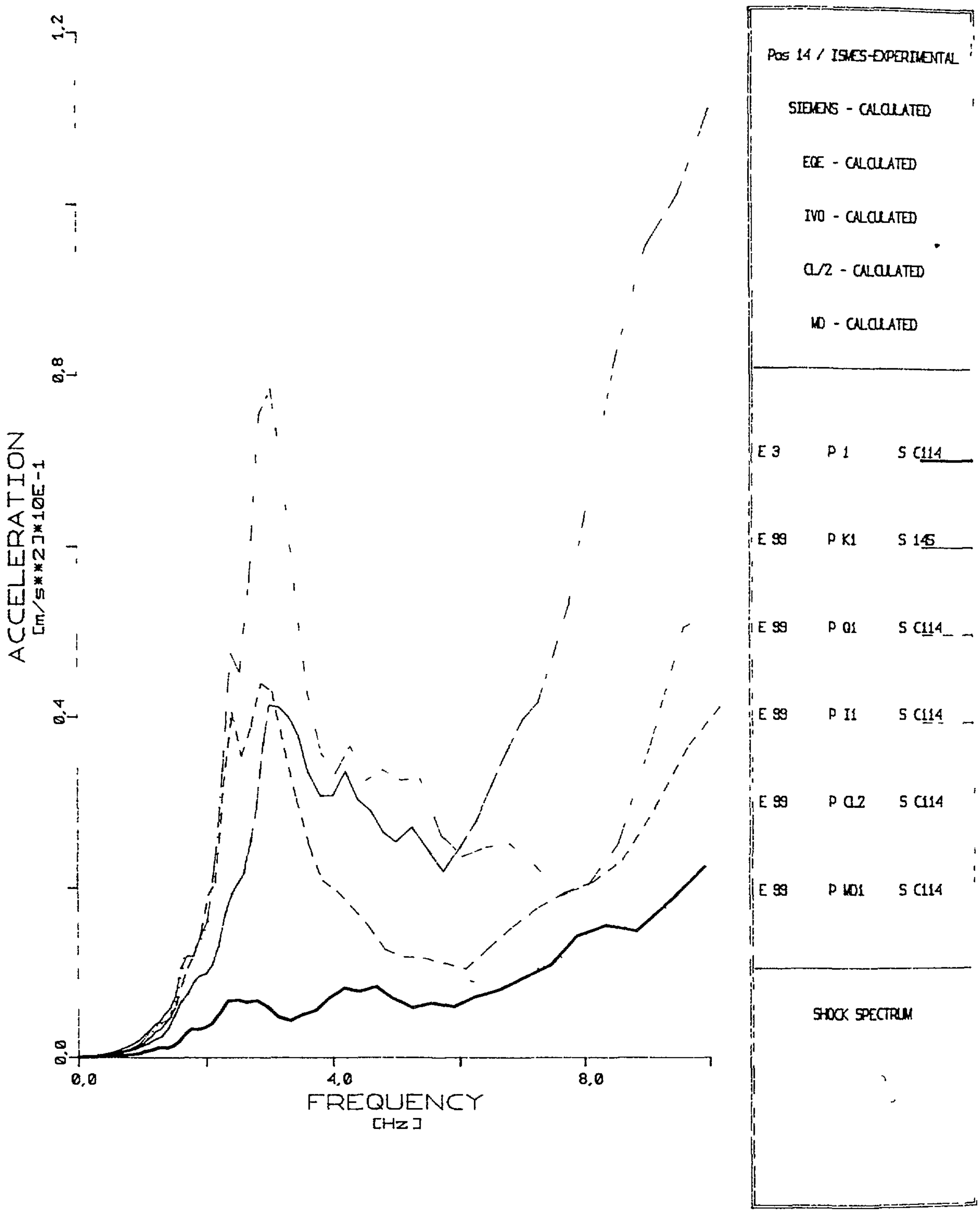


Fig. 17 Comparison between experimental and calculated response spectra at 2% damping ratio in position 37 (frequency range from 1 to Hz)



ISMES BERGAMO | FIG 18 | BENCHMARK REPORT ON PAKS

6. CONCLUDING REMARKS

Between 1993 and 1997 the IAEA co-ordinated the benchmark study for the analysis and testing of WWER Type NPP's, which included the dynamic testing investigation of the major structures of the Paks Nuclear Power Plant by means of buried explosions in order to induce earthquake-like ground motions. These tests have provided a large amount of useful field and structural response data. Blind prediction analyses performed on mathematical models by five different institutions have resulted in a detailed description of the structural response at a very large number of points.

A comparison of the measured and calculated structural response at a selected number of points was performed by ISMES on behalf of the IAEA.

The following general considerations can be drawn:

- Generally speaking the amplitudes of the calculated response spectra are higher than those obtained experimentally, at least for the frequencies above 8 Hz, while the shapes are more or less highlighting the experimentally determined frequencies.

- There is a high influence of the frequency energy content due to the nature of the explosion excitation in the frequency range around 15 Hz (as can be seen on the response spectra of the free field point, Fig. 4.) In order to get a more meaningful comparison of the spectra in the seismic range, the plots from 1 to 10 hz should be used for evaluation.

- It has to be expected that during a seismic event a higher excitation at the soil level will involve dissipating mechanism, both in the soil and in the structure, leading to higher values of the damping as well as inelastic deformation.

- With reference to the preceding remark, full scale dynamic tests provide a more refined estimate of the damping. This estimate, even if it is associated with lower excitation levels than seismic, is valuable in order to provide a lower bound to the damping values.

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SUMMARY OF FULL SCALE DYNAMIC TESTING OF PAKS NPP

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Abstract

Within the framework of the IAEA coordinated "*Benchmark Study for the seismic analysis and testing of WWER-type NPP's*", in-situ dynamic structural testing activities have been performed at the Paks Nuclear Power Plant in Hungary. The specific objective of the investigation was to obtain experimental data on the actual dynamic structural behaviour of the plant's major constructions and equipment under normal operating conditions, for enabling a valid seismic safety review to be made.

This paper gives a synthetic description of the conducted experiments and presents some results, regarding in particular the free-field excitations produced during the earthquake-simulation experiments and an experiment of the dynamic soil-structure interaction global effects at the base of the reactor containment structure. Moreover, a method which can be used for infering dynamic structural characteristics from the recorded time-histories is briefly described and a simple illustrative example given.

1. INTRODUCTION

An IAEA Coordinated Research Programme was initiated in the early nineties to assist the countries of Central and Eastern Europe in evaluating the actual safety conditions of their first generation nuclear power plants. This Programme fundamentally aims at providing technical bases to the safety related decisions to be taken by the countries operating the plants, with the consulting assistance of other countries providing technical and financial support.

Within the above-outlined context, a full-scale experimental investigation into the dynamic structural characteristics of a typical WWER-type Nuclear Power Plant has recently been performed at Paks in Hungary. Experimental data on the actual dynamic behaviour of the plant's major structures is obviously essential for validating computer models and allowing valid seismic safety analysis to be made. The Paks NPP site has thus been subjected to earthquake-like ground shaking through appropriately devised buried explosions - at a safe distance from the plant - and the dynamic response of the plant's major structures digitally recorded, together with the concurrent free-field excitation. The large amount of experimental data acquired during three successive earthquake simulation experiments is being analyzed for to extracting useful reference information.

2. PLANT AND SITE SHORT DESCRIPTION

There are presently four WWER-440 type V-213 reactor units in operation at the Paks NPP. The latter was originally designed in the former Soviet Union, but some adaptations were made by Hungarian design offices. The two first reactor units started commercial operation in 1983 and 1984.

In the design stage the seismic hazard of the Paks site was considered to be very low and thus no special regard was given to possible earthquake actions. Lately however, the seismic hazard of south-eastern Hungary is being revised and it was hence considered important that the seismic safety of the Paks NPP be rationally reviewed. The four reactors of the Paks NPP are arranged as two twins (Figure 1). The main building of each twin houses two reactor units in a symmetrical layout and is made up of a stiff reinforced concrete containment building, that is supported - together with an adjacent condensation tower - on a 2m thick continuous direct foundation slab. The foundation soil is a rather soft one, being composed of alluvial silts, sands and gravels becoming dense at around 16m depth.

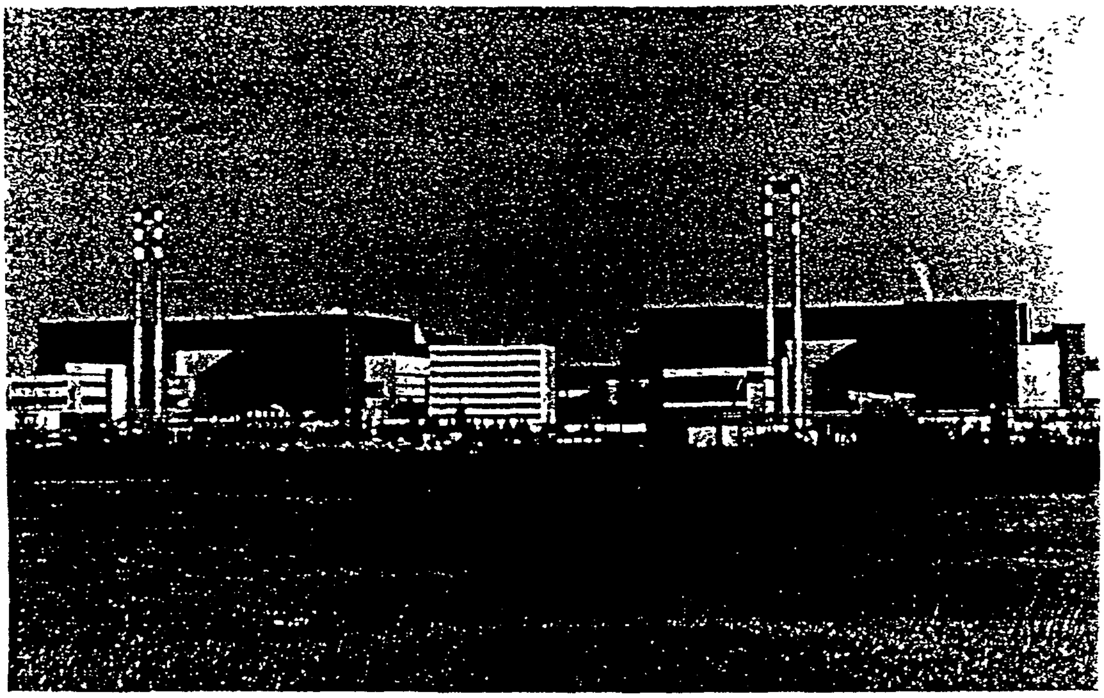

Figure 1. General view of the Paks Nuclear Power Plant.

3. SIMULATED EARTHQUAKE EXCITATION TESTS

The Paks NPP site was thus subjected to the effects of appropriately designed buried explosions, with the object of inducing an earthquake-type excitation of the plant's structures. By transmitting the vibratory energy to the structures through their own foundation soil - as actually occurs during real earthquakes - the full-scale dynamic soil-structure interaction effects are activated and can hence be realistically investigated.

Three different successive earthquake simulation experiments were performed at the Paks site, with the whole nuclear power plant under normal operating conditions. The experiments were performed by igniting TNT charges, installed in 50m deep boreholes at an overall horizontal distance of about 2,5km from the 1st unit reactor base centre.

- The first of the three experiments was a single blast one, which allowed to evaluate the blast-induced vibrations intensity and to conveniently calibrate the dynamic range of the measurement instrumentation.
- Subsequently, two time-delayed multiple blasts were produced, with the object of somewhat lengthening the ground excitation duration.

In fact, the duration of real earthquakes is obviously longer than that produced by a single underground explosion; but, even more important in the present context, a higher frequency resolution can be used in extracting structural behaviour information from the experimental records, if the latters are of longer duration. Each one of the earthquake excitation tests comprised a different layout of the measurement instrumentation, for the scope of acquiring a comprehensive experimental data set on the structural response of all the power plant's major constructions.

A large number of dynamic transducers were installed at appropriate locations in the nuclear power plant's structures. A series of sensitive velocity transducers (seismometers) were fixed against the reactor building foundation mat; in particular, three vertical and two horizontal sensors were set up around the base of the reactor shaft massive containment structure, as shown in Figure 2, and a number of identical sensors were installed at the upper reactor hall floor level.

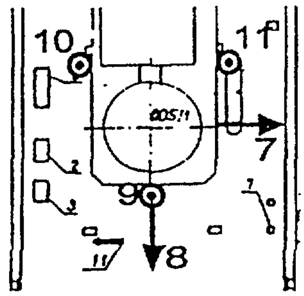

Figure 2. Measurement stations around the reactor shaft base.

For measuring the actual free-field excitation produced by the earthquake simulation experiments, three further seismometers were buried 1m deep into the natural soil at a 120m lateral distance aside the reactor base centre. Moreover, a series of piezoelectric accelerometers were used for measuring the vibrations at the upper levels of the reactor hall steel superstructures and close to the top of the nearby reinforced concrete twin chimneys. For the synchronous recording of all the structural response data, together with the concurrent free-field excitation, use was mad of an advanced multichannel data acuisition and analysis system, developed by ISMES and the hardware of which was set up in a mobile laboratory, parked beside the reactor containment building. This system is capable of simultaneously recording up to 52 signals at a 200Hz sampling frequency, with real-time analog to digital conversion; it is a submodule of "AIACE" (the Advanced ISMES Acquisition, Analysis and Control Environment), which was specifically developed for performing static or dynamic experiments, while providing also ample data analysis capabilities. In the case of time-history data to be collected, the acquisition process can be automaticallly triggered according to a specified criterion; data from all the connected transducers are fed to signal conditioners which, after on-line A/D conversion drive directly into the computer memory. At the end of the data acuisition process, the collected data are ready for graphical examinations by means of various plotting functions, as well as for applying time or frequency domain signal analysis procedures.

4. EXPERIMENTS PERFORMED AND RESULTS OBTAINED

As already outlined above, three different blast-induced ground excitation tests were performed at the Paks NPP site, with the plant in normal operating conditions. During each single experiment 52 digitized response signals were simultaneously recorded at a 200Hz sampling rate. Analogic low-pass filters were used for eliminating the high frequency noise prior to digitizing.

A preliminar test was carried out by simultaneously detonating two 50kg charges in 50m deep boreholes at a 2442m distance in the SSE direction from the NS oriented first reactor building. Subsequently, two time delayed multiple blasts experiments were performed with the scope of lengthening the overall excitation duration. The first multiple blasts experiment was carried out by detonating three 100kg charges, with two 1,64sec delays, at practically the same mean horizontal distance from the reactor building than before. A second multiple blasts test was later performed with two 150kg blows and a 1,58sec delay. Figure 3 shows the three-orthogonal velocity time-histories that were recorded in the free-field during the triple delayed blasts experiment.

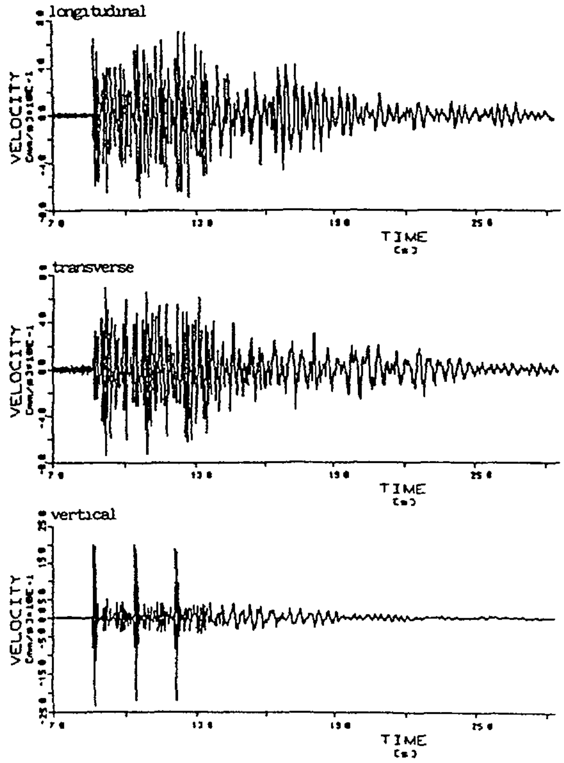

Figure 3. Free-field response records.

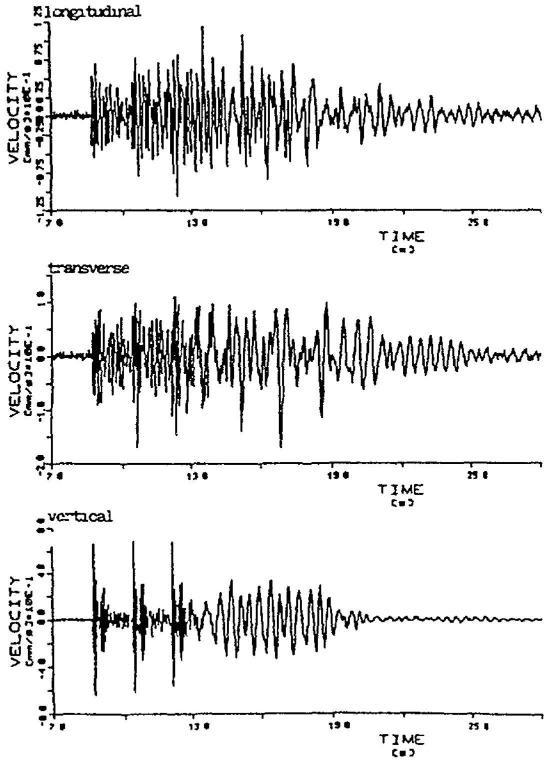

Figure 4. Reactor base response records.

The Paks NPP site appears to have been significantly excited by the buried explosions; about 20 sec long useful response signals were obtained. The free-field records show two consecutive rather distinct high and low frequency excitation phases, separated by an intermediate interference period. The following maximum peak velocities were recorded in the free-field during the respectively higher and lower frequency excitation phases:

- 0,081 and 0,071cm/s in the horizontal directions,
- 0,287 and 0,058cm/s in the vertical one.

These values are well below the 0,5 and 0,3cm/s conservative foundation velocity limiting values that are recommended in the DIN4150/3(1983) Standard for preventing any damage to occur in the case of blast induced vibrations in a "particularly sensitive building environment". The corresponding maximum peak horizontal accelerations are close to that of a M.M. grade III intensity earthquake, characterized by maximum horizontal accelerations up to 0,002g.

5. REACTOR SHAFT RESPONSE

Figure 4 shows the time-histories that were recorded during the triple blasts experiment at the reactor shaft base (see Figure 2) in the longitudinal, transverse and vertical directions. The reactor shaft base responses (recorded at the reactor building foundation slab level) appear to be significantly lower than the corresponding free-field excitations; with the exception of the lower frequencies vertical vibrations, which show to maintain almost the same amplitudes - however with a slower decay - at the reactor base than in the free-field. Just a slight amplification of the vertical response was measured around the metallical top of the reactor shaft, suggesting that a prevailingly "rigid" vertical response of the latter occured.

More detailed observations can be made by comparing the response spectra of the reactor shaft base induced motions to those of the free-field excitation. For that purpose, the 2% damping pseudovelocity response spectra were computed in the 1-100Hz frequency range for the excitations that were simultaneously recorded in the free-field and at the reactor base. These pseudovelocity spectra can be considered to reflect the amount of energy content that is present in the recorded motions at the various frequencies. The free-field and the reactor base response spectra of the triple blasts ground excitation records are shown in Figures 5 and 6.

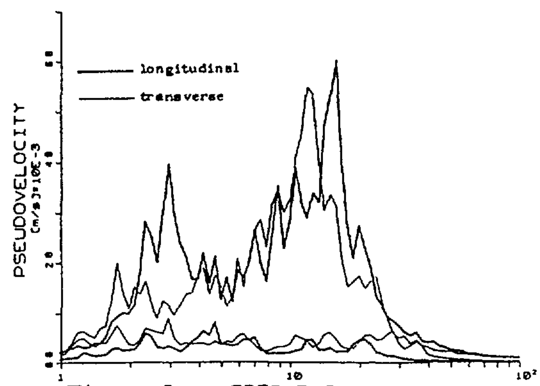

Figure 5.

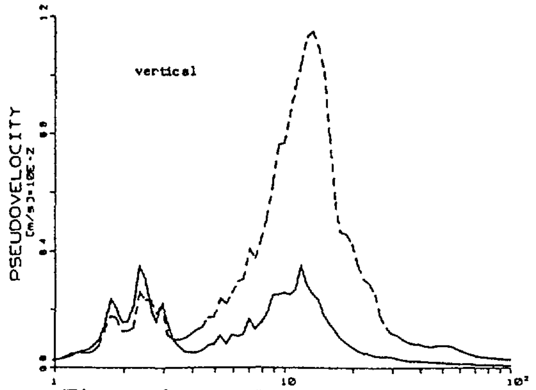

Figure 6.

Free-field (above) and reactor base (below) pseudovelocity response spectra.

While in the first diagram the spectra of the corresponding horizontal motions can easily be compared, the second diagram shows the difference in the vertical free-field and reactor base motions response spectra. It clearly appears that:

- The spectra computed from the horizontal motions at the reactor shaft basement are both well below that of the corresponding free-field excitations.
- The same observation holds for the vertical excitation in the higher frequency range. Around 2Hz however the reactor base vertical motion spectrum exceeds the free-field one; two small peaks are noticed at 1,75 and 2,34Hz.

These important observations indicate the activation of favourable dynamic soil-structure interaction effects: the thick reinforced concrete continuous foundation slab of the reactor containment building succeeds in remarkably attenuating the earthquake-like excitation levels: The horizontal excitation energies at the reactor shaft base show to be drastically attenuated over the whole frequency range in comparison to the free-field excitation and a considerable vertical vibration energy cut off is achieved above 3,12 Hz; below the latter frequency, however, the excitation energy of the reactor base is somewhat amplified with respect to the free-field one.

6. CONCLUSIVE CONSIDERATIONS

The IAEA promoted dynamic testing investigation of the Paks NPP site by means of buried explosions-induced ground motions has provided a large amount of interesting data on the structural response of the plant's major constructions. The technique used by the Hungarian mining specialists for carrying out the underground explosions actually succeeded in producing an earthquake-like excitation of rather low but quite well measurable intensity. High quality digital data acquisitions were made by means of the ISMES dynamic measurement instrumentation and data acquisition system.

A first series of analyses of the experimental data has recently been performed for examining the free-field excitations that were actually produced during the blast-induced ground shaking experiments and interesting information on the actual dynamic soil-structure interaction effects could be infered for low level seismic-like excitation. A further detailed analyses task has still to be conducted for extracting information on the structural characteristics and behaviour of the Paks NPP major constructions.

For determining the actual modal characteristics (f_n, ϕ_n and ξ_n), energy spectral density analyses [/1/] can be made of the collected data. As a simple illustrative example, the energy auto- and the cross-spectral density diagrams of the twin chimneys' top responses to the ground excitation are reproduced in Figures 7 and 8.

The auto-spectral density function, describes the vibration intensity (the variance of the measured quantity) distribution in the frequency domain and thus allows to identify the structural resonance frequencies at its peak values; moreover, the associated structural damping ratios can be estimated from the peak widths. On the other hand, the cross-spectral density function describes the frequency domain distribution of the covariance of the measured quantities in two different stations. The real (coincident) part of this function clearly shows the in- or out of phase relationships of the motions.

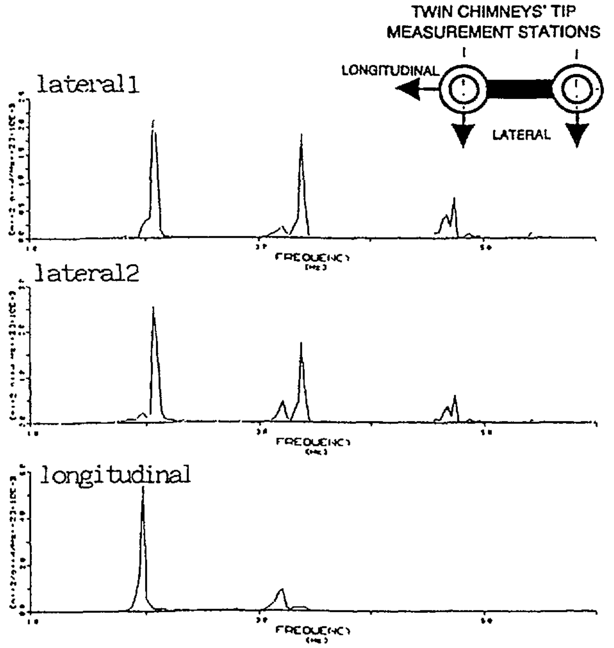

Figure 7. Energy auto-spectral densities of the twin chimneys' tip transverse and longitudinal responses

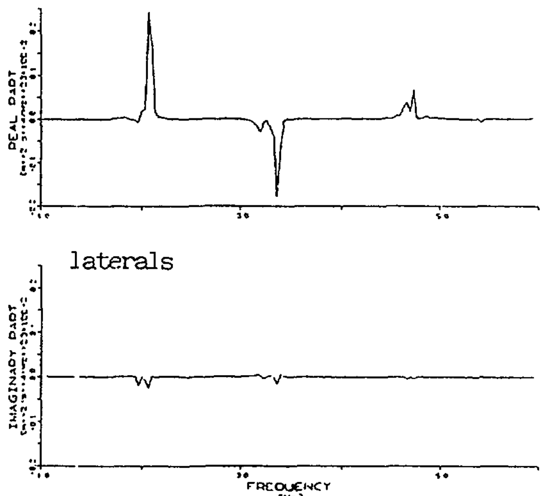

Figure 8. Energy cross-spectral densities of the twin chimney's tip transverse responses

From the above-reported diagrams, it can be concluded that:

- The first two longitudinal bending resonance frequencies of the twin chimneys are at 1,97Hz and 3,2Hz, with a further minor resonance frequency located around 4,6Hz;
- The first two synchronous lateral resonance frequencies of the chimney stacks are at 2,07Hz and 4,73Hz, while the first alternate lateral motion resonance occurs at 3,37Hz.

ACKNOWLEDGEMENTS

The positive attitude of the Paks NPP people (in particular: Dr. T. Katona, Chief Engineer and Dr. L. Turi, Head of the Experimental Section) during the preparation and execution of the above-described tests is gratefully acknowledged. Special thanks also to Dr. I. Szücs for the collaboration in the design of the multiple blasts experiments.

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[/1/] Bendat J.S., Piersol A.G.
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SUMMARY OF STRUCTURAL ANALYSIS AND COMPARISONS WITH EXPERRIMENTAL RESULTS FOR WWER 440/213 NPP PAKS

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XA0100513

Abstract

Final full scale experimental tests were performed for the VVER 440/213 Paks by ISMES Bergamo (under order from IAEA) in December 1994. Similar tests however were also carried out earlier (in 1990/91) within the framework of preliminary investigations of the seismic capacity of the VVER 440/213 PAKS initiated by the plant operator (PARt). In order to predict in advance the measured results by analytical procedures on the one hand and demonstrate the appropriateness of studying the earthquake-induced dynamic response of such complex structures on the other hand, blind preanalyses were performed in both cases before beginning the tests using various types of mathematical models and input data (discretization ratio of the structures, representation of soil capabilities, damping capacity of the complex vibrating system).

This paper presents the analytical and experimental results obtained by the blind preanalyses performed on the basis of the latest tests (12/94) and, for comparison, the results derived by the earlier tests (1990/91) are demonstrated.

INTRODUCTION

The results of the blind preanalysis related to the latest (12/94) tests were documented in References /6/ and /8/. In Reference /8/, however, preliminary comparisons were performed on the basis of experimental results provided by ISMES after the final analytical results of all participants on the benchmark studies had been submitted with the coordinators of the benchmark studies (IAEA/ISMES).

The results of preanalyses related to the earlier investigations (1990/91) were documented in References /1/ to /3/. It should be pointed out that the basis of the earlier preanalyses and comparisons were preliminary input data related to soil capabilities and the mathematical models based on the information given in the drawings. In contrast, the latest blind preanalysis are based on revised and updated soil data /4/ as well as on an updated mathematical model considering the as-built conditions.

MATHEMATICAL MODELS

The description of the coupled vibrating structures (Figure 1) as well as the complex mathematical model used for the investigations was given in several References /1/ to /3/ and /6/ to /8/.

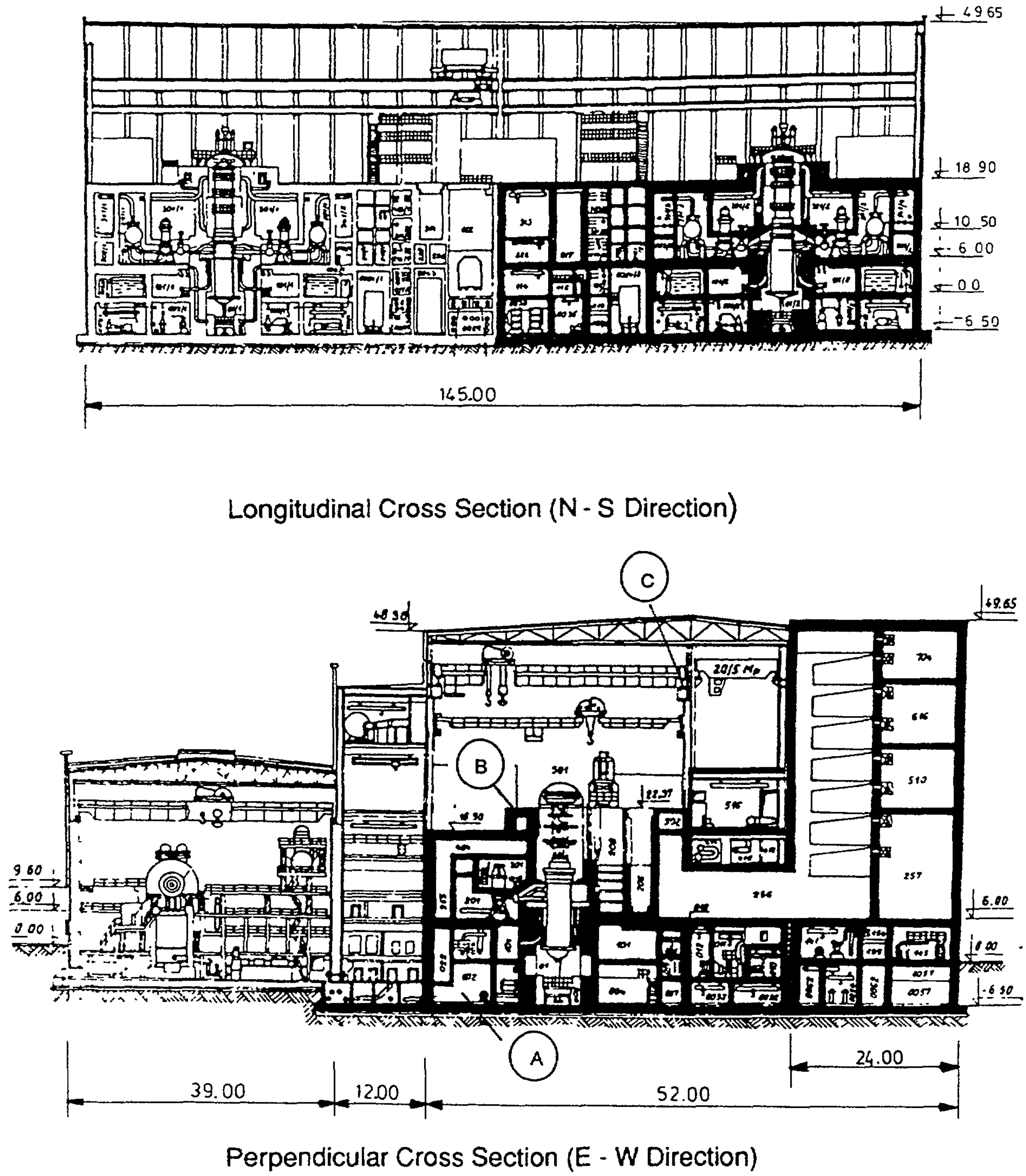

Fig. 1 Constructional Concept of a VVER-440/213

The 3-dimensional model (Figure 2) consists of 36000 DOF, the total weight is about 280 000 kN. The mathematical representation of the layered soil was based on the shear moduli and damping given /4/ for G_{max} (which are close to shear moduli for smallest strain, Figure 3). Impedance functions were calculated on the basis of the given soil layering for G_{max} and the assumptions of rigid capabilities of the individual foundations. Based on the impedance functions, global frequency independent stiffnesses and damping were derived matching the fundamental frequencies of the coupled soil structure system and finally distributed over all nodal points of the foundations.

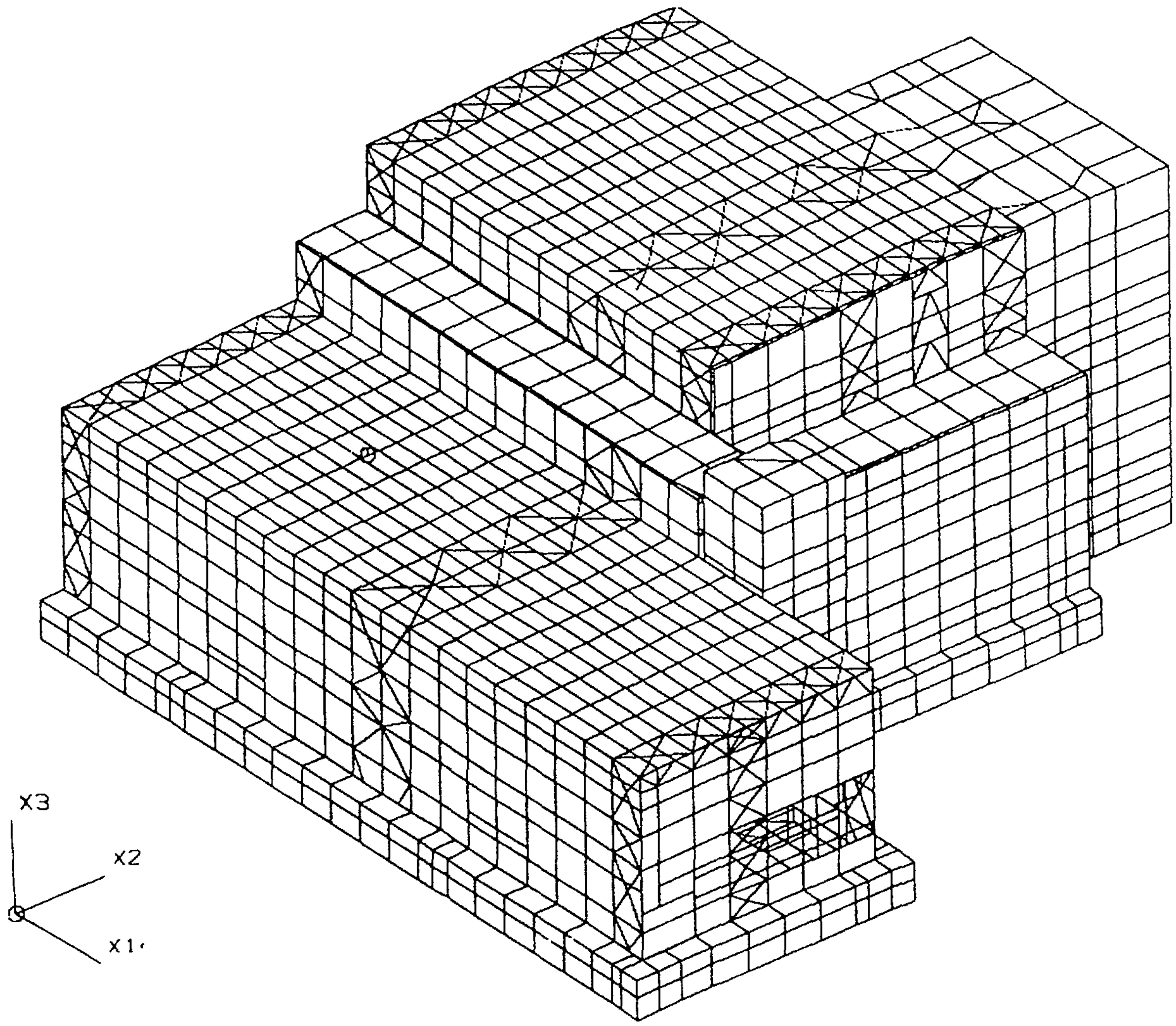

Fig. 2 Mathematical Model (3) of the main Building Complex Paks

LOADING FUNCTIONS

The free-field motion induced in the explosive tests measured at a distance of about 120 m (test of 12/94, location FF) and 240 m (tests performed 1990/91, location 2A) were defined as the input excitation for the blind preanalysis. In the earlier test series (1990/91) a number of explosive tests were performed using charges of 20 to 500 kg TNT located at a distance of 2.5 to 4.5 km (Figure 4).

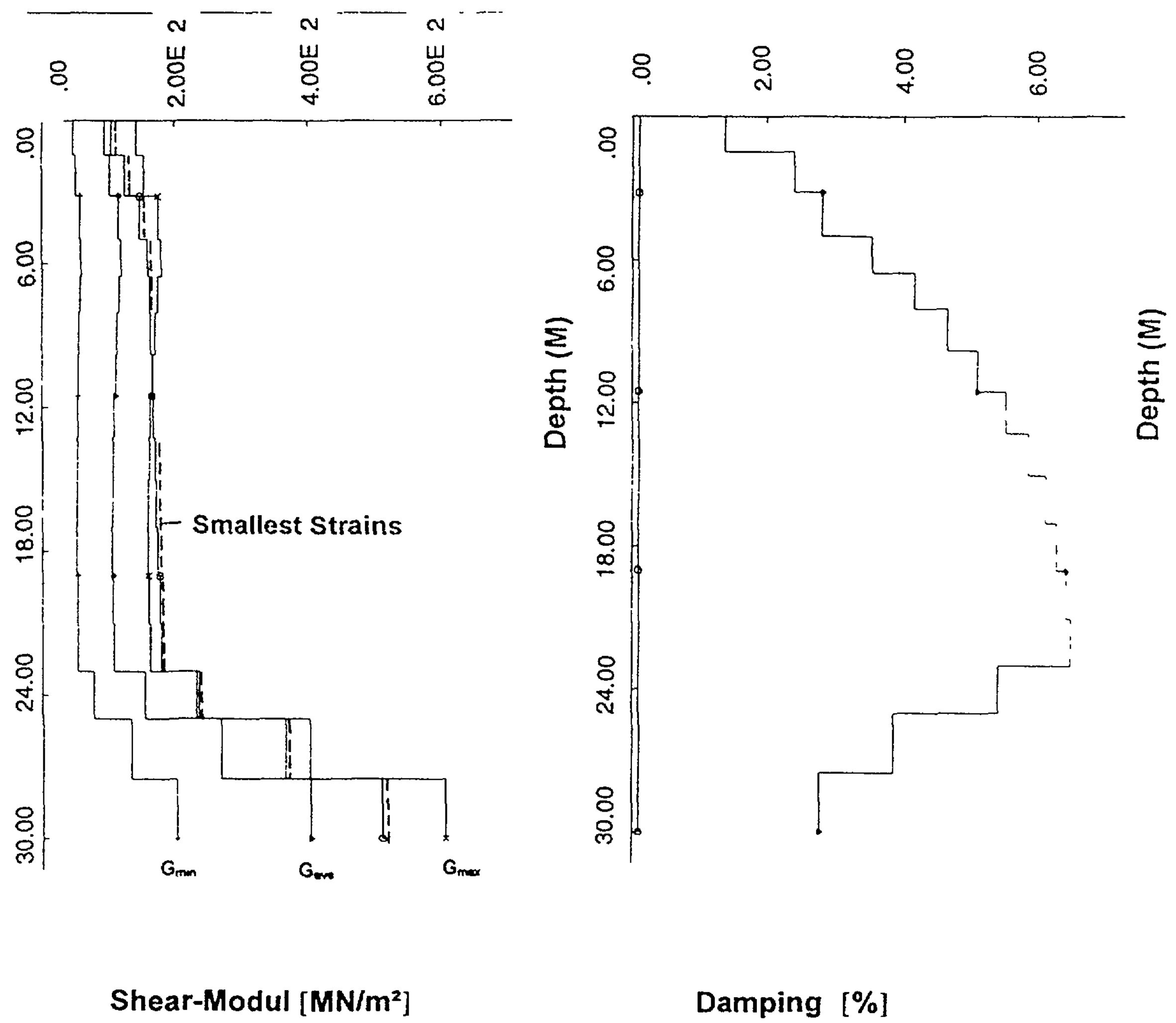

Fig. 3 Strain Compatible Shear Module and Damping

In the latest (12/94) tests, charges of 100 kg TNT located at a distance of about 2442 m from the center of the main building complex were used. The measured free-field time histories as well as the corresponding acceleration response spectra are shown in Figures 5 to 8).

Contrary to the earlier explosive tests (1990/91) using big charges (up to 500 kg TNT) detonated at the same point in time, the smaller charges (100 kg TNT) of the later (12/94) explosive tests (three 100 kg charges) were ignited sequentially (at intervals of 1.58 sec.). It can be observed that the shear waves generated by the bigger charge (of the 90/91 tests) contains free-field motions (Figures 7 and 8) which are able to excite the building structures in the frequency range of their fundamental frequencies. The later tests (12/94) contain a mixture of shear waves dominated by frequencies of about 12 - 15 Hz (Figure 6).

EVALUATION OF DYNAMIC CHARACTERISTICS OF THE COMPLEX SYSTEM

It is well known that a structural system excited by transient loading functions can (independently from the frequency content of a transient loading function) provide dynamic response results only in the frequency range of their eigen-frequencies and modes which are able to contribute to the vibration process, i.e. the eigenmodes of which modal parameters (modal masses, participation functions) are practically of importance.

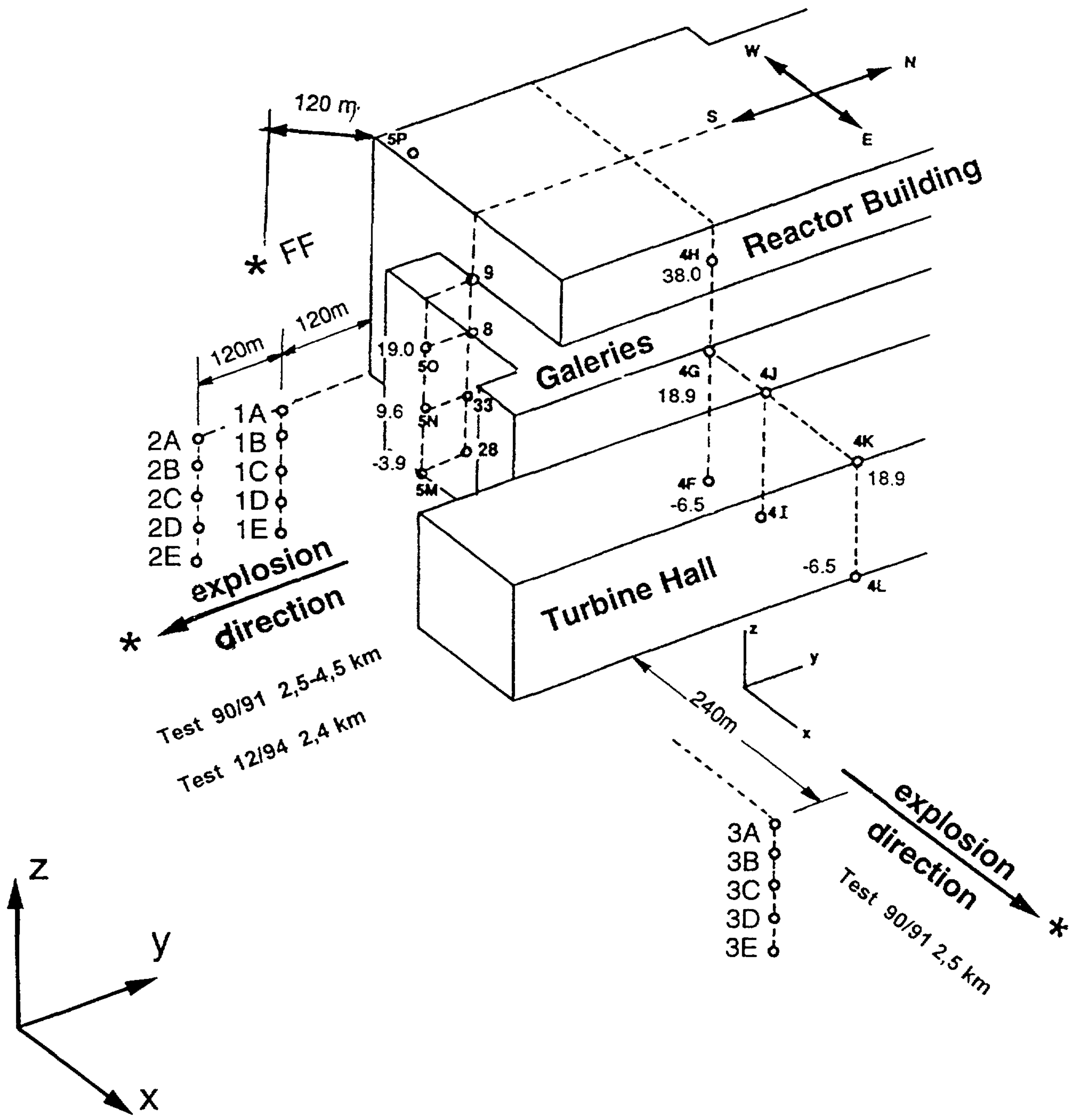

Fig. 4 PAKS Nuclear Power Station Site Arrangement of Measuring Points (2nd Series of Experiments)

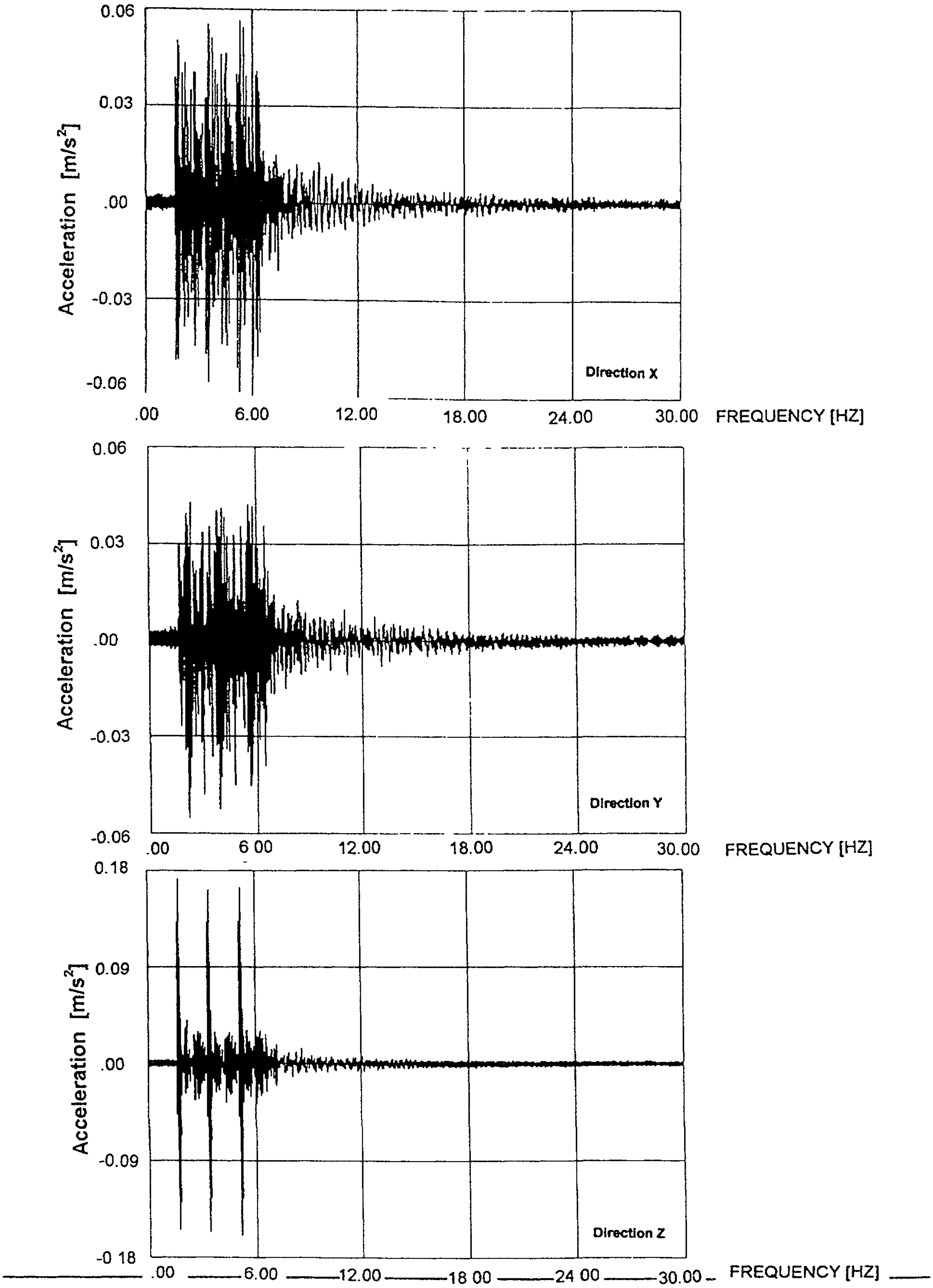

Fig. 5 Free-Field Acceleration Time Histories nearby the Reactor Building (Test 12/94,FF 3x 100 kg with Delay)

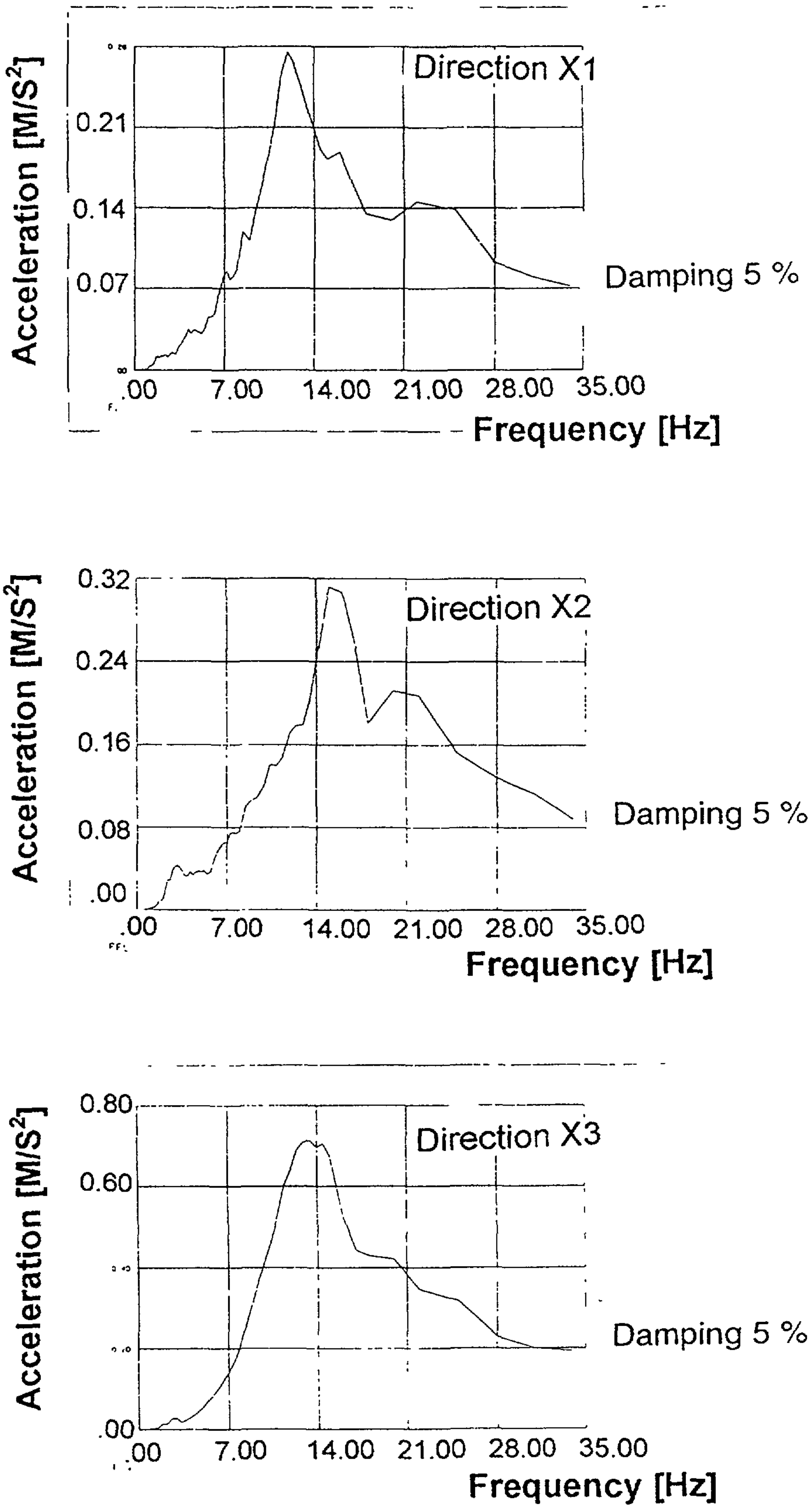

Fig. 6 Free-Field Response Spectra (2 % Damping) nearby the Reactor Building (Test 12/94,FF 3x 100 kg with Delay)

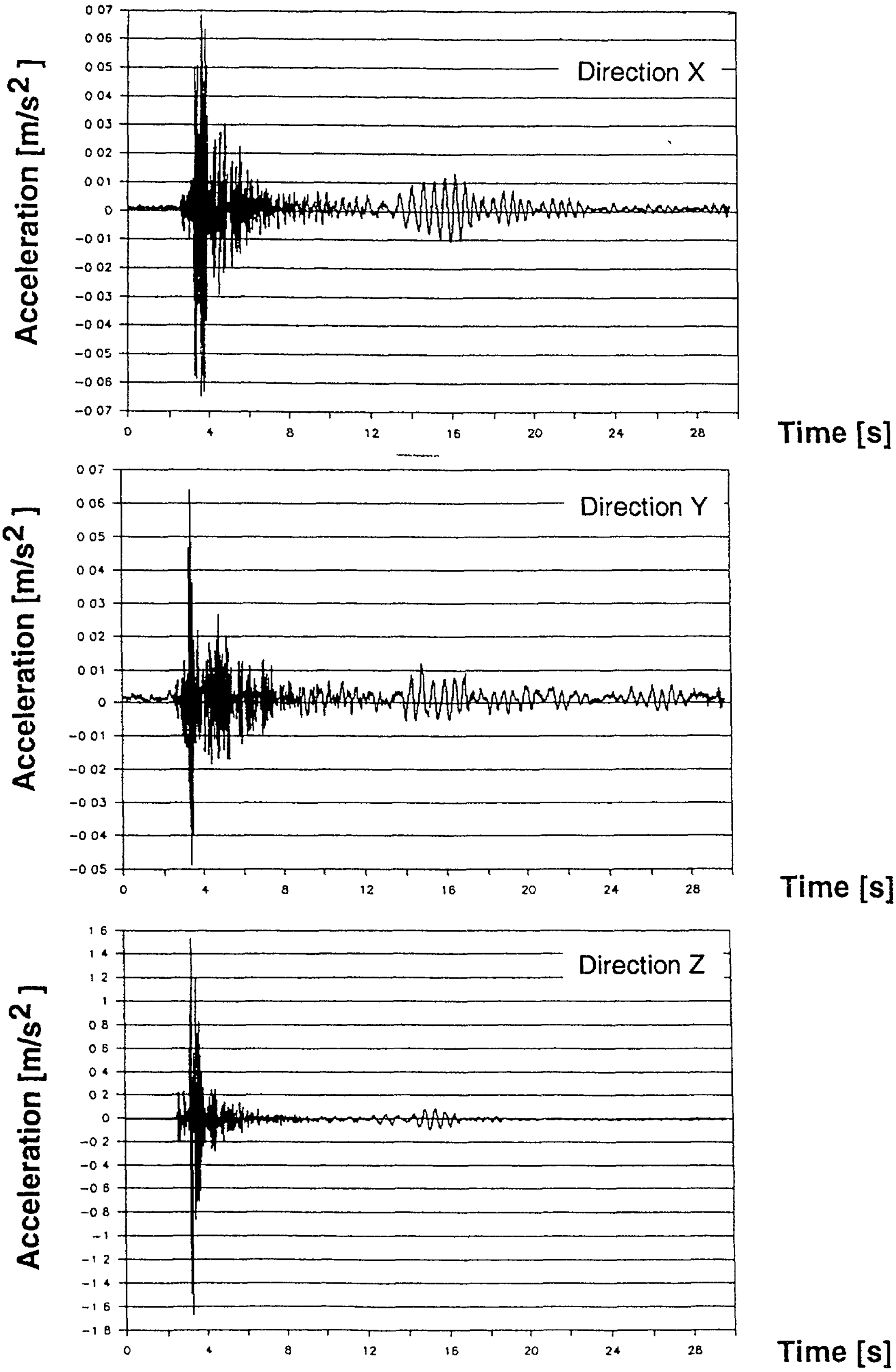

Fig. 7 Free-Field Acceleration Time Histories at 240 m from the Reactor Building (Test 90/91,1x 500 kg wihtout Delay)

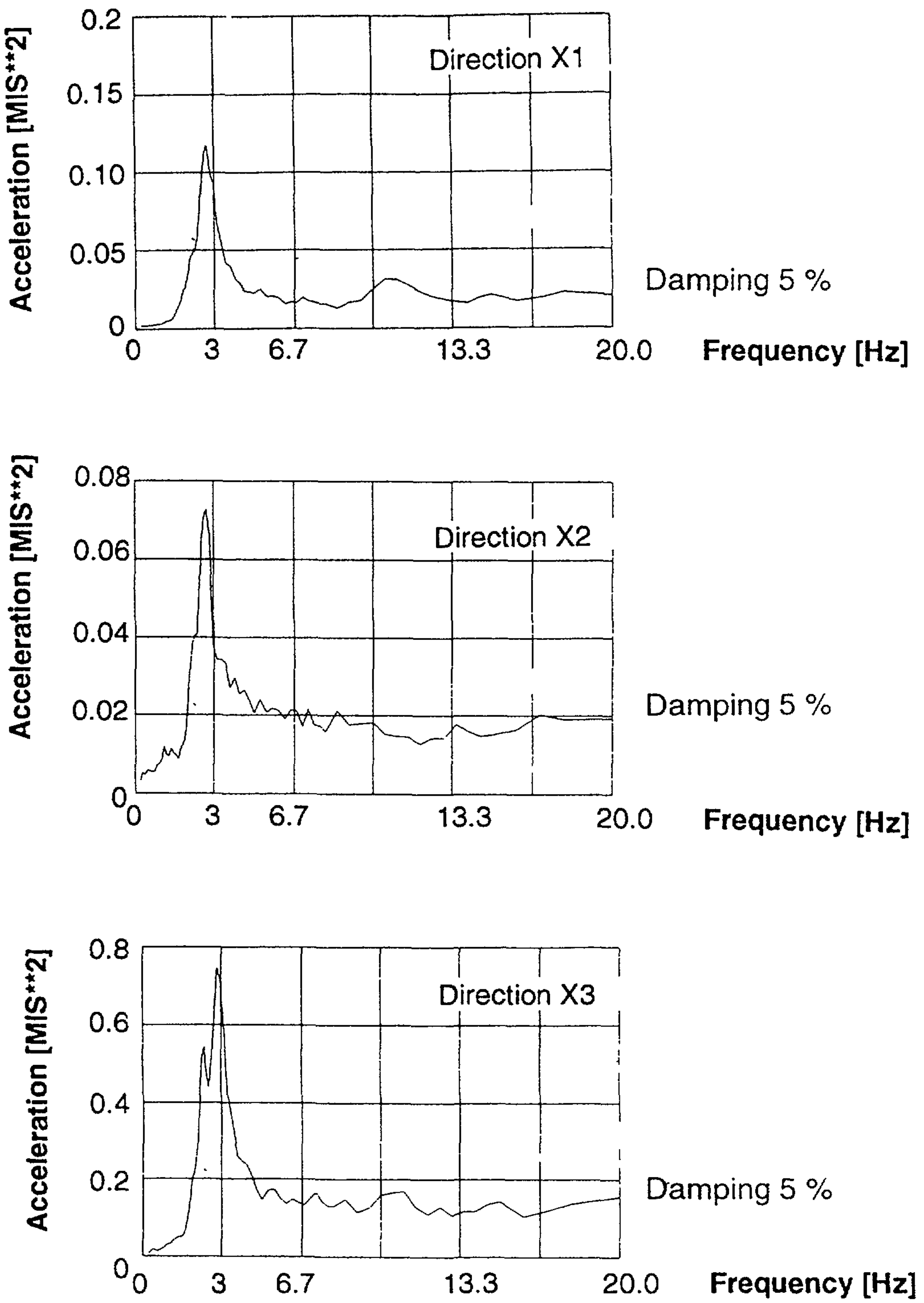

Fig. 8 Free-Field Response Sprectra at 240 m from the Iina Reactor Building (Test 90/91,1x 500 kg wihtout Delay)

When evaluating the eigenfrequencies and modes obtained for the shear modulus related to small strains (G_{max}) it can be observed that in the frequency range up to about 5 Hz the total modal masses have reached more than 94 %.

The outstanding modal masses are related to local vibration models of individual structural members as steel profiles and braces. The influence of these modes on the dynamic response of the concrete block (on the foundation level, on the upper level of the concrete block 18.9 m or on the level of the crane support) is negligible. Due to this fact (and bearing in mind the most relevant frequency content of the measured excitation of the later tests, Figure 6 is about 15 Hz) there is no possibility that the dynamic response of the structures of the main building (fundamental frequencies below 2.1 Hz for the horizontal and 4.8 Hz for the vertical direction respectively) can be changed significantly when considering further modes.

In the case of the later (12/94) tests, only the dynamic response calculated and measured by tests in the significant frequency range of the building structures (0 about 5 Hz) should therefore be compared and evaluated. The measured dynamic response (characterized practically by the highest amplification in the frequency range of 15 Hz) represents rather the effect of transfer of the shock waves introduced at the foundation level (soil) to the corresponding regions in the building (but not the capabilities of the structural model of the building) regarding their appropriateness for analysis of low frequency (seismic) loadings.

The same should be stated regarding the calculated results using the high frequency excitation. The analytically obtained response spectra are the results of filtering of the transferred shock waves by the mathematical model not designed for analysis of seismic excitation effects.

Much more effective for the excitation of the VVER 440/213 building structures were the shear waves generated during the earlier tests (1990/91). It can be observed (Figure 7) that after the high frequency content of the shear waves passed the building the following shear waves of much lower frequency content were able to excite the structures in the frequency range of their fundamental frequencies.

On the basis of the measured dynamic response result obtained during the earlier tests it was possible to identify the fundamental eigenfrequencies (peak frequencies), the damping effects (logarithmic decrement) as well as the scattering of the results due to variation of the soil data (Tables 1 and 2). The excitation produced during the later explosive tests (12/94) were in fact useful to qualify the appropriateness of the mathematical model for investigation of high frequency loading cases (aircraft crash, explosion).

The dynamic responses obtained using the excitation obtained by the earlier tests (1990/91) were therefore in surprisingly good agreement with the measured results.

Tab. 1 Comparison of Measured (1990/91) and Calculated Eigenfrequencies of the Reactor Building [2] with New Results (1997)

Mode No.	Eigenfrequencies [Hz]			
	Measured		Calculated	
	[2]	1997	[2]	1997
1	1.6-2.1	not	1.84	1.65-1.90
2	1.9-2.1	Published	2.12	2.22-2.33
3	2.3	till	2.38	2.33
4	2.5	now	2.82	3.77
5				4.36
20 vertical	3.9-4.1		4.16	2.07-4.07

Tab. 2 **Damping [%] Values Obtained from Measured Results (1990/91)**

Structure	Region	Explosion East			Explosion South		
		X	Y	Z	X	Y	Z
Reactor Building	4G	6.9		-		9.1	10.1
	4H	7.5	6.8			8.7	(5.5)
	5P	10.0	7.7		11.8	10.4	6.4
Turbine Hall	4K	6.4			9.8	9.4	8.1
Galleries	5O	6.1	9.4		8.4	9.2	14.8
Average Value		7.38	7.97		10.0	9.36	9.85

Damping Values used in the Calculation	X	Y	Z
	8	8	10

COMPARISON OF DYNAMIC RESPONSE RESULTS

When evaluating the response spectra of the free-field motion measured close to the reactor building (Figures 6 and 9) it can be recognized that the level of excitation in the frequency range of the fundamental eigenmodes of the concrete block (about 2.1 Hz) as well as the reactor hall (1.1 Hz) is only very low. It is therefore no surprise that only moderate response could be identified by measurements in the selected regions of the concrete block and the reactor hall (Figures 10 to 17).

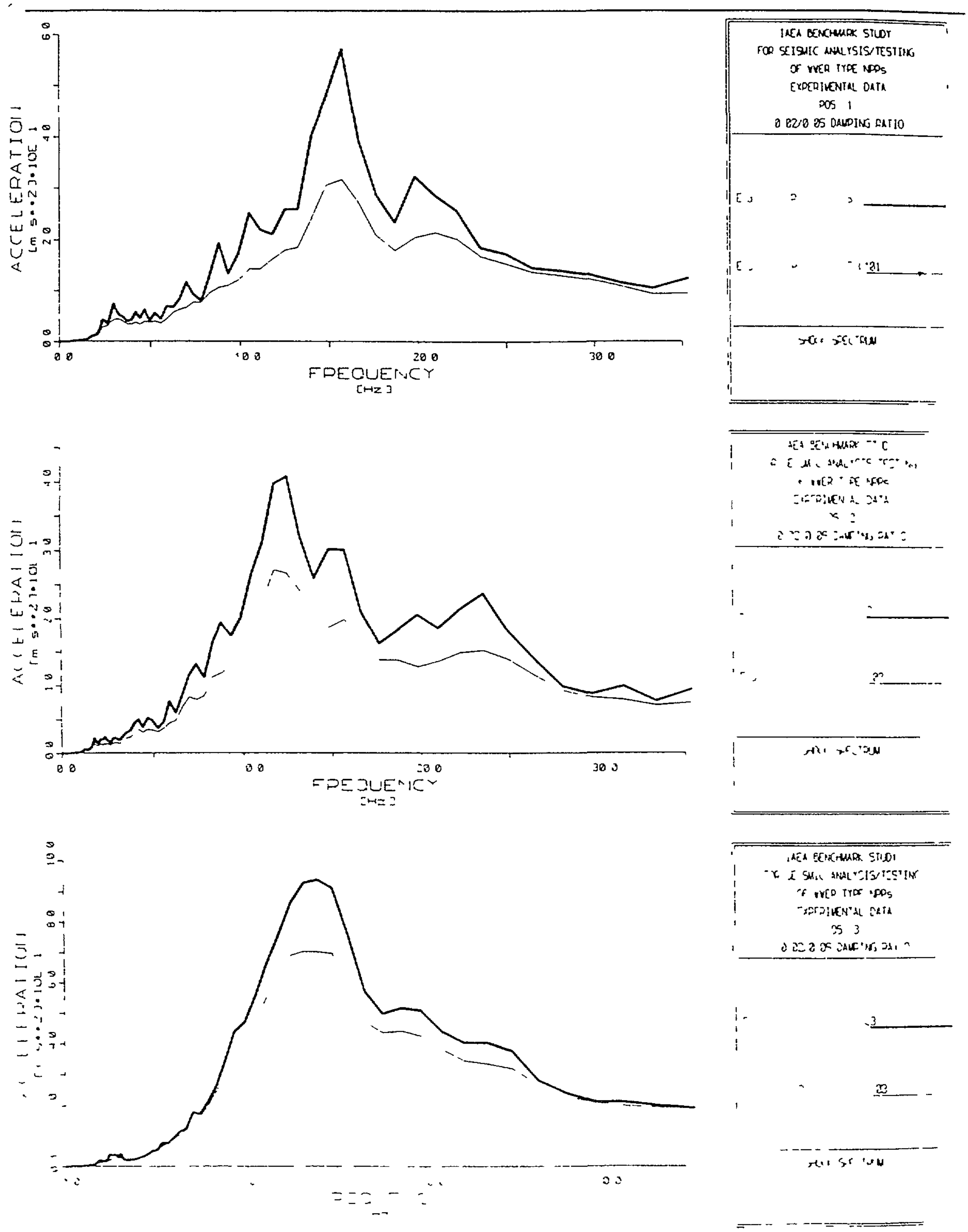

Fig. 9 Free-field Acceleration Response Spectra of the Explosive Excitation (Test 12/94)

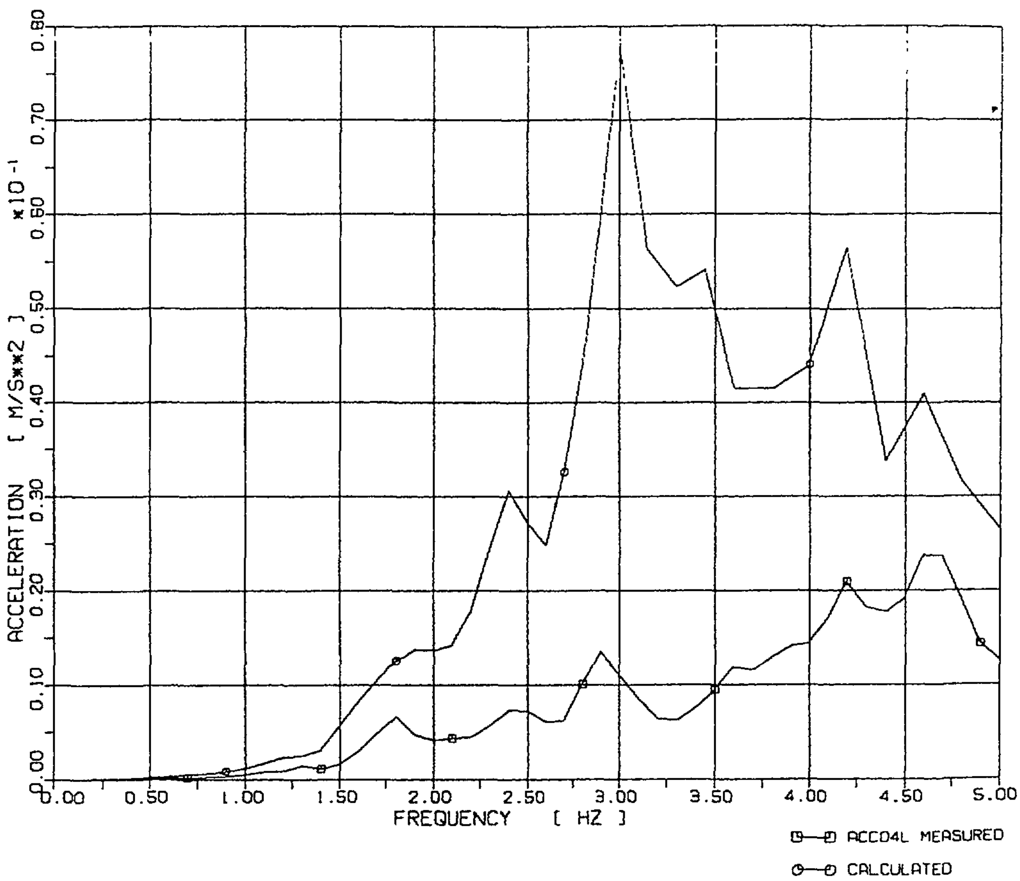

Fig. 10 Comparison of Calculated and Measured Response Spectra, Foundation Level X1

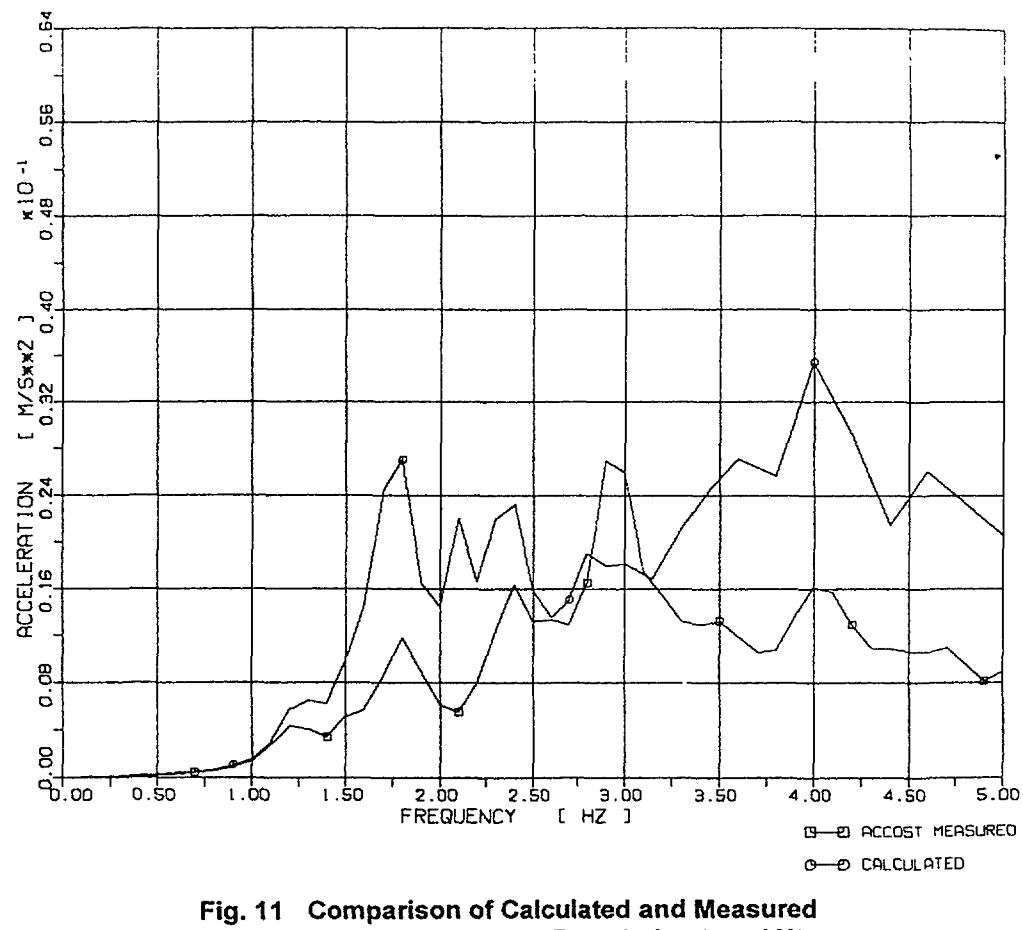

Fig. 11 Comparison of Calculated and Measured Response Spectra, Foundation Level X2

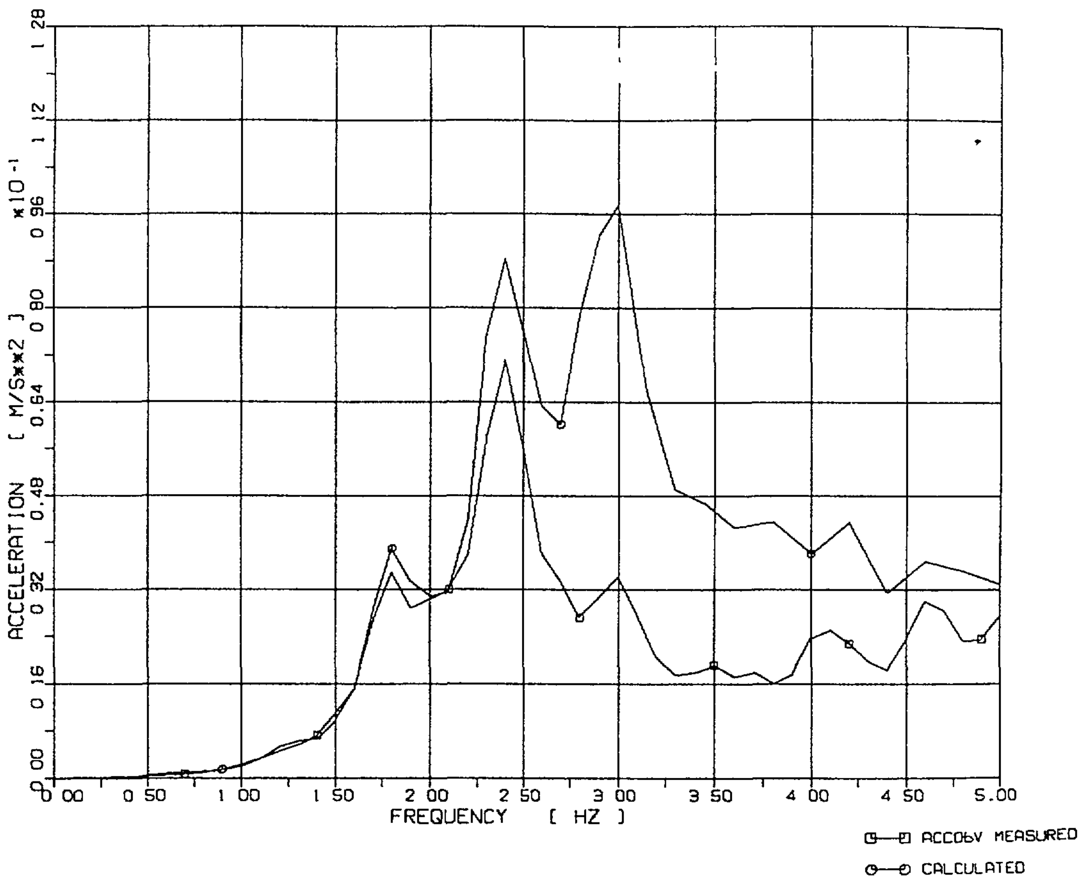

Fig. 12 Comparison of Calculated and Measured Response Spectra, Foundation Level X3

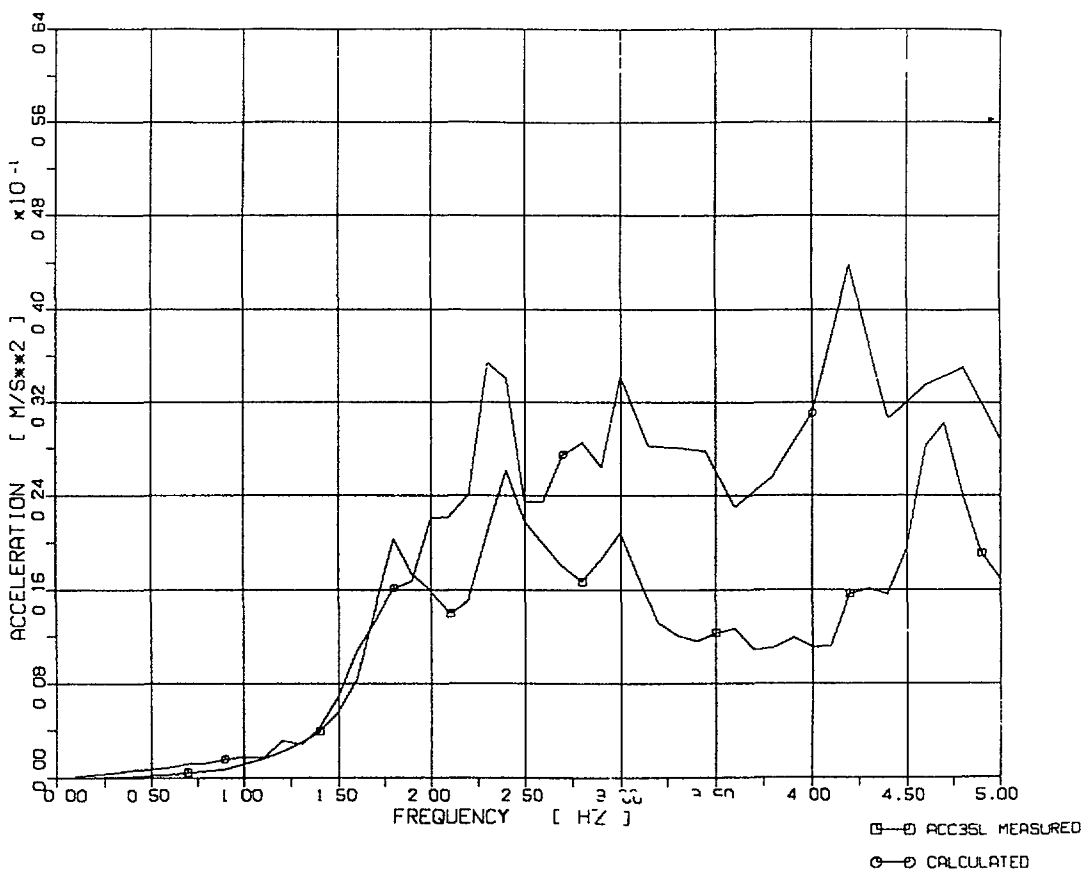

Fig. 13 Comparison of Calculated and Measured Response Spectra, Elevation 18.9 m X1

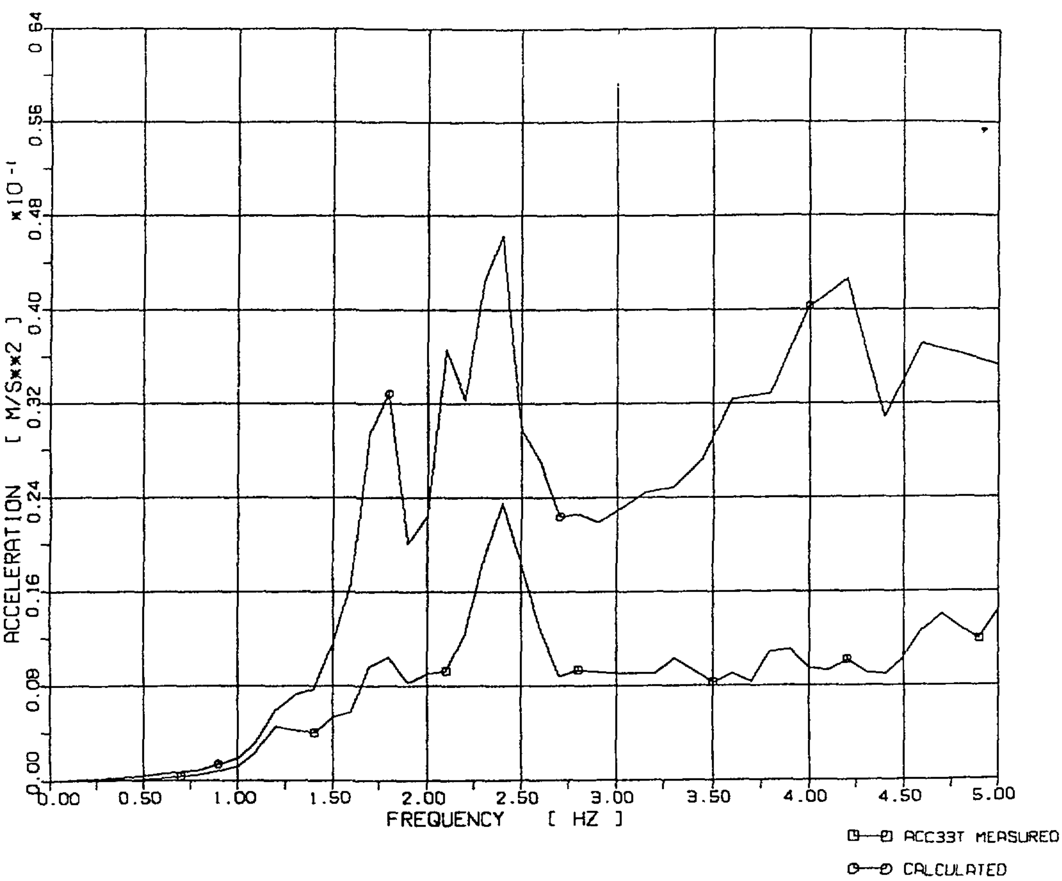

Fig. 14 Comparison of Calculated and Measured Response Spectra, Elevation 18.9 m X2

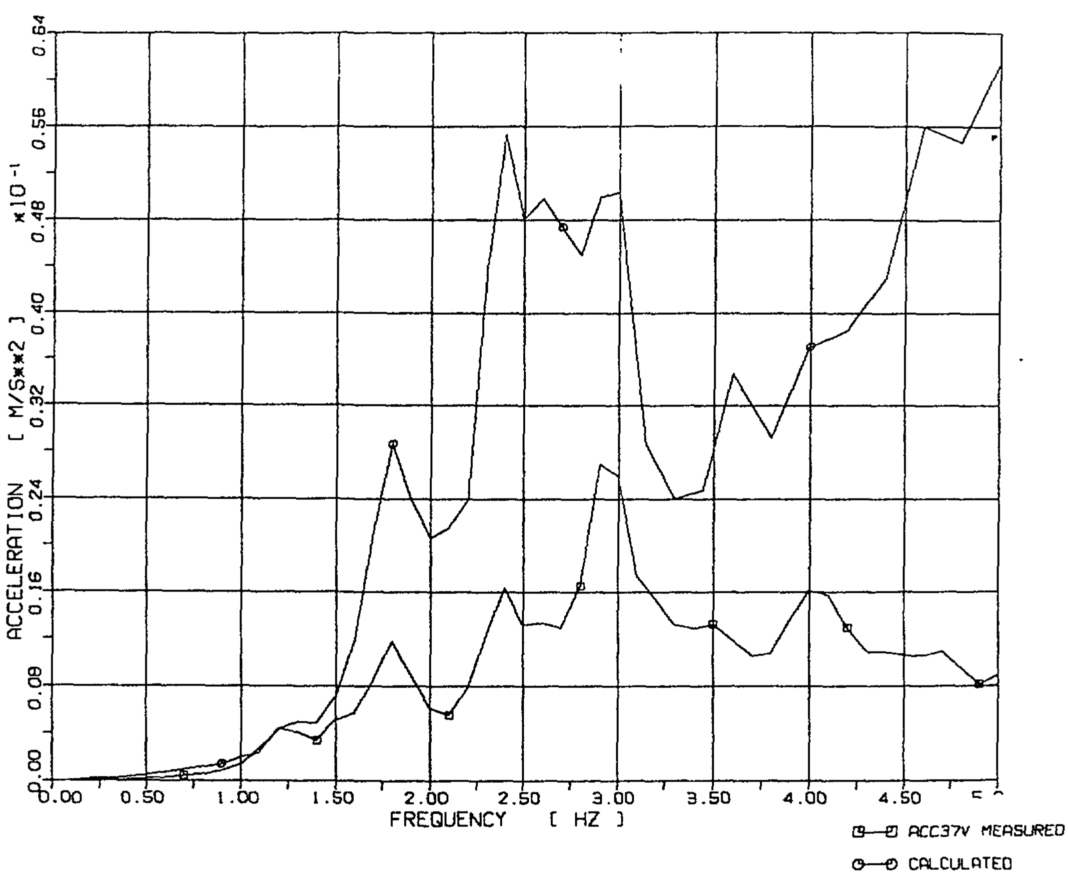

Fig. 15 Comparison of Calculated and Measured Response Spectra, Elevation 18.9 m X3

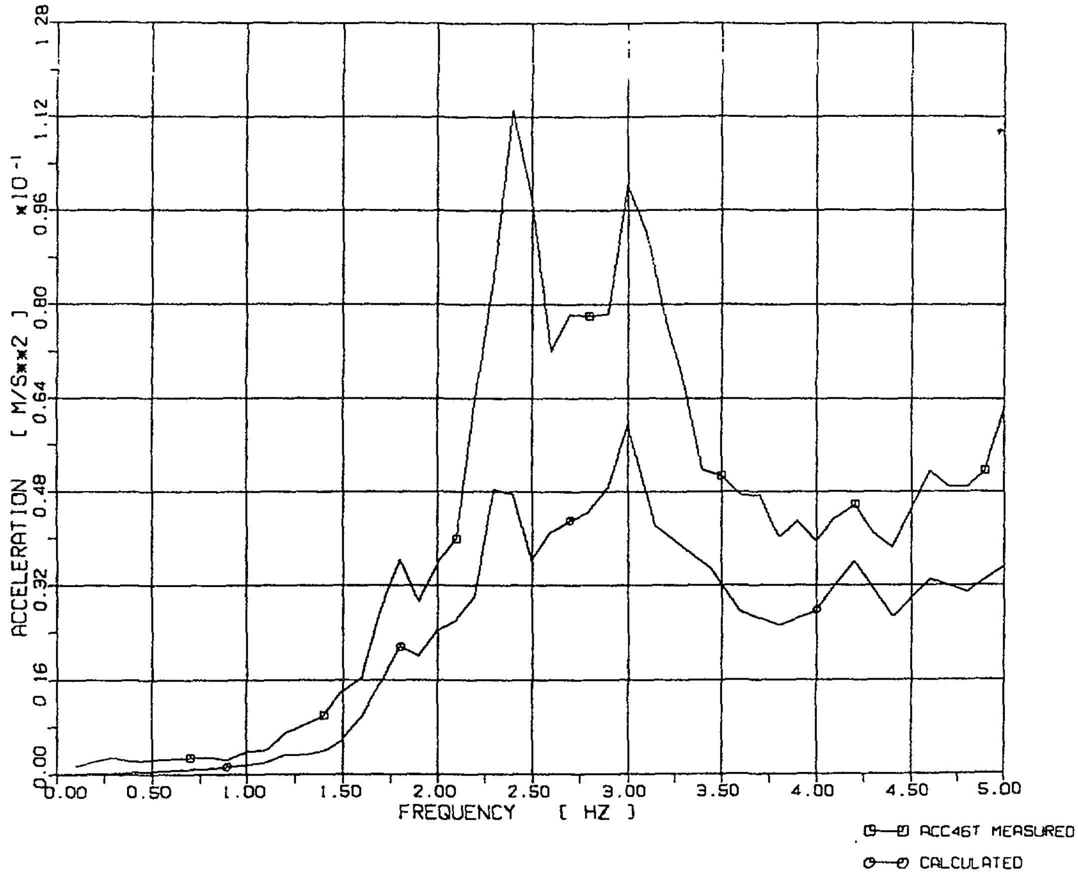

Fig. 16 Comparison of Calculated and Measured Response Spectra, Elevation of Crane Support X2

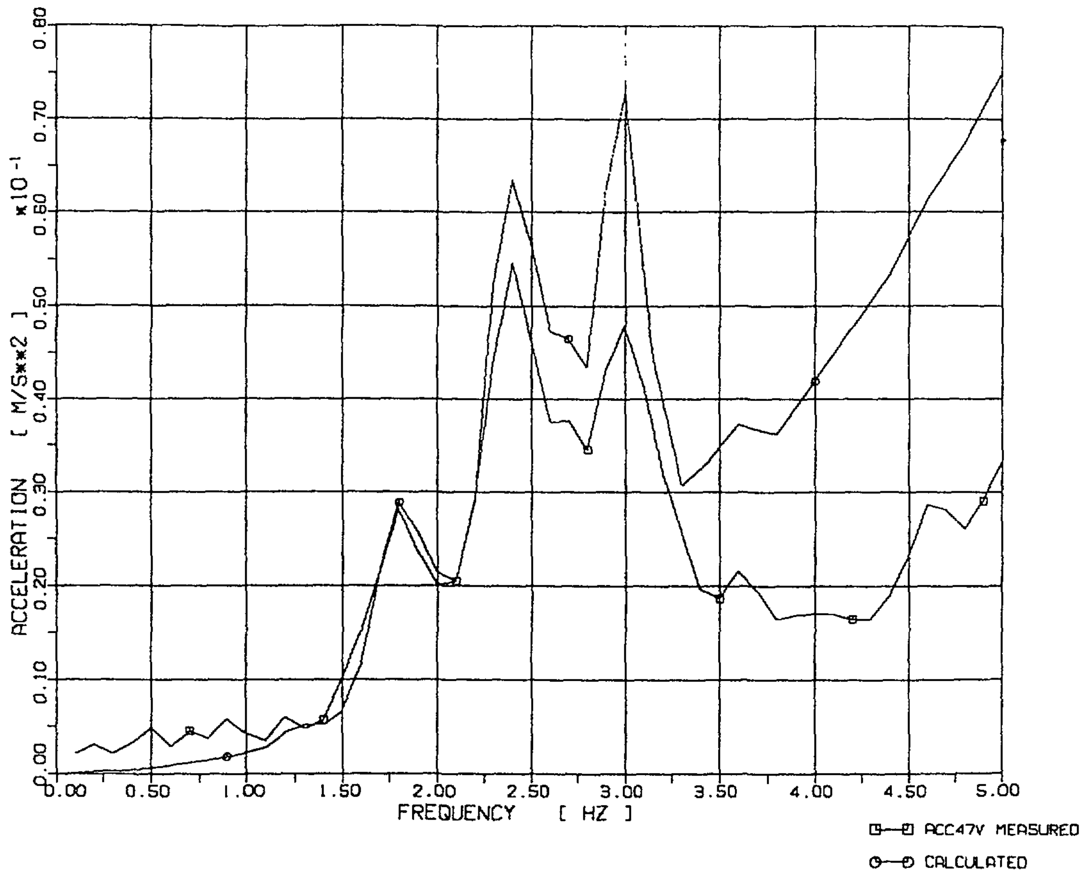

Fig. 17 Comparison of Calculated and Measured Response Spectra, Elevation of Crane Support X3

This applies especially to the horizontal directions of the foundation Positions A (Figures 10 and 11).

However, it can be observed that the analytically obtained spectra result in higher spectral acceleration than the measured results.

This may be explained by the rather low modal damping used in the calculation (8 % for horizontal and 10 % for the vertical direction). The real damping capacity seems to be somewhat higher.

The other explanation could be that (due to the capabilities of the mathematical model) and the strong excitation in the frequency range of about 12 - 15 Hz the analytical results obtained for the frequency range of the concrete structure (about 2.1 Hz) are higher (amplification Ω/ω) than measured in practice.

However, the selected damping (of 8 and 10 %, respectively) seems to be too high regarding the steel structures and therefore for the spectra calculated for the steel structure of the reactor hall (Positions 46 and 47) Figures 16 and 17 are smaller than the measured results.

When evaluating the analytical and experimental results obtained by the earlier tests 1990/91, a much better comparison of results may be observed (Figures 18 and 19).

CONCLUSIONS

- The mathematical models of the VVER 440/213 for investigation of seismic loadings could be verified by experimental tests performed on the site Paks by two series of tests (1990/91 and 1994).

- The free-field excitation generated during the earlier (1990/91) tests were more adequate and resulted in excitation effects of the coupled structure in the frequency range of earthquake loading

- The free-field excitation derived during the later tests (12/1994) resulted in free-field excitation characterized by higher frequency content useful to qualify the models for short duration loads (i.e. pressure waves or impact loading)

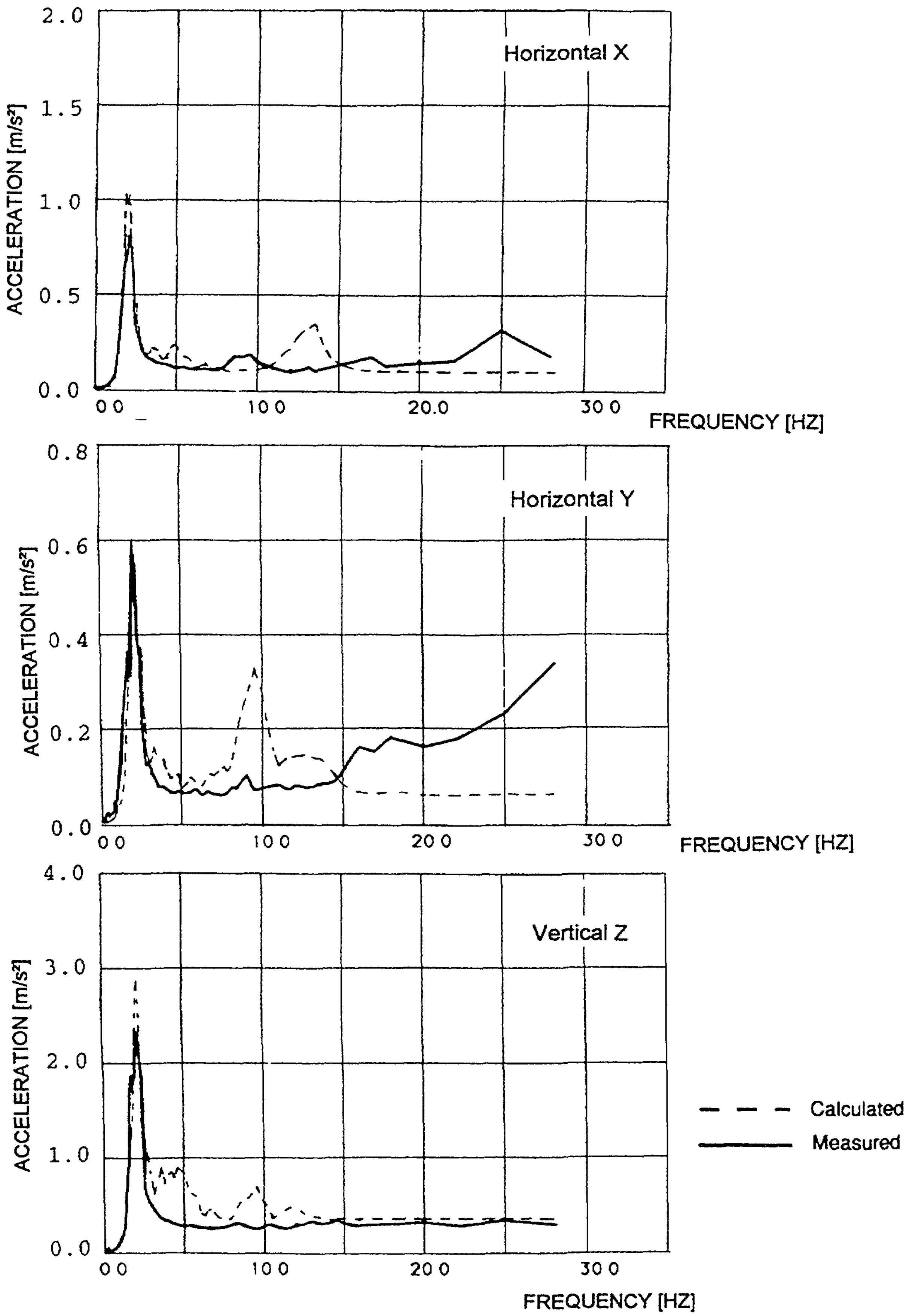

Fig. 18 Comparison of Floor Response Spectra (D = 2%) at the Elevation 18.9 m (Test I / S, 500 kg)

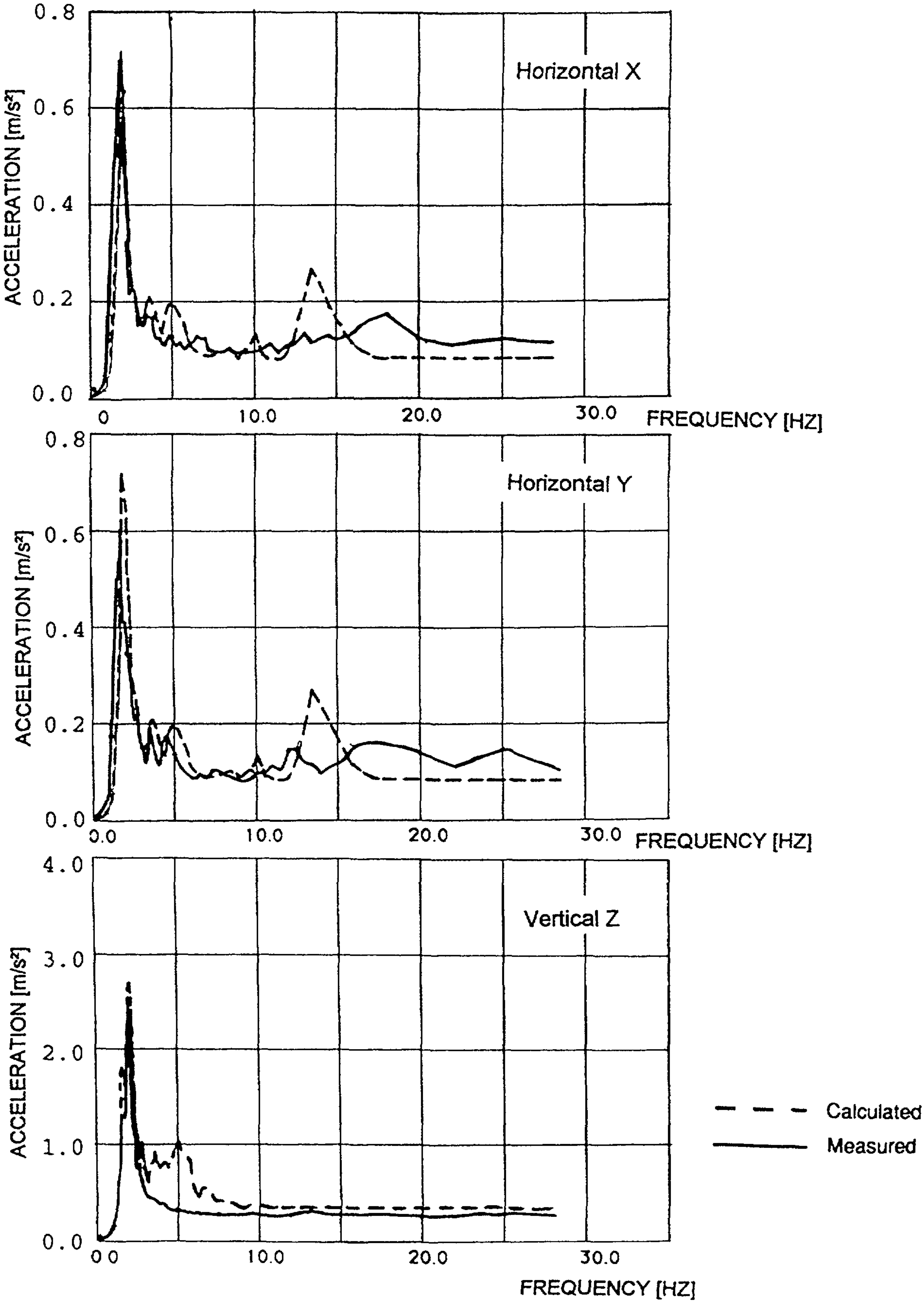

Fig. 19 Comparison of Response Spectra (D = 2%) at Foundation Level (Test I / S, 500 kg)

- It can be stated that the VVER 440/213 units of Paks are one of only few operating NPPs in the world of which the dynamic characteristics and real soil-structure effects were verified by natural scale models in similar extend.

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RESPONSE OF THE MAIN BUILDING, PAKS NPP (HUNGARY) TO EXPLOSION INPUT MOTION

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ABSTRACT

The purpose is to do dynamic analysis of the main building and to assess the response of the structure to the explosion ground motion and finally to compare the analytical results with the recorded motion at preliminary selected points in the structure. In this report are shortly discussed soil and structure modelling, analytical results and the comparisons with the measured response in terms of acceleration response spectra at two locations of the reactor structure, i.e. at the base foundation and at the main service floor at elevation 18.15m.

INTRODUCTION

The main building of Paks NPP is one of the two selected prototypes for benchmark study. In general the benchmark study program involves seismic analysis and testing of VVER type Nuclear Power Plants. The full scale dynamic testing of the Paks NPP is one of the most significant parts of the study. The test was performed in December 1994 and consisted in two main blasts with time delay so that a motion of about 20s to be recorded at different locations. One set of free field records was given up to the participant institutions for the benchmarking.

DYNAMIC EXCITATION

The input ground motion is presented by three acceleration time histories (three components) recorded at free field of the NPP site during the second explosion of the blast test. They are shown in Fig.1. The vertical component is very strong. The corresponding acceleration response spectra are given in Fig. 2. The analysis of the response of the main building is carried out in two variants: case 1 - the free field acceleration time histories are used as input motion acting at the foundation level; case 2 - the free field motion is transferred by deconvolution procedure to the foundation level taking into consideration the local ground conditions.

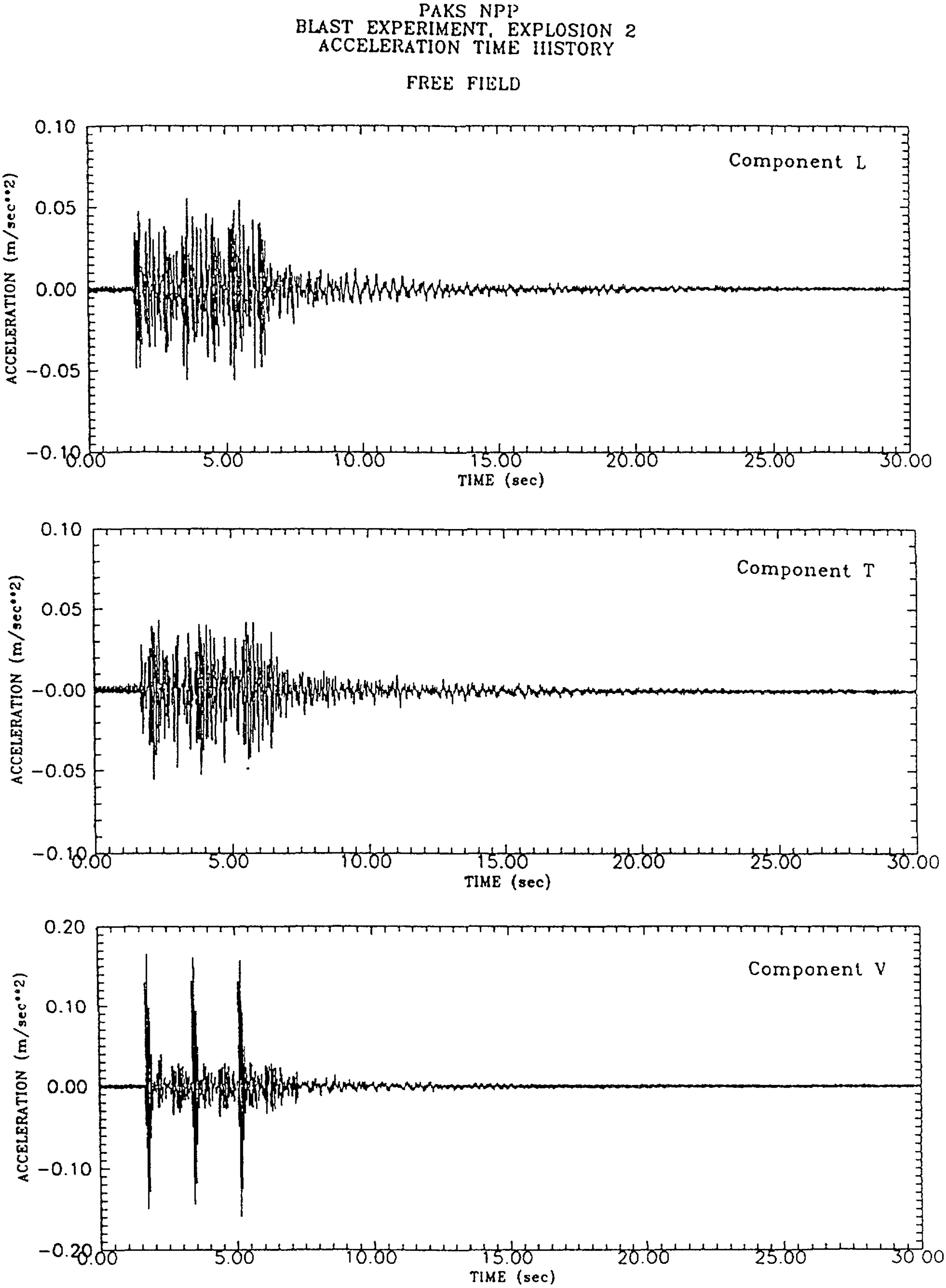


Fig.1. Time histories of the input excitation, free field

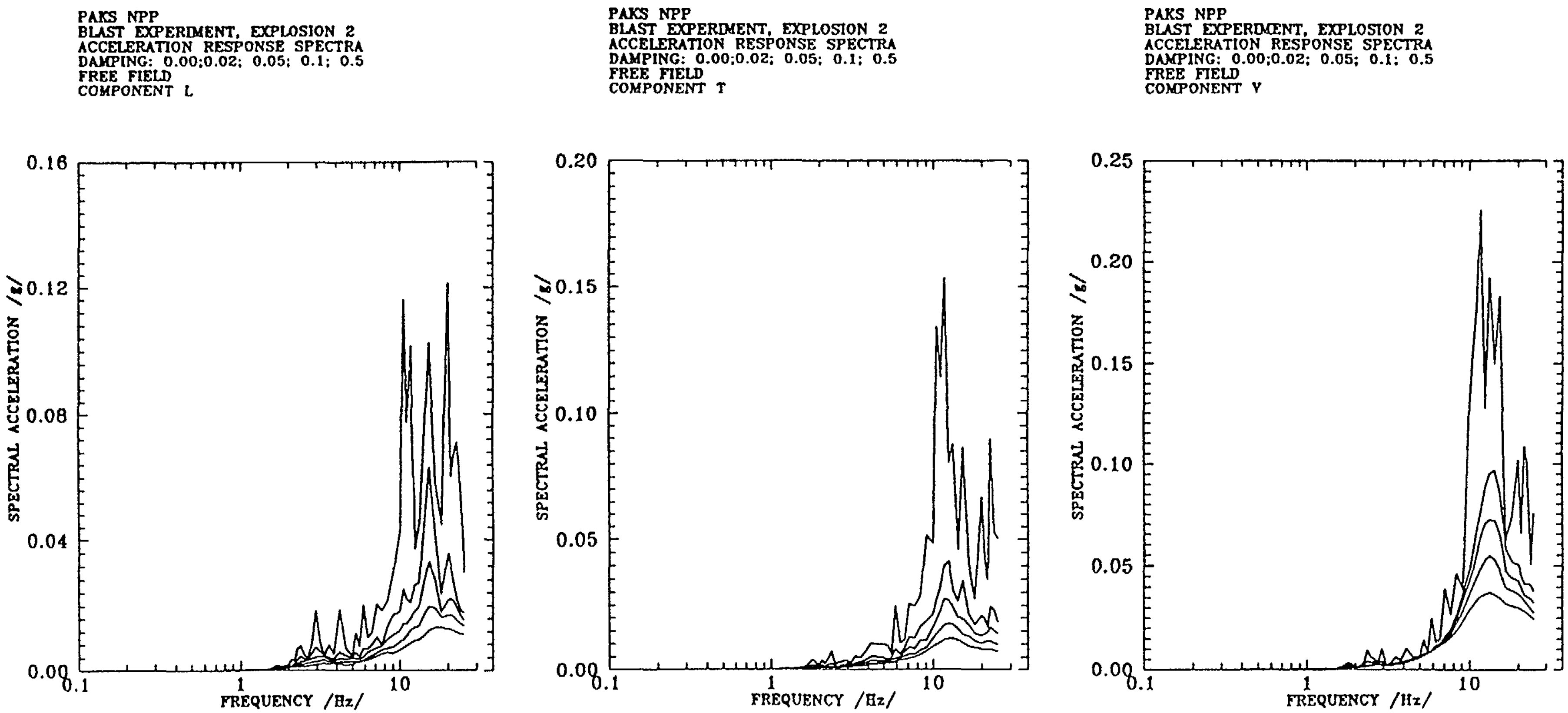


Fig.2. Acceleration response spectra of the input excitation, free field

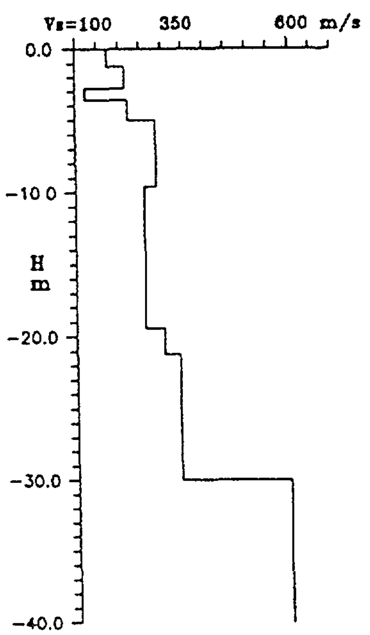
GEOLOGICAL CONDITIONS AT THE SITE

Four soil profiles are given in „Preliminary Input Data. Site Seismicity and Soil Mechanics at Paks NPP Site" (December 1993) with different characteristics of the soil layers. In this investigation the second profile is accepted. The respective soil layer characteristics are shown in Table 1. The velocity profiles of S-wave and P-wave are given in Fig.3 . Taking into consideration those characteristics the free field acceleration time histories are transferred to the foundation level. In Fig.4 are shown the acceleration response spectra of the free field motion, deconvoluted motion and

Table 1

Soil Profile

Layer thickness m	From level... to level... m - m	Density g/cm^3	S-wave velocity m/s	P-wave velocity m/s
1.2	0.0 - 1.2	1.95	174	301
1.6	1.2 - 2.8	1.95	215	380
0.8	2.8 - 3.6	1.95	124	215
1.4	3.6 - 5.0	1.95	222	384
4.6	5.0 - 9.6	1.95	288	500
9.8	9.6 -19.4	1.95	260	450
1.8	19.4 -21.2	1.95	306	525
8.8	21.2 -30.0	1.95	344	596
	> 30.0	2.10	600	1040



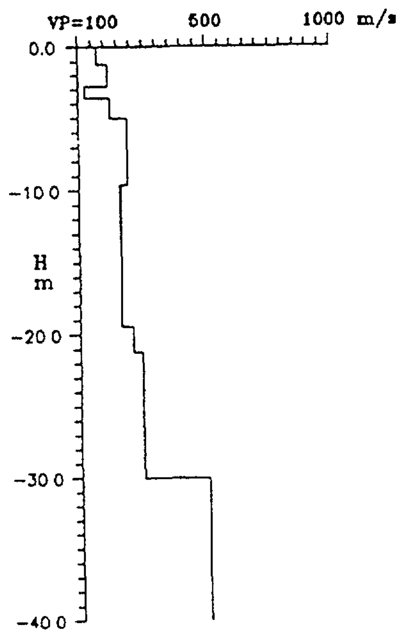


Fig.3. Velocity profiles of local geology

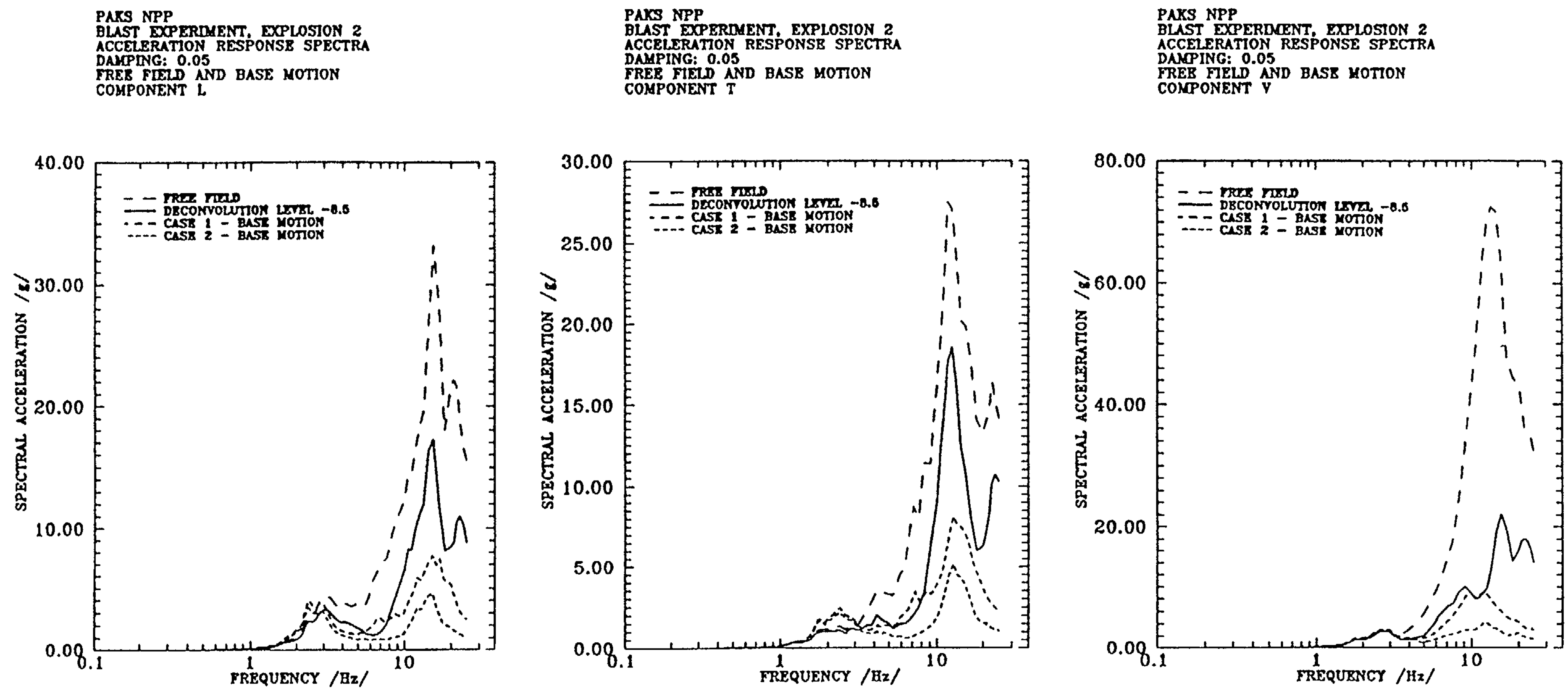


Fig.4. Comparison between free filed response and the foundation response

the respective response spectra at the foundation plate. One can see the amplification effect of the surface soil layers.

MODEL OF THE STRUCTURE

One of the twin units of Paks NPP is modeled and analyzed. 3-D model by finite elements is elaborated. The main building is founded on a monolith foundation slab with 2 m thickness at elevation of -8.5 m. On this basement block is located the condensing tower which rises to an elevation of 50 m. The turbine hall and the gallery buildings are not included in the present model. The material damping used is 4% of the critical one. Fig.5 presents general views of the 3-D model. The computation

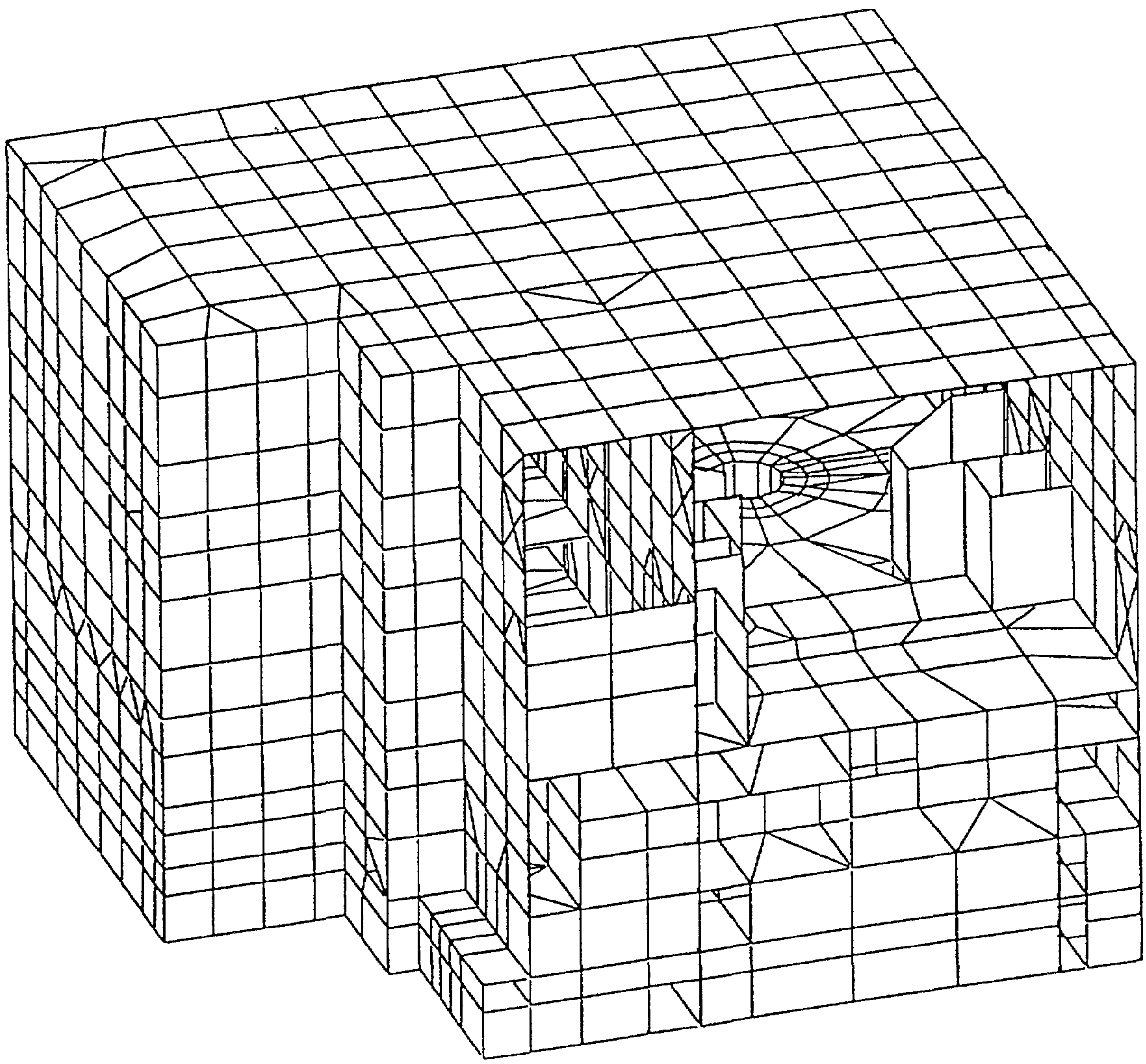


Fig.5. General view 3D FE model

model consists of 2195 nodal points, 920 beam elements, 3069 plate elements (2721 trapezoidal and 348 triangular elements) and 8080 dynamic degrees of freedom.

ACCELERATION RESPONSE SPECTRA

The in-structure acceleration response spectra are generated for two cases depending on the input motion. The nodal points for which the spectra are compared are located at foundation level and at elevation 18.15 m. The components 14 and 15 (horizontal) and 16 (vertical) are located at the base mat, the components 34 and 35 (horizontal), and 36 (vertical) are located at elevation 18.15m. The analysis is performed by the computed code SASSI. The comparisons are presented in Figures 6 and 7. The spectra presented are computed for 5% critical damping. A detail of the spectra for the frequencies up to 10 Hz is presented in Fig.8.

DISCUSSION AND CONCLUSIONS

On the base of the analysis carried out the following conclusion can be done:

- The surface soil layers have considerable influence on the response of the soil-structure system. The amplifying effect is very high. The difference of the soil characteristics in the upper 10 m is not very big but the effect is remarkable due to the peculiarities of the input motion.
- The response of the structure is very slight. The soil-structure interaction is clearly demonstrated - the fundamental modes of vibrations are dominated by the response of the soil. This is a typical example of a dynamic response of rigid structure founded in deformable soil.
- The input motion used in the analysis is high frequency motion due to the blast excitation. Such motion with small amplitudes can not provoke a real response to seismic excitation. The damping in the soil and in the structure is very small. On the opposite, during a real earthquake the damping increases and reduces the response of the soil-structure system. The behavior of the system could be completely different.
- The blast test gives the possibility to assess the fundamental period of vibrations of the soil-structure system.

BENCHMARK STUDY
FOR THE SEISMIC ANALYSIS AND TESTING
OF WWER TYPE NPPs

DYNAMIC ANALYSIS OF PAKS NPP STRUCTURES:
REACTOR BUILDING

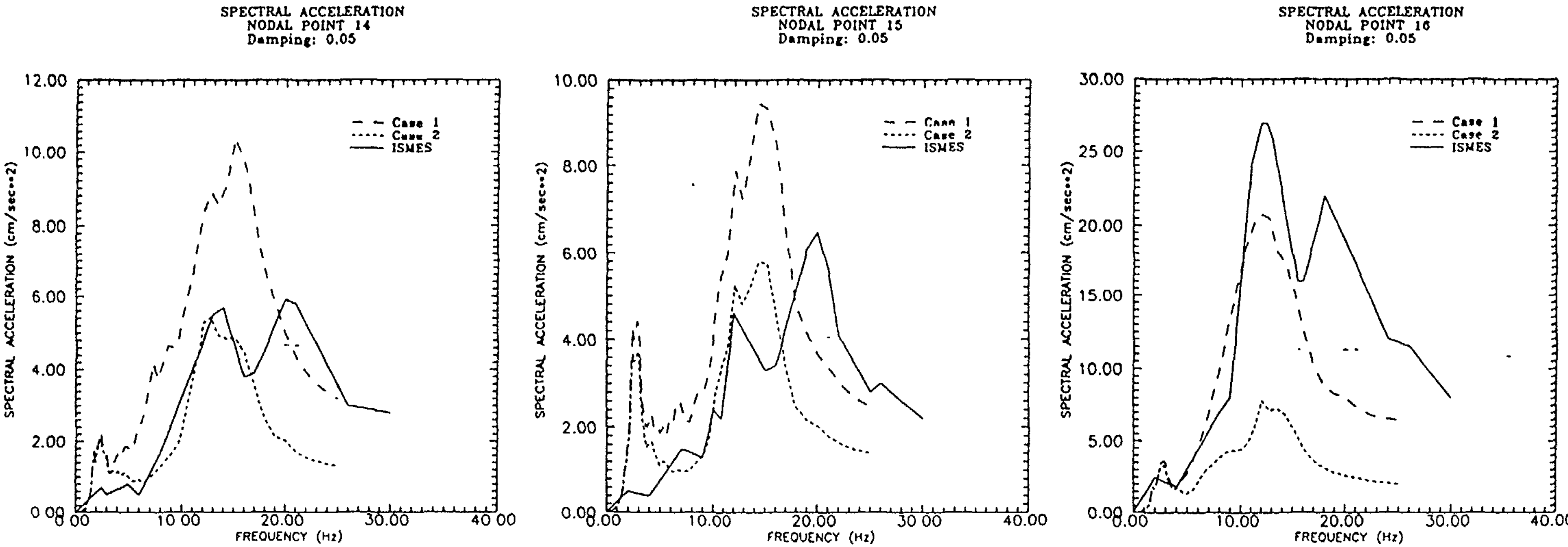


Fig.6. Comparison between analytical and measured response at foundation level

BENCHMARK STUDY
FOR THE SEISMIC ANALYSIS AND TESTING
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DYNAMIC ANALYSIS OF PAKS NPP STRUCTURES:
REACTOR BUILDING

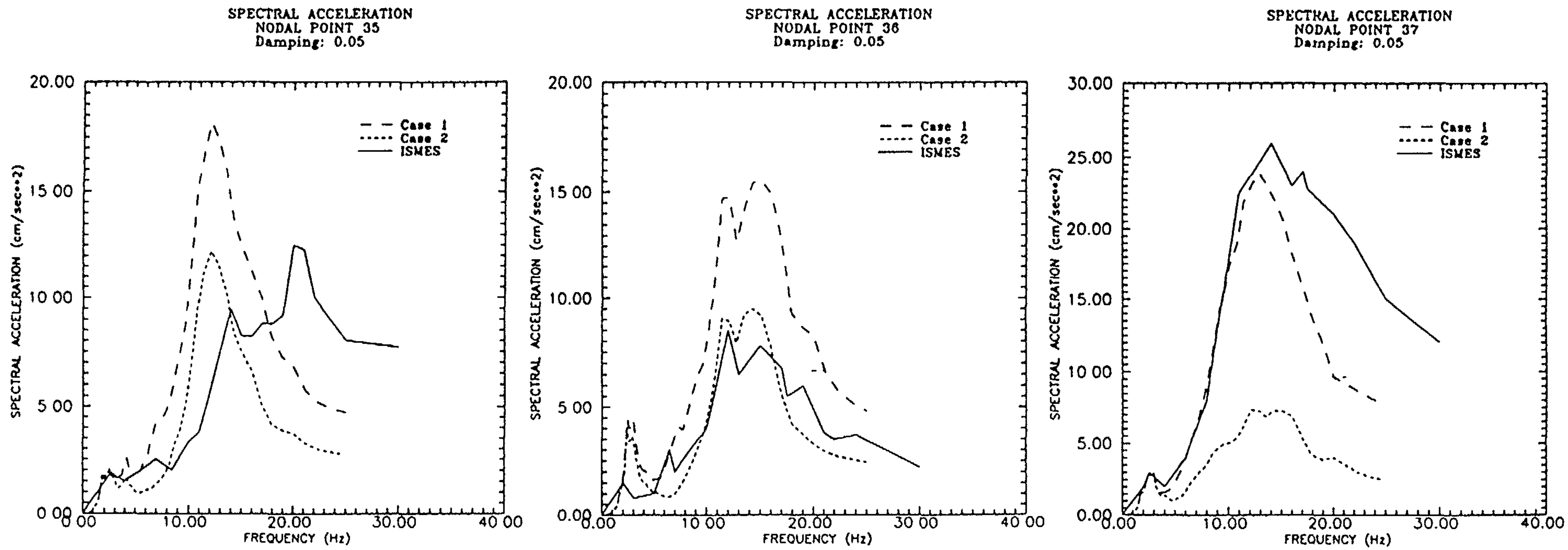
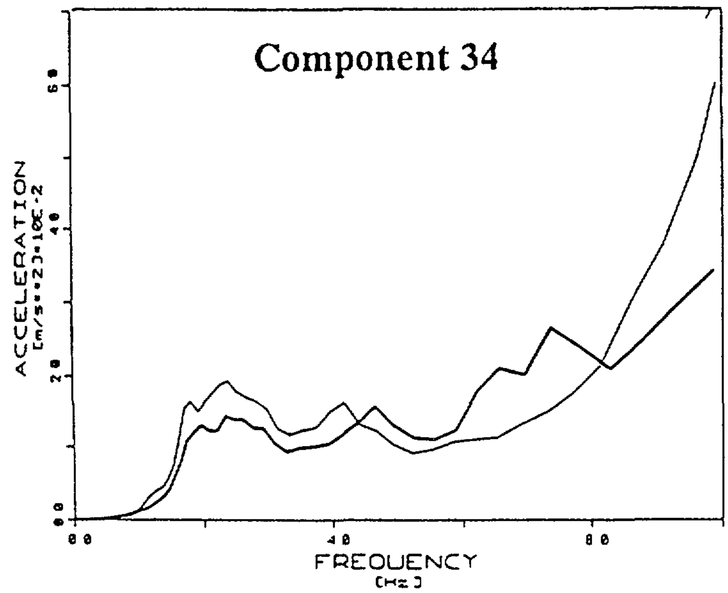


Fig.7. Comparison between analytical and measured response at elevation 18.15



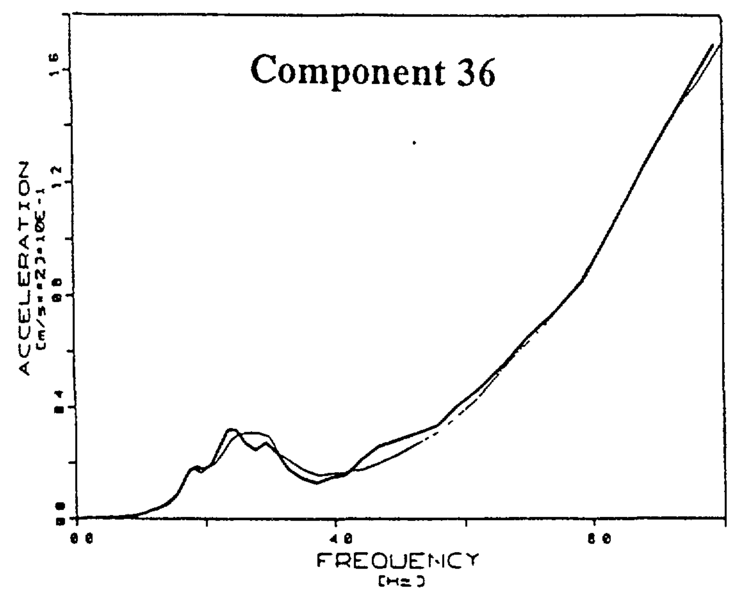


Fig.8. Comparison between analytical and measured response at elevation 18.15, detail.

- The explosion input motion provokes a considerable vertical component at all nodal points and elevations - it is commensurable with the horizontal components, even grater in some points.

The comparison between the analytical results and the measured response leads to the following conclusions:

- There is relatively good prediction of the major features of the seismic response, i.e. the first natural frequency of the soil-structure system is about 2 Hz, the predominant frequency of the response is about 15Hz.
- The response prediction in case 1 (not deconvoluted input motion) matches relatively well the measured vertical response but the predicted horizontal response is greater than the measured one.
- The response prediction of case 2 (deconvoluted input motion) matches relatively good the horizontal response but the vertical response is smaller than the measured one.
- Generally in most of the cases the analytically predicted response in the frequency range above 18-20Hz is underestimated.

Possible explanation of the differences between measured and predicted response:

1. It is well known that the analytical procedure of deconvolution is limited to relatively simple wave environment - vertically propagating waves in horizontally stratified medium. In the benchmark case the vertical motion is determined predominantly by P wave. For the deconvolution of the P waves different material damping should be used than in the case of S wave deconvolution. The underground water level also plays an important role. The investigated structure is founded probably bellow or very near to the water table so that the P waves induced by the blast excite directly the foundation.

2. The underestimated response in the high frequency range (above 18Hz) could be caused by:
- overestimated material damping;
- overestimated radiation damping;
- selection of the seismic environment (wave field);
- definition of the transmitting boundaries.

or the measured acceleration response (obtained by differentiation of the velocity response) is erroneous.

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SUMMARY OF STRUCTURAL ANALYSIS AND COMPARISON WITH EXPERIMENTAL RESULTS FOR PAKS NPP

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M. DAVID
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Abstract

This contribution deals with the analysis and comparison of the dynamic response, calculated and measured by the explosion test in Nuclear Power Plant Paks, Hungary. Some details of the calculation model are also presented. The calculated and measured data of dynamic response are compared in selected points of the NPP Paks reactor building. Conclusions and recommendations are derived from this comparison.

1. INTRODUCTION

This contribution describes the calculation of dynamic response of the NPP Paks reactor building to the full scale blast testing and the comparison of calculated and measured data. This work has been carried out within the scope of IAEA co-ordinated research - Benchmark study for seismic analysis/testing of NPPs type WWER [4].

The dynamic input for the dynamic analysis was presented by ISMES according to the measurements during the blast test, but the results of dynamic response measurements were not available for the calculation. The comparison of calculated and measured floor response has been carried out by ISMES, Bergamo [4] and then handed over to all those who participated in for further analysis.

2. DYNAMIC INPUT

Dynamic input was presented by ISMES in the form of time histories of velocities and relevant accelerations, recorded during the blast test in the position FF (free field) on the soil [2]. Time history of accelerations was used in the calculations taking into account that the accelerations were obtained through simple derivation of recorded velocities.

3. DYNAMIC ANALYSIS

Calculation Model

The FEM calculation model corresponds to the data of papers [3] and includes all structures of the main building even though the dynamic response is required in the reactor building only (ref. Fig. 1). All structural parts of the main building are connected together and they behave as a unit. The calculation model of the coupled vibrating building structures was created by means of beam elements (476), trusses(411), plate elements (1087), spring and mass elements (252). The total number of DOF is 6450.

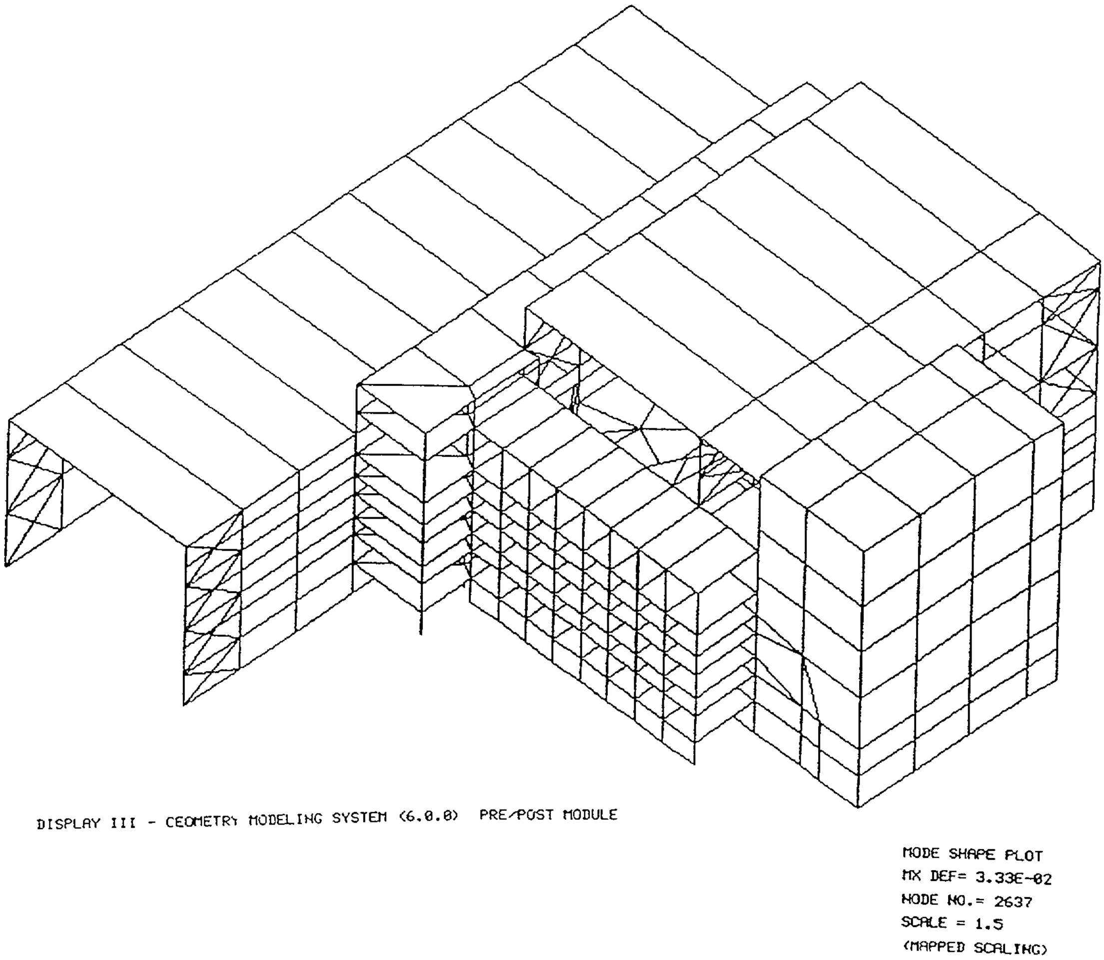

Figure 1. FEM model of reactor building and turbine building.

Soil - Structure interaction

The soil characteristic data have been assumed according to the NPP Paks Cross-Hole tests in the soil-structure analysis. In order to take into account the embedment level of the building, the time histories of accelerations were deconvoluted to the foundation level at - 7,00 m with the help of the software SHAKE [8].

The simplified soil-structure interaction has been carried out, using a system of springs to represent the soil. The spring constants have been determined for different frequencies, however the spring constants corresponding to the most important modes of vibrations on the soil have been used in the calculation as frequency independent.

The assumed soil properties are presented in Table 1.

Tab. 1: Soil profile

No.:	Level [m]	Thickness of layer [m]	Material	Density [t/m³]	ν	G [MPa]	v_s [m/s]
1	-20.	20	fine sand	1,96	0,28 - 0,48	122,50	250
2	-27	7	medium sand	1,96	0,48	313,60	400
3	-110	83	large grain sand	2,16	0,48	653,40	550
4	-510	400	gravely sand	2,16	0,48	1058,00	700

The spring constants as well as damping for the vibration modes on the soil can be found in Table 2.

Tab. 2: Stiffness and damping of the soil

Direction	Stiffness [MP/m]	Damping [%]
x	3,6	15
y	3,6	15
z	10,0	30

Dynamic response

Dynamic response to the blast loading has been calculated in the form of floor response spectra in selected nodes of the calculation model (ref. Figs. 2-6). These points correspond to the measured points of the explosion test of NPP Paks The notation of nodes in the model and number of positions indicated in ISMES report [1] are different. The relationship between model and ISMES report notation can be found in Table 3.

The method of modal analysis executed in time steps has been used for the calculation of dynamic response. The response of time histories of acceleration in selected nodes were evaluated and from these time histories the floor acceleration response spectra have been determined Modal damping for the global modes of vibration on the soil were introduced in the calculation as indicated in the Table 2, for all structural modes of vibrations the damping was assumed by a factor of 5% of critical damping The Softwaresystems NISA II [5], [6] and STARDYNE [7] have been used for all calculations

4. COMPARISON OF MEASURED AND CALCULATED DATA

The analysis of calculations and measurement is based on the comparison of calculated and measured floor response spectra of acceleration, determined of selected points of the structure The

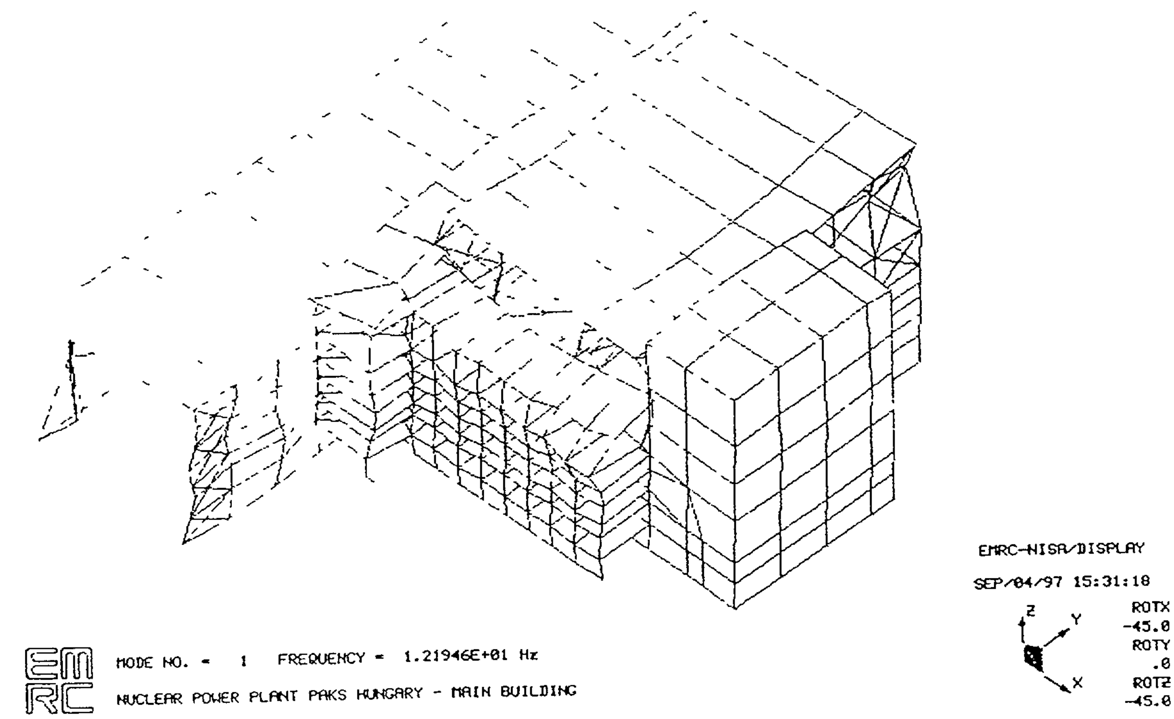

Figure 2. Deformed shape - example of mode n1.

INSTRUMENTATION LAYOUT

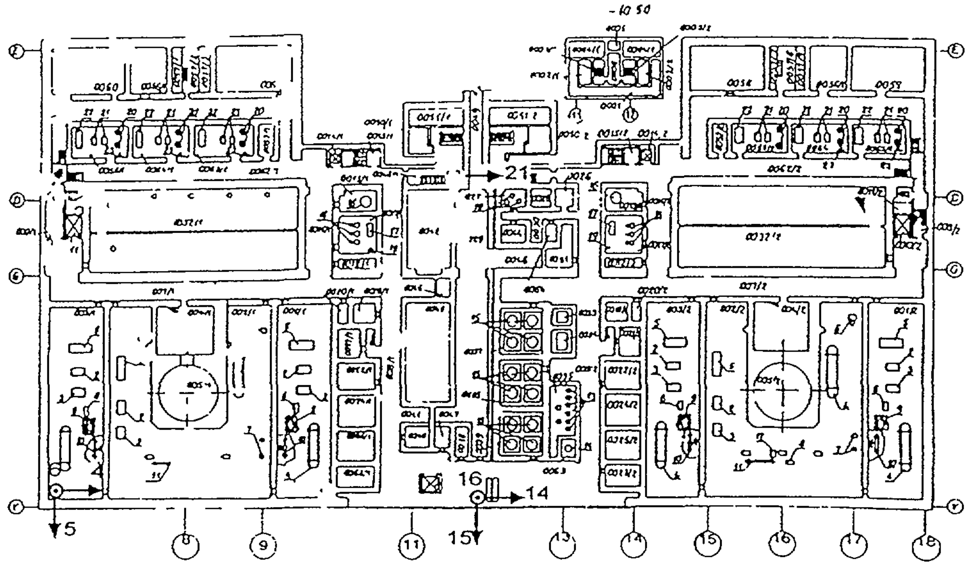

Figure 3. Instrument positions for comparison with analysis.

INSTRUMENTATION LAYOUT

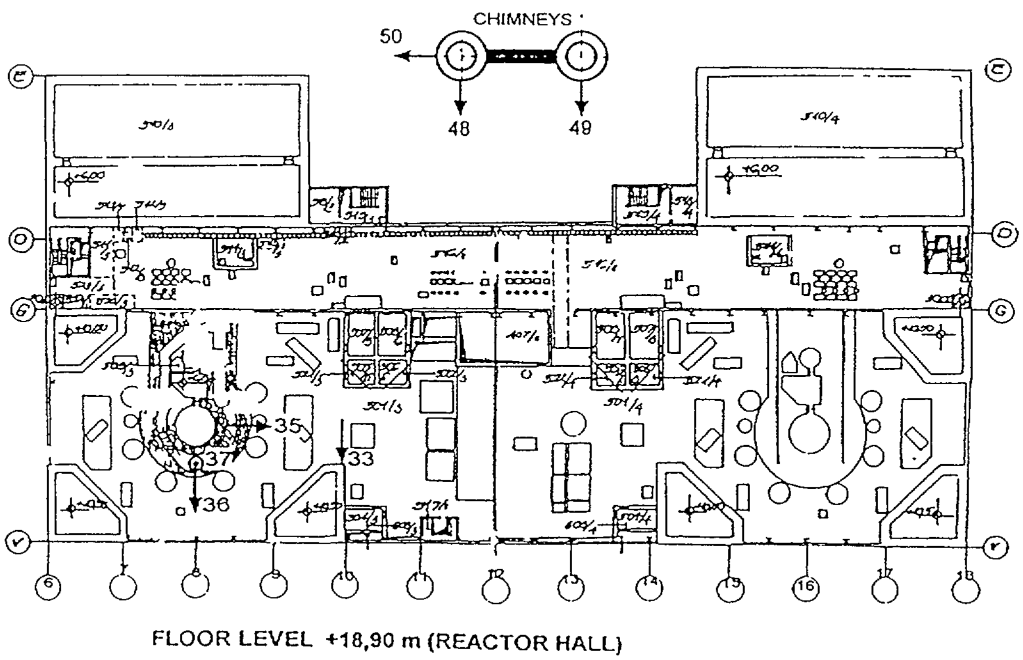

Figure 4 Instrument position for comparison with analysis.

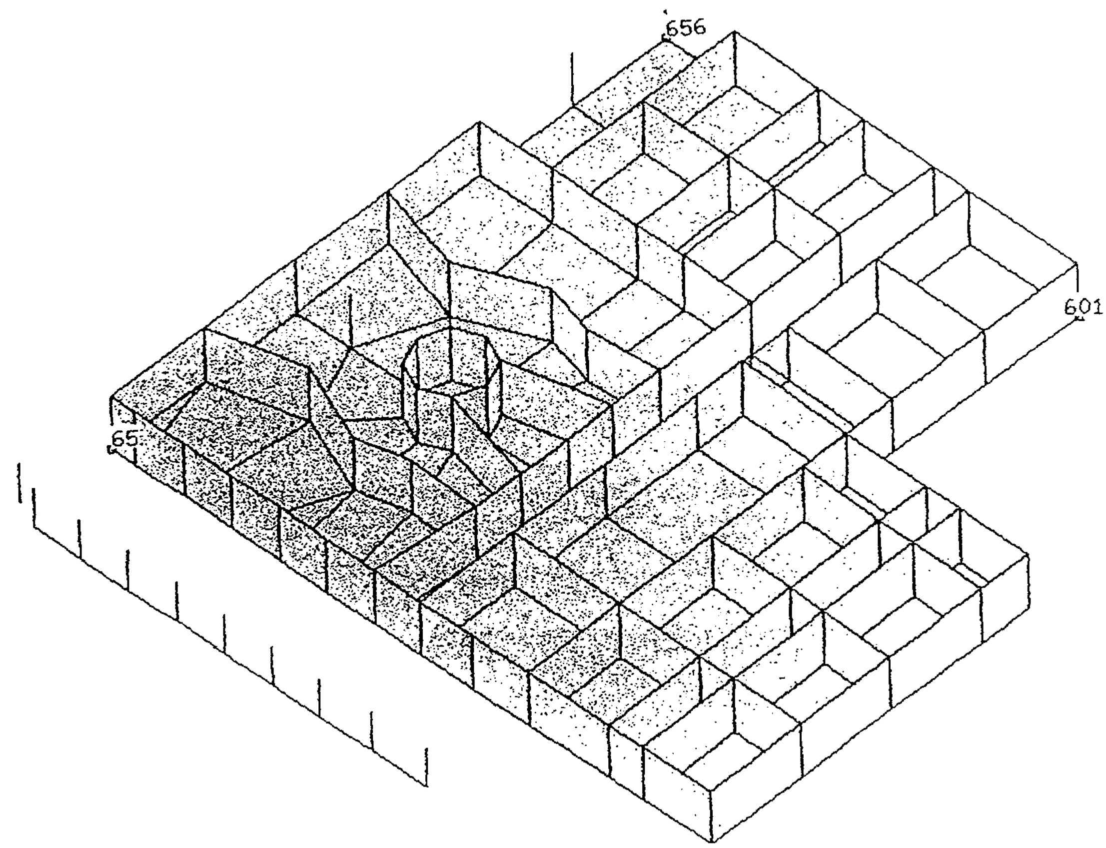

Figure 5. Selected nodes for response evaluation.

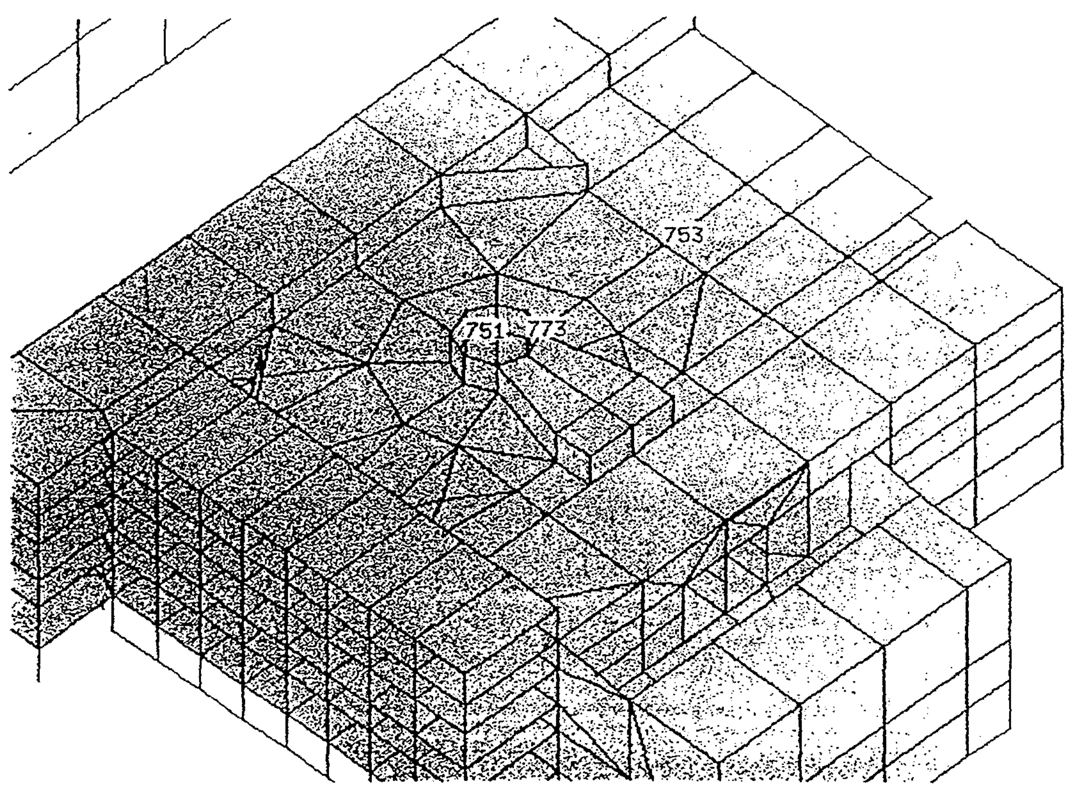

Figure 6. Selected nodes for response evaluation.

calculated and measured floor response spectra have been plotted together by ISMES in corresponding points. The points for the comparison of floor response spectra have been selected on the level - 6,50 m (foundation slab), + 18,90 (reactor hall) and on the crane supporting beam in the reactor house. The notation of the selected points according to the ISMES notation and according to the authors calculation model can be found in Tab. 3. Very brief characteristic about the comparison of measured and calculated data is also presented in this table.

Table 3 Comparison

ISMES No.:	ISMES Direction	MD No.:	MD Direction	Level [m]	Fig. No.:	Brief characteristic of comparison of measured / MD Data
1	L	1	Y	- 6,50		
2	T	2	X			
3	V	3	Z			
4	L	65	Y	- 6,50	7	good in frequencies, calculated data higher than measured
5	T	65	X	- 6,50	8	good in frequencies, calculated data higher than measured
6	V	65	Z	- 6,50	9	good in frequencies, calculated data higher than measured
14	L	656	Y	- 6,50	10	calculated peaks found at 3 -5 Hz higher frequencies than measured, calculated data higher than measured
15	T	656	X	- 6,50	11	calculated peaks found at 3 -5 Hz higher frequencies than measured, calculated data higher than measured
16	V	656	Z	- 6,50	12	calculated peaks found at 3 -5 Hz higher frequencies than measured, calculated data higher than measured
21	L	601	Y	- 6,50		calculated peaks found at 3 -5 Hz higher frequencies than measured, calculated data higher than measured
33	T	753	X	+18,90	13	relative good, discrepancies around 25 Hz
35	L	773	Y	+18,90	14	relative good, discrepancies around 25 Hz
36	T	751	X	+18,90	15	relative good, discrepancies around 25 Hz
37	V	751	Z	+18,90	16	good, main calculated peak is higher than measured, measured data higher than calculated around 20 Hz
46	T	2387	X	crane support.	17	measured data higher than calculated,
47	V	2387	Z	crane sup port beam	18	measured data higher than calculated, second peak was not found in the calcul.

The pure comparison of floor response spectra, calculated and measured, would lead to very different conclusions. Some of floor response spectra calculated and measured are in very good agreement, some discrepancies can be found in others. However, it is very difficult to define what is very good agreement. Full agreement cannot be expected between calculated and measured data, because there are a lot of uncertainties in the structure, in the soil - structure behaviour and in simplifications which have to be done for the evaluation of the calculation model. The authors of this contribution have concentrated their attention on the analysis of calculated and measured data particularly on the following items:

- the agreement of the common characteristic of the compared floor response spectra, (the number of peaks, the ratio of peaks calculated and measured etc.)
- the check of frequency of peaks
- the check of accelerations calculated and measured

The most important aspect is to find out the origin of discrepancies. The authors use the method of dynamic identification for this task. By this method some parameters of the calculation, for example damping, spring constants, Youngs modulus of concrete etc. will be changed and the calculation will be repeated. The right parameters are identified, when good agreement of calculated and measured data is obtained.

The above mentioned procedure has not yet been completed for the analysis of NPP Paks explosion test. The analysis of the measured and calculated results will be now presented on the basis of some preliminary investigations. All points will be denoted according to the ISMES notation. For the comparison of floor response spectra calculated and measured the reprints from ISMES report have been used. The thick line in the figures always refers to the measurement and the comparison is presented for the damping of 2% and 5 % of critical damping.

Foundation slab - 6,50 m

Pos. 4, 5, 6, (directions L, T, V) are located on the foundation slab in the corner (rows V, 6). Very good agreement in peaks frequencies can be found. On the other hand, the calculated response of acceleration is higher than the measured one (ref. Figs.7, 8, 9). The presented comparison has indicated relative good soil representation in the calculation but the real damping of the soil seems to be higher than the one assumed in the analysis, or the decovolution of the time history from the level $\pm$0,00 to the foundation level, calculated by SHAKE, was not effective enough in the reduction of acceleration.

Pos. 14, 15, 16, (directions L, T, V), are located in the opposite corner (rows V, 12), close to the thermal expansion gap, in between two units of NPP Paks. The agreement of peaks frequencies is not as good as before, the calculated frequencies are about 2 - 5 Hz higher than the measured ones. The cited discrepancies have most probably been caused by the neglecting of the second unit in the calculation (ref. Figs. 10, 11, 12).

Reactor hall - Level +18,90 m

Pos. 33. (direction T) is located in the row 10. A relative good agreement has been found in the characteristic of calculated and measured spectra. Some discrepancies have been found in the region of 20 Hz. The local peak at cca 6 Hz has not been indicated in the calculation (ref. Fig. 13).

Pos. 35, 36, 37 (directions L, T, V) are located on the concrete reactor shaft. This part of the reactor building is very important, because the main technological equipment is located there. Relative good agreement can be found in main peaks frequencies with the exception of the region of 20-30 Hz, where the calculated peaks can be met on higher frequencies. Very good agreement can be demonstrated in the part 0 - cca 12 Hz, particularly for the pos. 37, but the local small peaks around 7 Hz have not been found in the calculation (ref. Figs. 14, 15, 16).

Pos. 46, 47 (direction T, V), are located on the crane beam. The calculated response in V direction is lower than the measured one. The calculation has not indicated one important peak, measured at cca 21 Hz. These discrepancies have most probably been caused by the calculation model, because the representation of the crane beam and its connection to other structures were not elaborated in sufficient detail (ref. Figs. 17, 18).

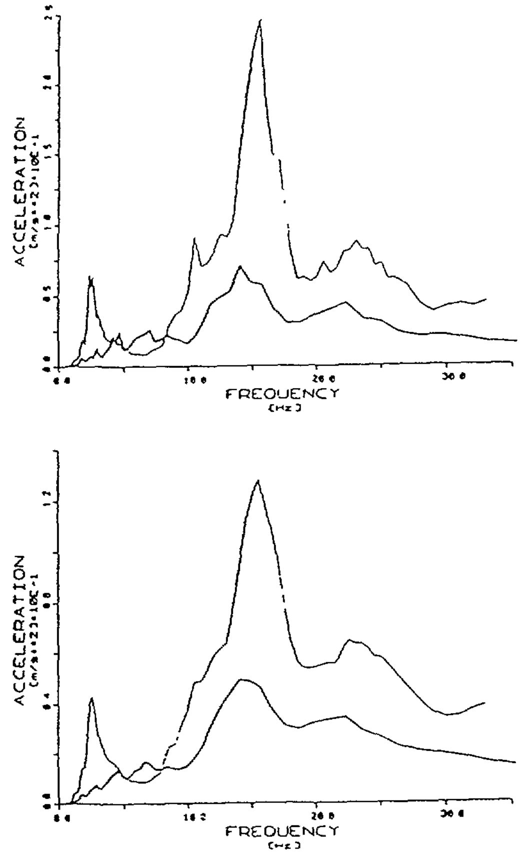

Figure 7. Results at node 65 (Y)

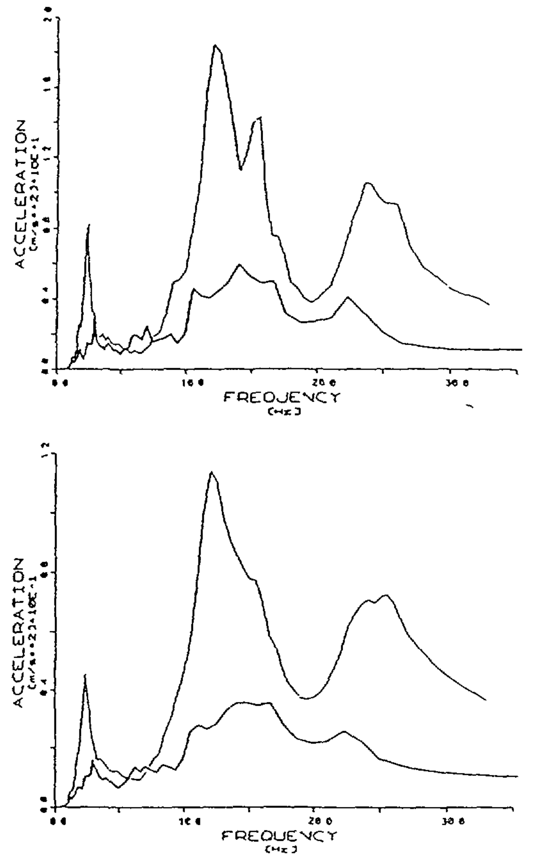

Figure 8. Results at node 65 (X)

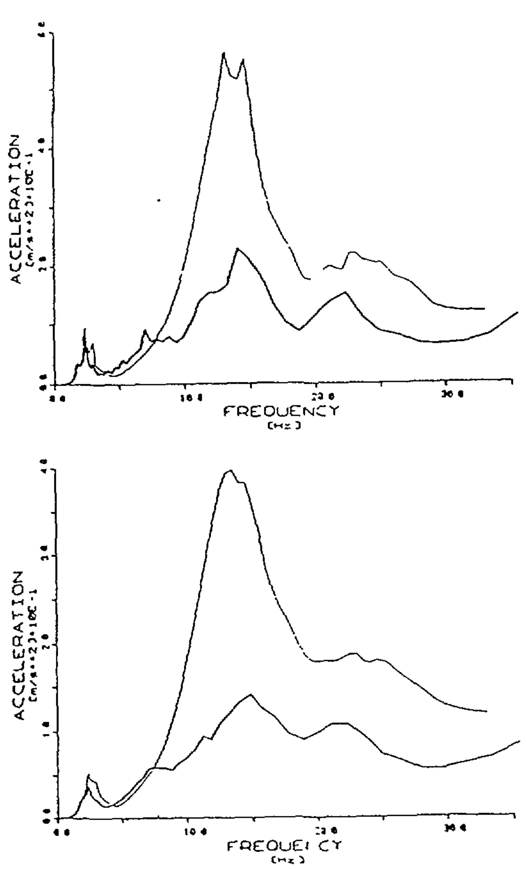

Figure 9. Results at node 65 (Z)

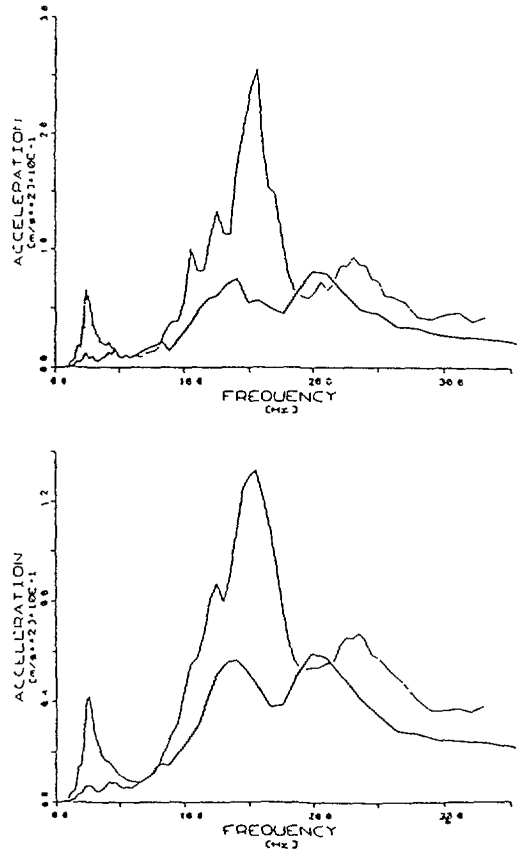

Figure 10. Results at node 656 (Y)

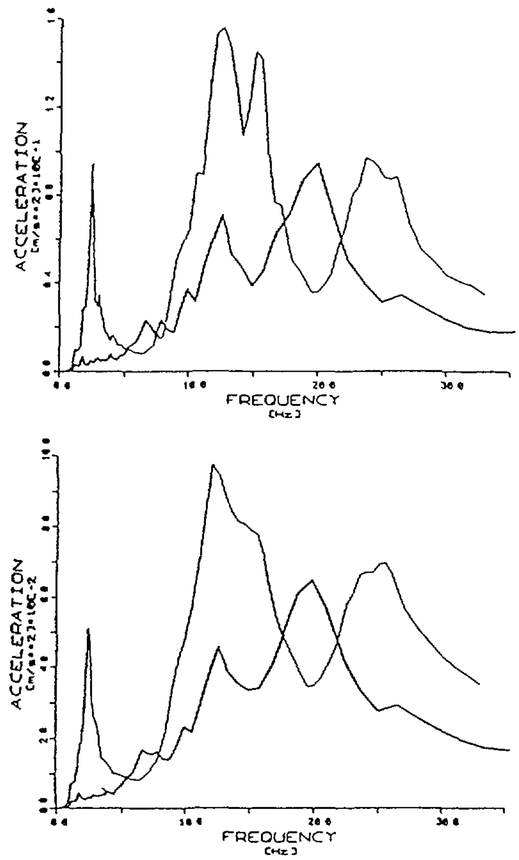

Figure 11. Results at node 656 (X)

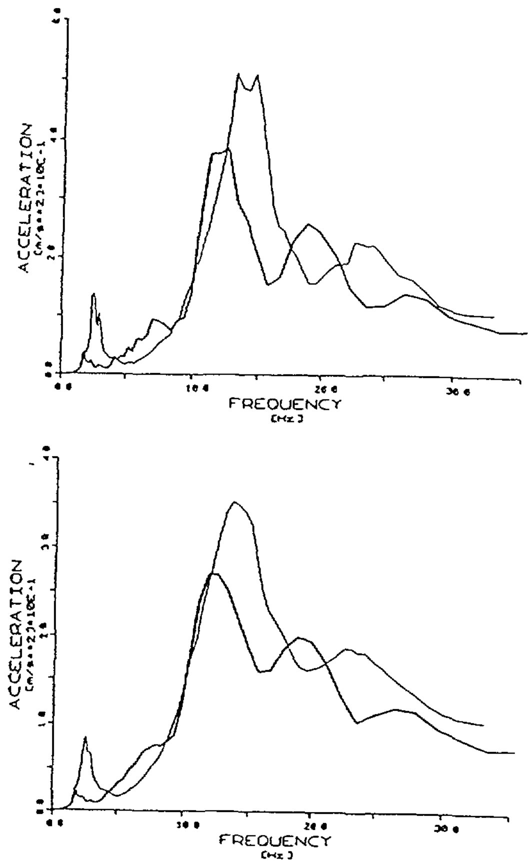

Figure 12. Results at node 656 (Z)

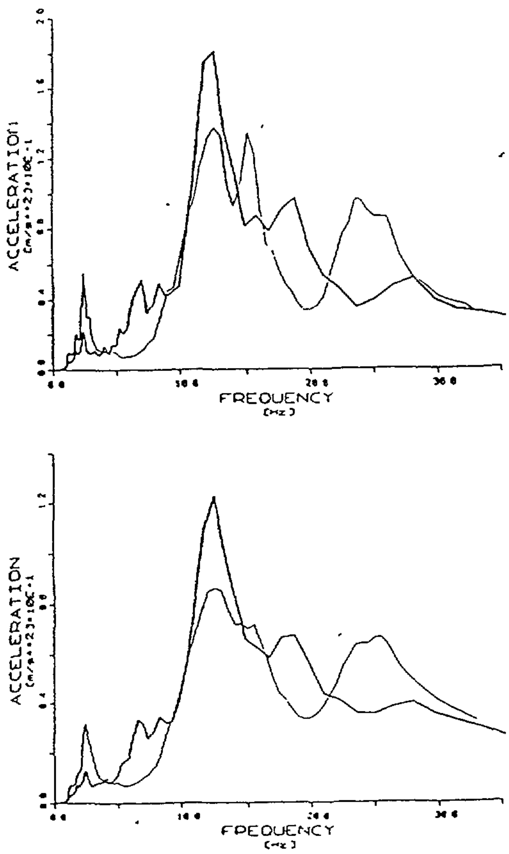

Figure 13. Results at node 753 (X)

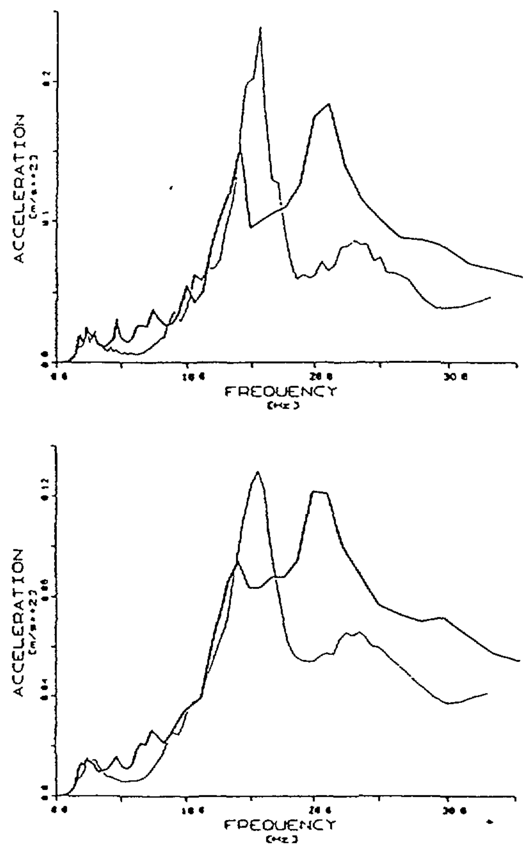

Figure 14. Results at node 773 (Y)

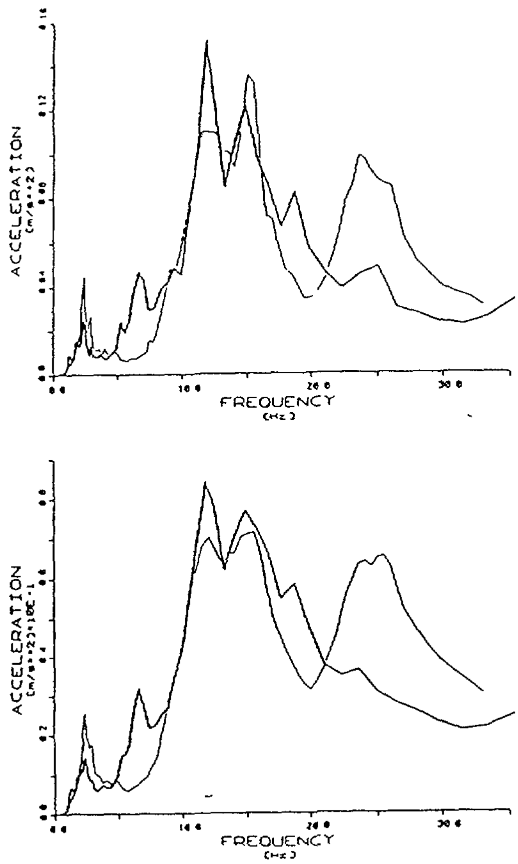

Figure 15. Results at node 751 (X)

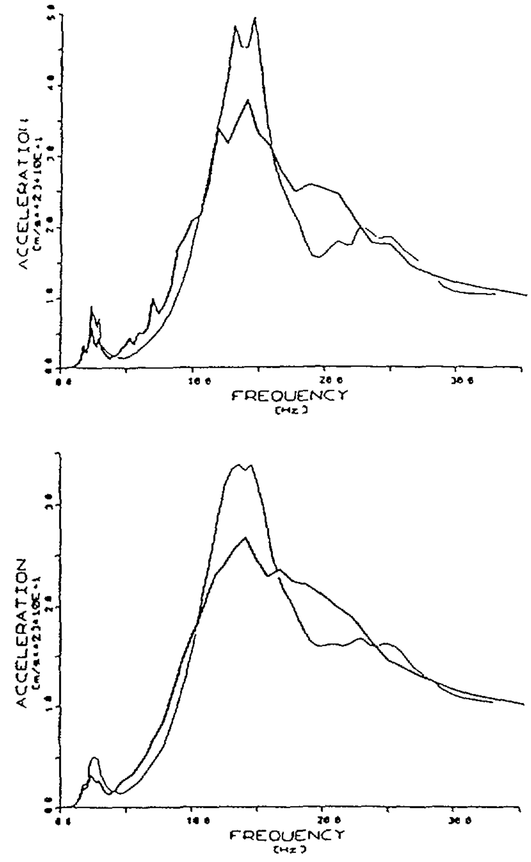

Figure 16. Results at node 751 (Z)

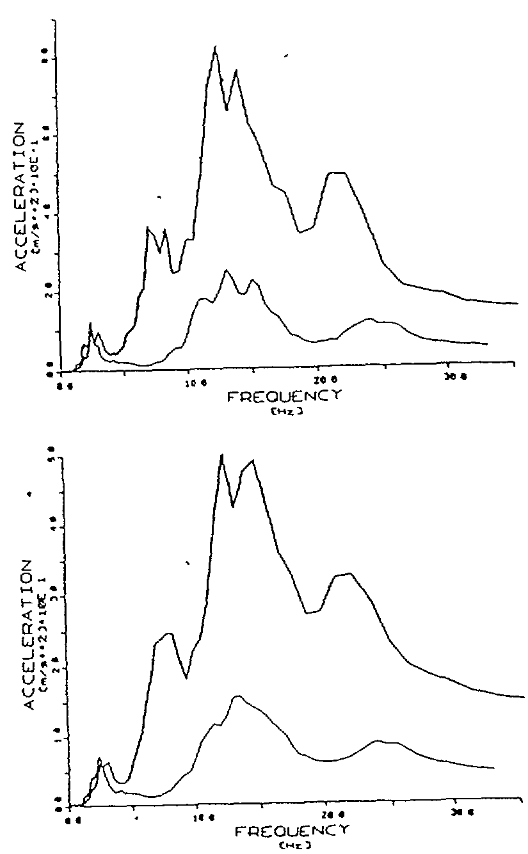

Figure 17. Results at node 2387 (X)

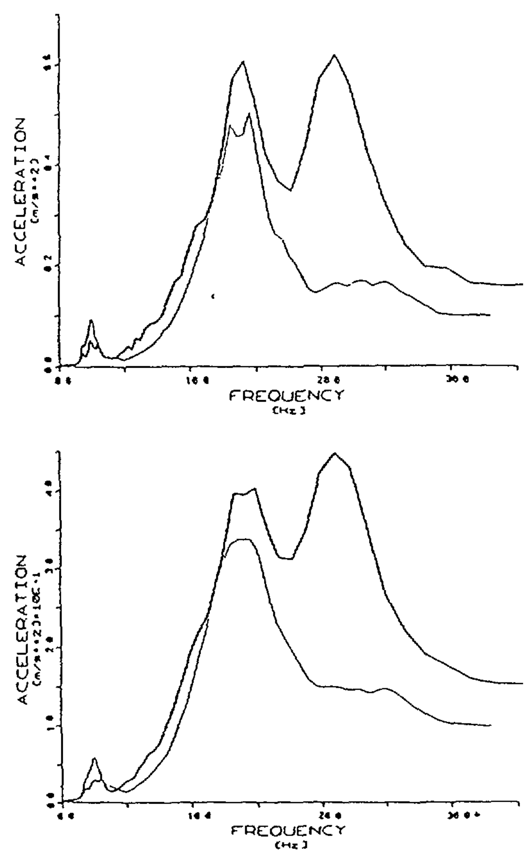

Figure 18. Results at node 2387 (Z)

5. CONCLUSION

The authors try to explain the agreement and disagreement of calculated and measured data with more precision and in greater detail by comparative calculations with different parameters. At the moment the following conclusions can drawn be out from this contribution.

- The real structural behaviour under shock loading can be described with more or less accuracy by the calculation, despite very complicated complexity of structures.
- The analysis of soil structure interaction is very important for the reliability of the calculation. It can be recommended to take into account in the soil-structure analysis the influence of all substantial structures located near to the investigated building.
- It seems that the damping ratio of the soil is higher than assumed in the calculation. In the calculation was assumed to be 30 and 15 % of critical damping for vertical and horizontal vibration modes. The damping of the structure was assumed to be 5% of critical damping. This value was most probably too high with respect to the very low level of stresses involved in the structure during the test.
- The global simplification of the structure which was used in the calculation model has not substantially affected the results, on the other hand the vicinity of measured points (the crane beams and its connection to the global structure) should be represented by a finer calculation model.
- It would be very useful to compare the results of all teams with respect to their different assumptions, different calculation models, different methods of calculation etc. in order to explain the agreement or disagreement of calculated and measured data.

ACKNOWLEDGEMENT

The authors would like to express their thanks to IAEA Vienna for the organisation of the NPP Paks explosion tests and their analysis and to ISMES, Bergamo, Italy, for carrying out the explosion tests and their primary analysis with comparison of all data, calculated and measured and for supplying this comparison to all participants of this Benchmarking. The authors do believe that the above mentioned activities will make a substantial contribution to a better understanding of the seismic behaviour and an improvement in the safety of of NPPs.

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SUMMARY OF IVO PARTICIPATION IN PAKS BLAST TEST ANALYSIS

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ABSTRACT

The paper deals with the numerical simulation of the triple blast test performed at Paks NPP. A detailed background analysis was carried out to complete the geological and geotechnical properties and, consequently, special frequency dependent soil stiffnesses have been evaluated. The structural model (3D) allowed a very refined result presentation in terms of profiles of displacements and forces at different elevations, for direct comparison with the experimental output.

1. INTRODUCTION

The IVO participation in IAEA benchmark was initiated in the research coordination meeting in St. Petersburg June 1995. The research contract with IAEA was signed in the beginning of 1996 and the funding for IVO participation was arranged by IVO R&D unit in March 1996. The volume of work for the years 1996 and 1997 has been 3 man months. In these years IVO has participated in blast test analyses for Paks and Kozloduy nuclear power plants. The Paks blast test analysis was performed in 1996 and the Kozloduy analysis is still going on. The measured responses of the Paks blast test were provided for IVO by ISMES in May 1997. The responses were mostly in form of velocities. For free field both velocities and acceleration was available. For points 46 and 47 only accelerations were available. For the rest of the points only velocities were available. The layout drawings of the Paks plant were provided for IVO by Paks plant and Dr. Gürpinar provided the soil investigation reports. This paper reports the results of the plant structural response based on the listed input data and analysis carried out by IVO.

2. BACKGROUND AND HISTORY FOR PAKS SEISMIC STUDIES

2.1 GEOLOGICAL ASSESSMENT OF THE PAKS NPP SITE

According to geological studies [1],[2], three main formation groups contribute to the construction of the geological structure of the area: Pleistocene-Holocene surface sediments, neogenic basin sediments and the Paleozoic- Mesozoic basin bottom. There are no direct data about the basin bottom in the Paks area. Its depth amounts to about 1600 - 1700 meters from the surface as obtained from geophysical investigations. Boreholes deepened around the site show the bottom to be formed of Mesozoic formations to south and west, and of crystalline masses to east. The longer part of the basin sediment on the basin bottom is formed of deposited volcanogenic layers of Karpat - Ottnang age and of a thickness of more than 500 m. The Upper-Miocene is formed of Badenian riolite tuff, suffit, sandstone conglomerate, clayey mari, "lajta" Limestone, Sarmatian conglomerate, sandstone, aleurite, clay marle, Lower-Pannonian calcareous mari and clay mari. The depth of Miocene sequence exceeds even 1100 meters in the Paks area. Deep boreholes at Paks give an unambiguous evidence of presence of drift planes often intersecting Lower-Pannonian layers. Depth of Pliocene (Upper-Pannonian) sediments amounts near to 600 m and they are representing the closing section of basin sediment with their clayey, stone powder sequence becoming more and more sandy upwards. Most of Paks environment is covered of Pleistocene-Holocene surface sediments in a large variety of structures. Most frequent formations on the west side of the Danube are red clay mud and loess with a thickness often exceeding 60 meters On the south and west of Paks, similarly as in between the Danube and Tisza, sandy stone powder and drift sand of aeolic origin are frequent formations and can be hardly distinguished from similar Holocene analogues At the west edge of Danube valley surface is formed of Pleistocene alluvial pebble and sand layers, covered in the Danube valley with younger Pleistocene-Holocene alluvial sand, argilous sand and clay Depth of Pleistocene layers is about 30 meters in the surroundings of the plant site

3. ASSESSMENT OF THE PLANT SITE SEISMICITY

In 1978 Geophysical Research Institute of the Soviet Academy of Sciences assessed the intensity of design basis earthquake to be 8 grade However, the Hungarian officials adopted the value of 5 grade and it served as a design basis In 1985 the same Soviet researchers set the characteristic intensity to 7 grade, but their model supposed the plant site to be a unique block despite the fact that they identified hints to existing fault fines in the seismic profiles In the period 1987 through 1989 Geophysical Institute of the Soviet Academy of Sciences insisted on pointing out with comprehensive studies that no fault fines cross the plant site This position was opposed by the opinion of Hungarian experts stating that one of the main fault fines of the basin bottom, the "Zagreb-Kapos-Szolnok lineament" goes practically beneath the plant site Since that time the evaluation of the seismic hazard to the plant site depends essentially on the assessment of the influence of that structure Seismological Department of the Geological and Geophysical Institute of the Hungarian Academy of Sciences in 1990 set to 0 19 g the horizontal Peak acceleration value caused by the SSE Reviewing the existing data an English Company OVE ARUP declared in early 1992 the most probable intensity value of the SSE to be 8 grade with peak acceleration of 0 34 g Relying on the Paks-Kecskemet correlation, the Scientific Coordinating committee, when determining different levels of seismic hazard took into account the possibility of Kecskemet-type earthquakes in the Paks area This is the prevailing opinion also currently The relative position of the Paks plant and the Kecskemet fault line are given in Figure 1

Figure 1 The relative position of Paks plant and the Kecskemet fault

As can be judged from the previous paragraph the seismicity of Paks site has been controversial The controversy seems to continue even today

4. GEOTECHNICAL CARACTERISTICS OF THE PLANT SITE

The naturel surface level of the site changes between 93 and 97.6 metres above the Baltic Sea lever (BSL) Before the construction the site area was levelled to 97 metres (BLS) which is the level of grade at plant Beneath the surface between 0 and -4 metres there is fine sand (loose structure, mean density). Highest water table level is at -4 m. Between 4 and 24 m there is sand changing from fine to moderately saturated with a few pebbles (of mean density and dense), from 24 to 30 metres from moderately saturated sand to saturated sand containing pebbles (of mean density and dense). Beneath 30 metres there are Pannonian sediments. For the essentially loose sediment of 30 m thickness covering the area, characteristic shear wave velocity is about 250 m/s. Shear modulus for the strong deformations is about 120 MPa. The summary of the soil investigations at Paks site can be given in the following Table 1:

Table 1 Summary table of the soil geotechnical characteristics at Paks site.

Layer id	Material	Thickness	Density	Shear modulus	Shear wave velocity	Poisson ratio
		m	kN/m^3	MPa	m/s	
1	fine sand	9.5	19.6	121	250	0.25
2	medium sand	10.5	19.6	121	250	0.45
3	medium sand	7	19.6	310	400	0.45
4	gravelly sand	8.3	21.6	608	550	0.45
5	gravelly sand	400	21.6	608	550	0.45

The graph of the cross-hole investigations at Paks site is given in Figure 2:

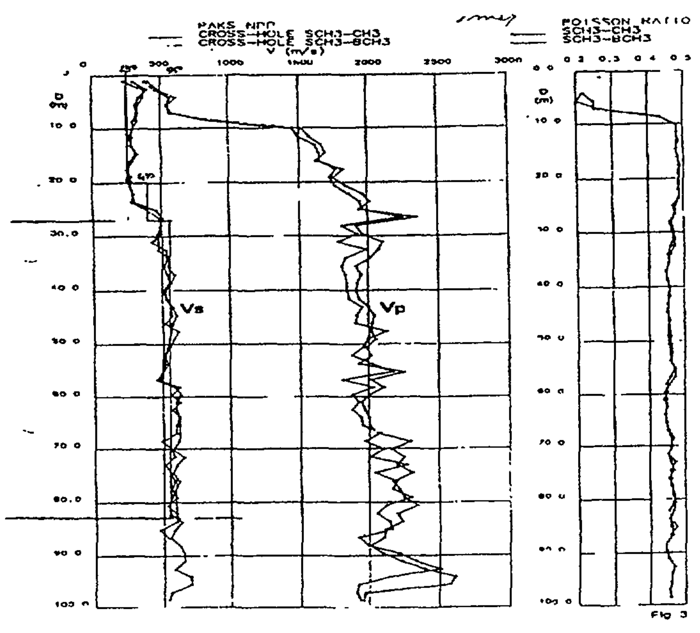


Figure 2 The profiles of shear and longitudinal wave velocities and Poisson ratio

5. DESCRIPTION OF THE BLAST TEST

The blast test was carried out in December 1994. The site was subjected to the effects of buried explosions. The aim of the tests was to induce an earthquake like excitation to plant structures and components. During the blast tests the plant operated normally.

The experiments were performed by igniting TNT charges in deep boreholes. The distance of the charges from the plant was 2442 meters from center of the base slab of unit one. The explosion consisted of three 100kg TNT charges detonated with the delay of 1.58 seconds. Each single charge consisted of two 50kg charges detonated simultaneosly in two boreholes situated 7.5 meters apart from each other.

5.1 MEASUREMENTS OF THE RESPONSES

For the synchronous recording of the above-said free-field excitation data, together with the related structural response signals during the earthquake-type excitation experiments, use was made of an advanced multichannel data acquisition and analysis system, developed by ISMES and the hardware of which was set up in a mobile laboratory parked beside the 1 st reactor unit building. This computerized data acquisition and analysis system is capable of recording simultaneously up to 52 signals at a 200 kHz sampling frequency, with real-time analog to digital conversion; it is a sub-module of AIACE (The Advanced ISMES Acquisition, Analysis and Control Environment), a hardware and software environment, that has been specifically developed for the performance of static or dynamic experiments, while providing wide data analysis capabilities. In the case of time-history data to be recorded, after the onset of the data acquisition process - which can be automatically triggered according to a specified criterion - data from all the connected transducers are fed to signal conditioners which, after on-line A/D conversion, drive directly into the computer memory. At the end of the data acquisition process, the experimental data are thus ready for graphical examinations through appropriate plotting functions, as well as for applying time or frequency domain signal analysis procedures. Once the first instrumentation layout was installed, the related shielded cablings connected and the data acquisition set up, a series of measurements were made during plant normal operating conditions, for examining the ambient vibrations' intensity levers and frequency contents. As significant noise levels were noted to be present at the higher frequencies, it was decided to make use of analogie low-pass filters in the recordings to be made, for eliminating the high frequency noise prior the digitization. Acquisitions were made with 20 Hz low-pass filters inserted in all measurements channels. These filters performed also the anti-aliasing funtions. The sampling rate of 200 Hz was chosen for ensuring the satisfactory definition of the blast-induced time histories. The full description of the blast tests is in the reference [3]. The positions of the measured responses at the base slab level are given in the Figure 3.

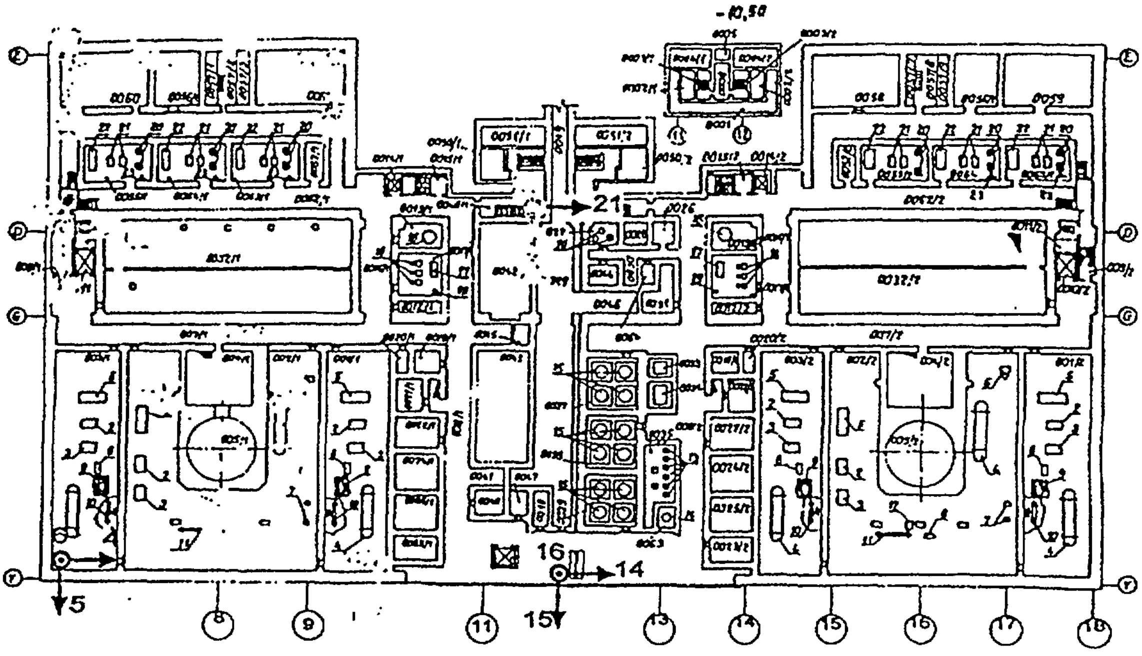
6. DESCRIPTION OF STRUCTURE AND THE USED FEM MODEL

Paks NPP is four unit VVER-440/213 type plant. The reactor bulding complex consists of four main parts: the reactor building, the condenser tower, the electrical gallery building and the turbine building. The height of the building from the grade is 50 meters and the embedment of the buildings is six and half meters. The plane dimensions the building are 72 meters in length and 52 meters in width. The thickness of the base slab is 1.7 meters. The condenser tower is based on the same base slab as the reactor and its plane dimensions are 42x24 meters. The condenser tower is a monolithic reinforced concrete structure. The reactor building consists of two separate parts. The lower part below the main operational level (+18.9) is monolithic reinforced concrete structure and the upper part so called reactor hall is the steel framed structure having the stiffness characteristics significantly less than the reinforced concrete part. The electrical gallery and the turbine building are also steel frame buildings. In modeling the steel frame was modeled with 3D beam elements and the reinforced concrete with shell elements. The general view of the finite element model is given in Figure 4.

7. SOIL STRUCTURE INTERACTION

For soil structure interaction the techniques developed by Lysmer and his coworkers in University of California, Berkeley were used [4]. The assumption of massless infinitely rigid base slab embedded to the foundation soil was adopted. Four foundation stiffness impedances and

INSTRUMENTATION LAYOUT



FLOOR LEVEL -6,50 m (BASE MAT)

Figure 3 Instrumentation at base slab level

damping impedances were developed. The soil properties needed for input for SASSI program are given in Table 2:

Table 2 Input soil properties for soil structure interaction analysis program SASSI

Shear Wave Velocity (m/s)	Density (kN/m3)	Poisson's Ratio	Damping Ratio	Thickness (m)
250	1.96	0.45	0.02	9.5
250	1.96	0.45	0.02	10.5
400	1.96	0.45	0.02	7.0
530	2.16	0.45	0.02	83.0

The resulting impedances can be plotted in X-Y plot as functions of frequency. Alltogether, three impedances were developed. Two translational impedances for horizontal and vertical directions, respectively. One rotational impedance around the longitudinal horizontal axis of the base slab was developed and this impedance was used for rotations about both longitudinal and transversal horizontal axes as well as for torsion. Also same horizontal impedance was used for both longitudinal and transversal directions. Examples impedance stiffnesses and dampings are given in the following figures.

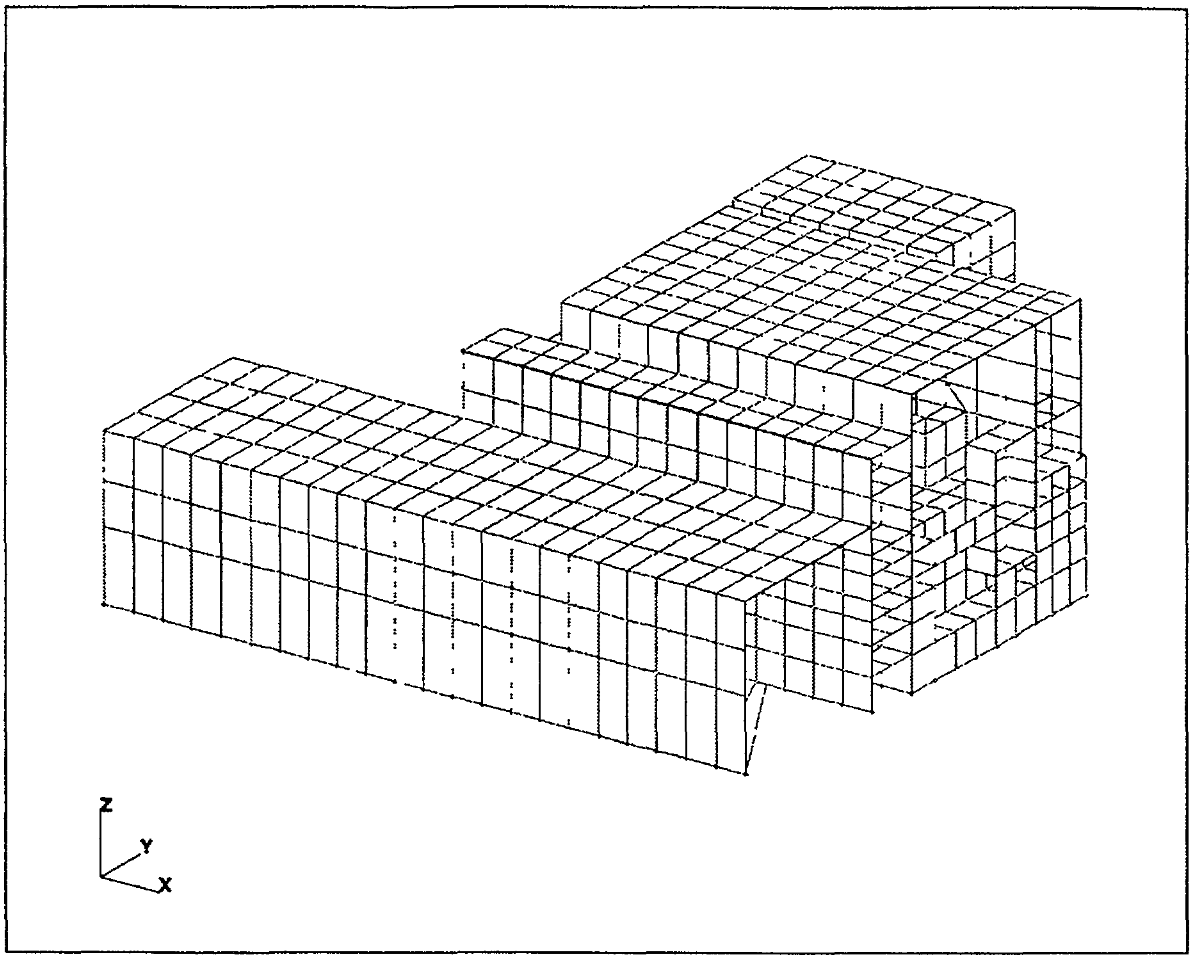


Figure 4 Finite element model of the reactor complex

8. THE ANALYSIS METHODOLOGY

The 3D structural model was analyzed in frequency domain The number of modes extracted was 530 and cut-off frequency was 25 Hz In the response history run the responses of 13 selected points were evaluated. The input values for response history run were the three components of the blast excitation which were transformed from time domain to the frequency domain with the aid of Fourier transform The analysis was carried out in frequency domain and responses were transferred back to time domain with inverse Fourier transform First the modal extraction run for the finite element model was carried out with the aid of Nastran program The three lowest frequencies calculated assuming fixed base and using the mean values developed from the frequency dependent stiffnesses are given in Table 3.

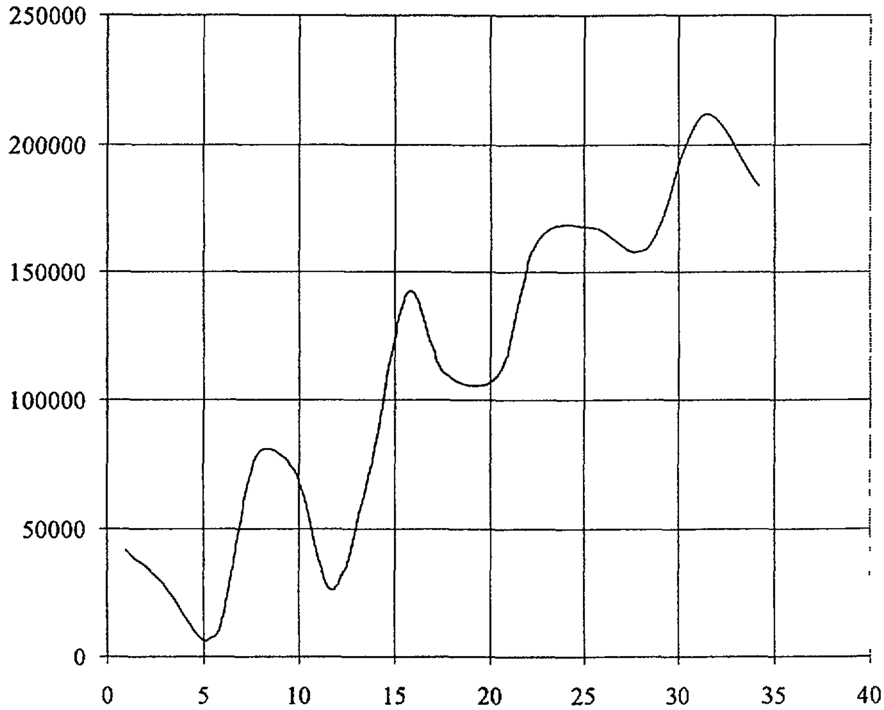
Table 3 Lowest eigenmodes of the structural model

Mode id	Fixed based frequency Hz	Frequency when mean values of impedances are used Hz
1mode	2 06	1.52
2mode	2 56	1.96
3mode	2 76	2.38

9. RESULTS

One of the main aims of the analysis was to clarify the effect of soil-structure interaction and that's why the excitation and responses are shown in the results in the same plots in order to facilitate the comparisons.

Horizontal stiffness impedance Kx(MN/m)



Rotational stiffness impedance Kyy(MNm/rad)

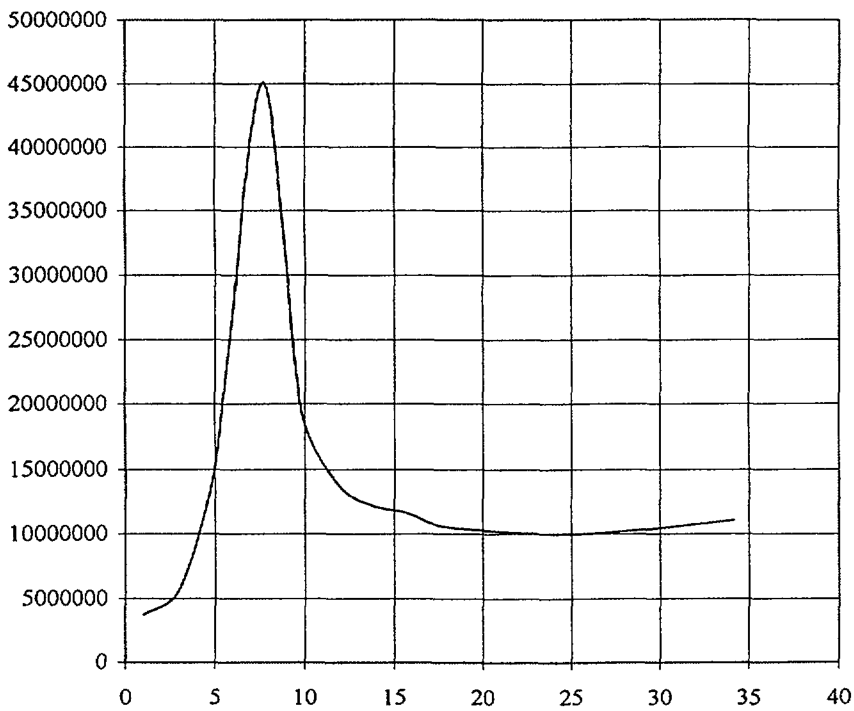


Figure 5 Examples of frequency dependent foundation stiffnesses

In the following three figures the responses at main operational level at the top of reactor shaft are depicted in longitudinal, transversal and vertical directions with the corresponding components of excitation. These results were plotted first before the measured responses were delivere by ISMES. The black line denoted as free field presents the blast excitation in North-South, East-West and Vertical directions, respectively, and the overlaid gray line presents the response of the level +18.90 of the rector building in the corresponding directions. The North-South direction corresponds to global coordinate x in Figure 4. The location of points p35, p36 and p37 is on top of the reactor shaft. In longitudinal (x) direction the motion is deamplified by a factor of 4 at maximum (base slab level) and by a factor of 3 at minimum (main operational level). In transversal (y) direction the motion is deamplified by a factor of 4 at maximum (base slab level) and by a factor of 2.5 at minimum (main operational level). However, in the crane level the motion is amplified and the peak acceleration in transversal direction is about 0.1 m/s2 compared to 0.0555 m/s2 of the free-field excitation. In vertical direction the deamplification of the motion is strongest. The peak vertical acceleration response is at crane level and is about 0.05 m/s2 and the minimum acceleration response in vertical direction at base slab level is about 0.02 m/s2. The peak acceleration of the free-field excitatation in vertical direction is 0.1729 m/s2.

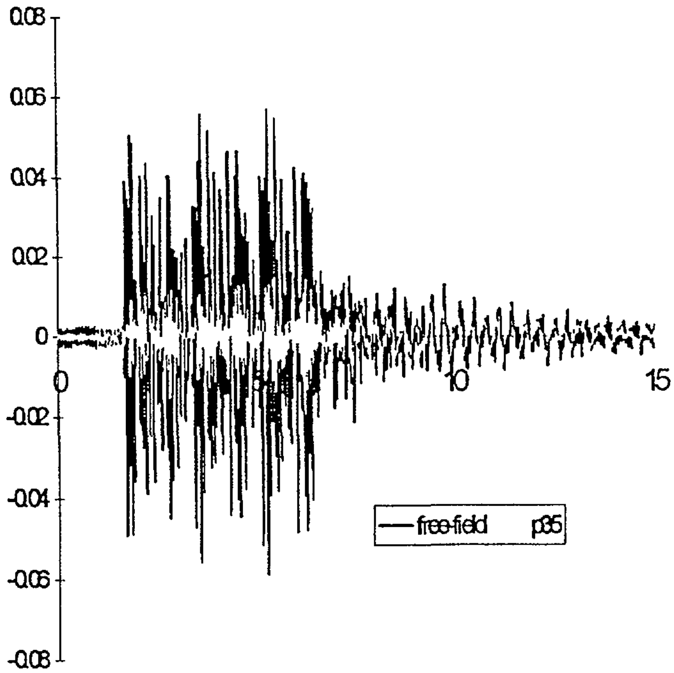


Figure 6 The longitudinal calculated response at main operational level

The measured responses were obtained by IVO in May 1997 and after that date some comparisons to the measured values have been already performed for the acceleration time histories. The comparisons have been made in acceleration for base slab, main operational level and crane level for longitudinal, transversal and vertical directions. In the following acceleration time histories the measured and calculated responses have been plotted for points P4 and P35 in longitudinal direction; for points P5, P36 and P46 in vertical directions and for points P6, P37 and P47 in vertical direction. In general, below the main operational level the maximums of measured responses are less than the calculated. This is true especially for transversal response and in lesser extent for longitudinal response. For vertical response the measured is greater than the calculated for all elevations of the reactor building. As for the amplification of the base excitation it is significant only in transversal direction and crane elevation. For vertical motion the response is less than the free field base exitation for all elevations of the reactor building. For calculated vertical response the maximum deamplification factor is 0.15 and for measured response 0.23, respectively.

For longitudinal motion the response values are calculated and measured only in for elevations up main operational level. For longitudinal motion the deamplification factor is always less than 0.5. For transversal direction the deamplification is at its maximum 0.23 and the amplification is 2.16.

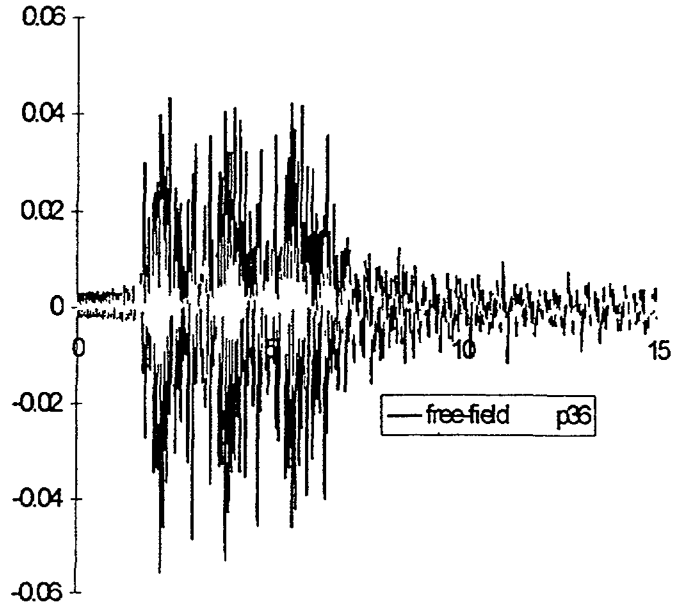


Figure 7 The transversal reponse at main operational level

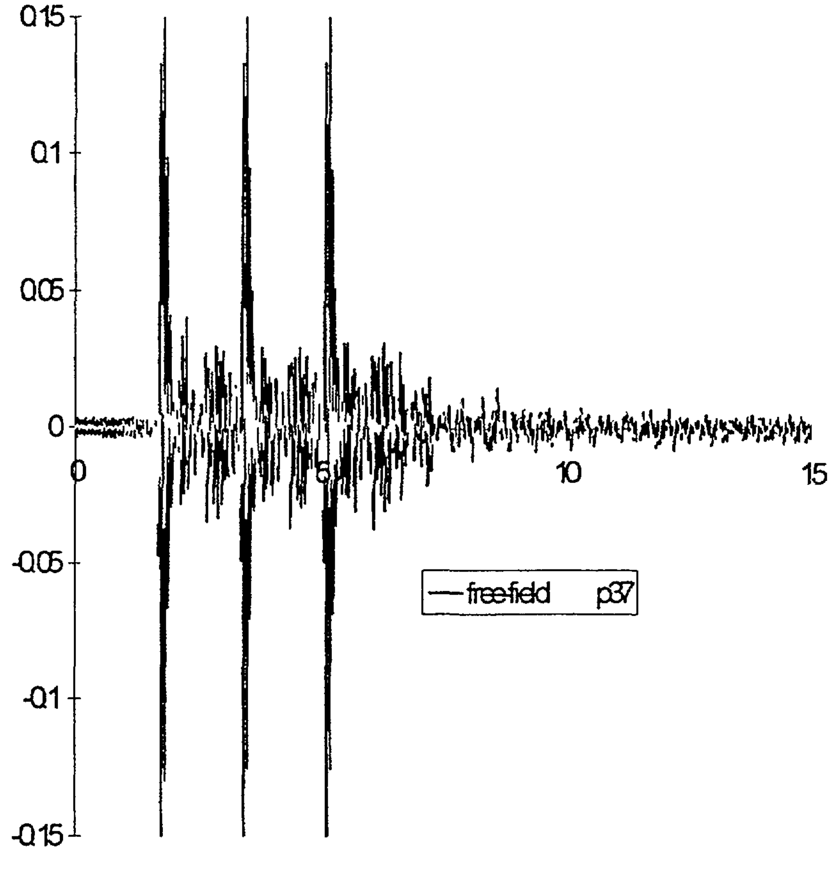


Figure 8 The vertical reponse at main operational level

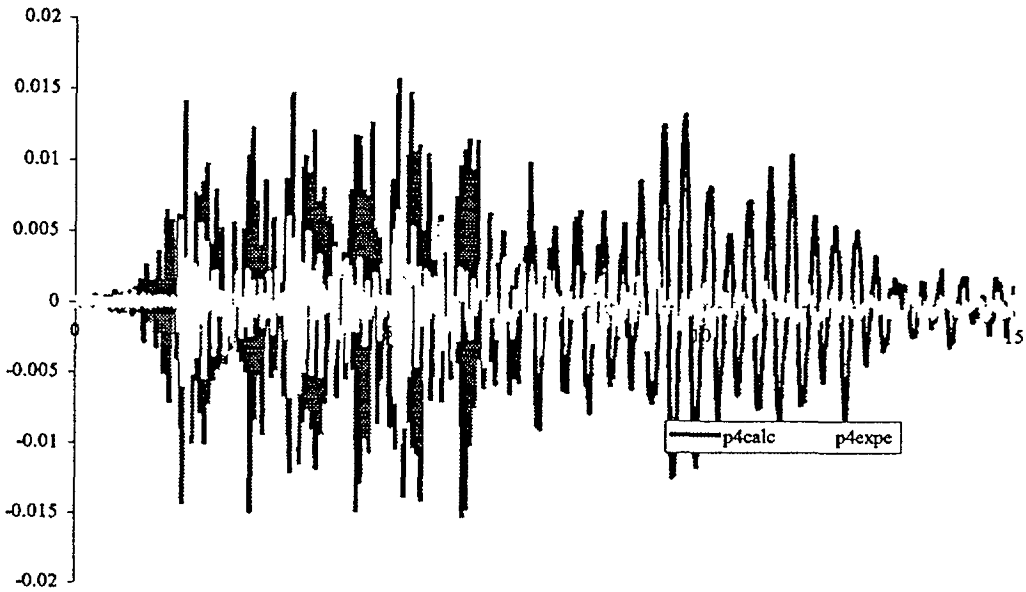


Figure 9 Calculated and measured responses in point P4

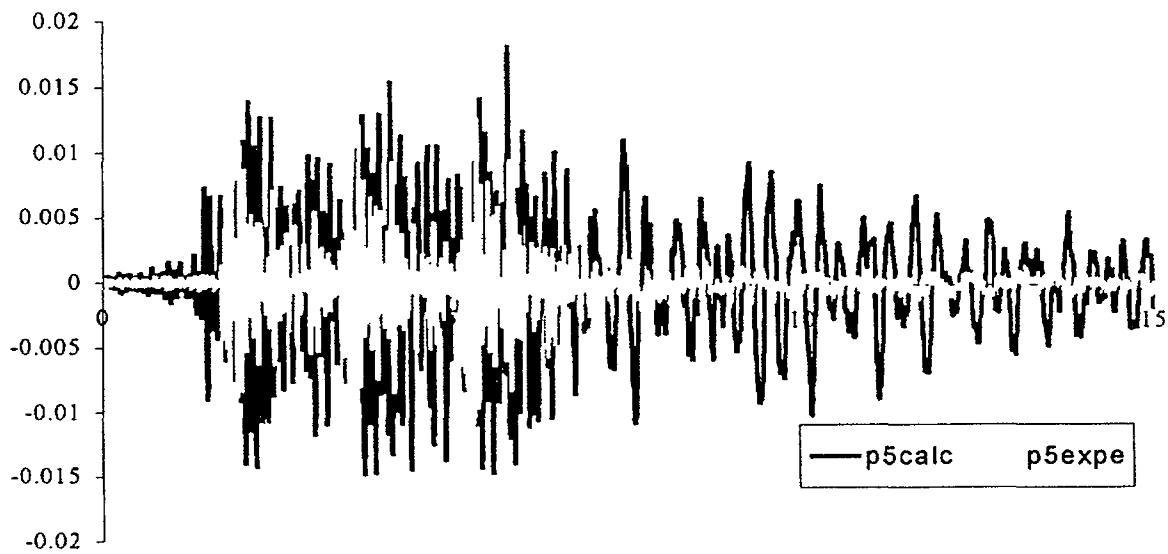


Figure 10 Calculated and measured responses for point P5

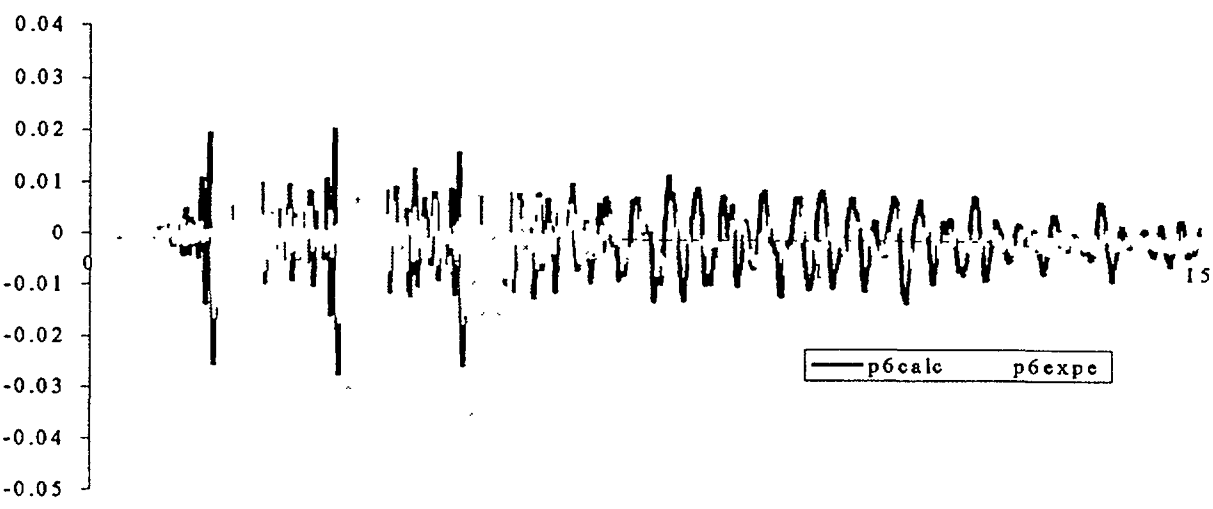


Figure 11 Calculated and measured responses for point P6

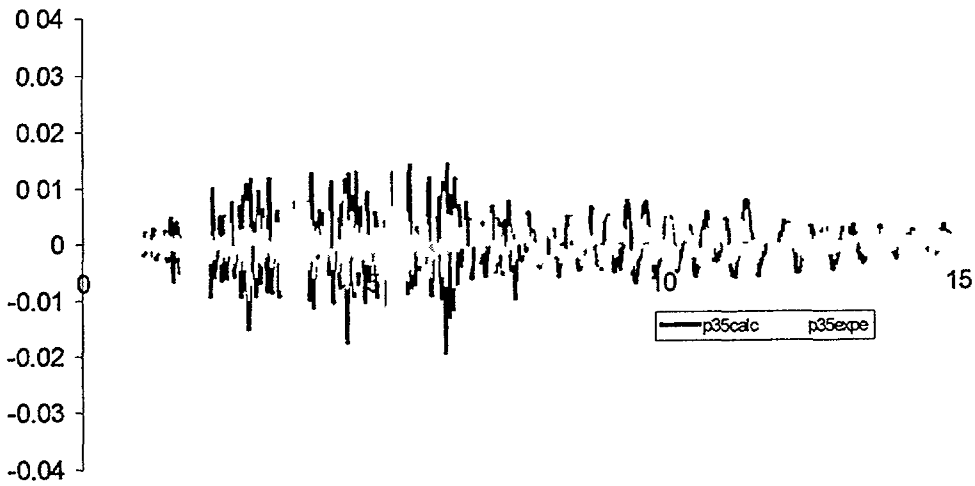


Figure 12 Calculated and measured responses for point P35

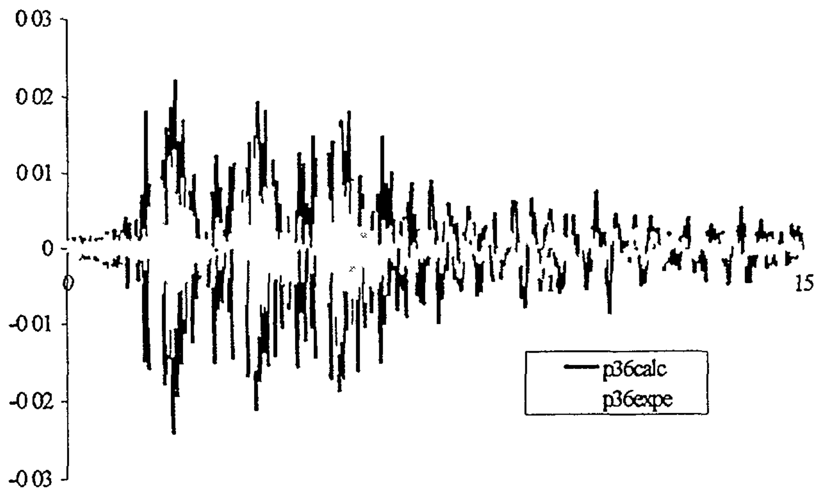


Figure 13 Calculated and measured responses for point P36

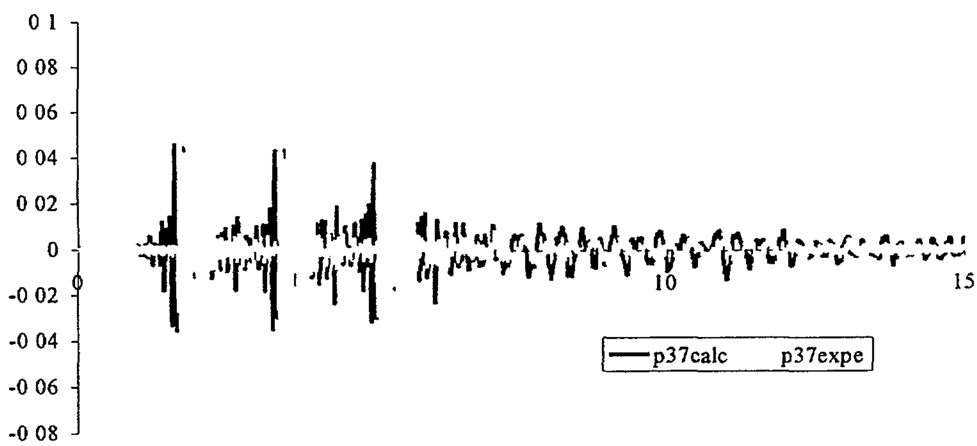


Figure 14 Calculated and measured responses for point P37

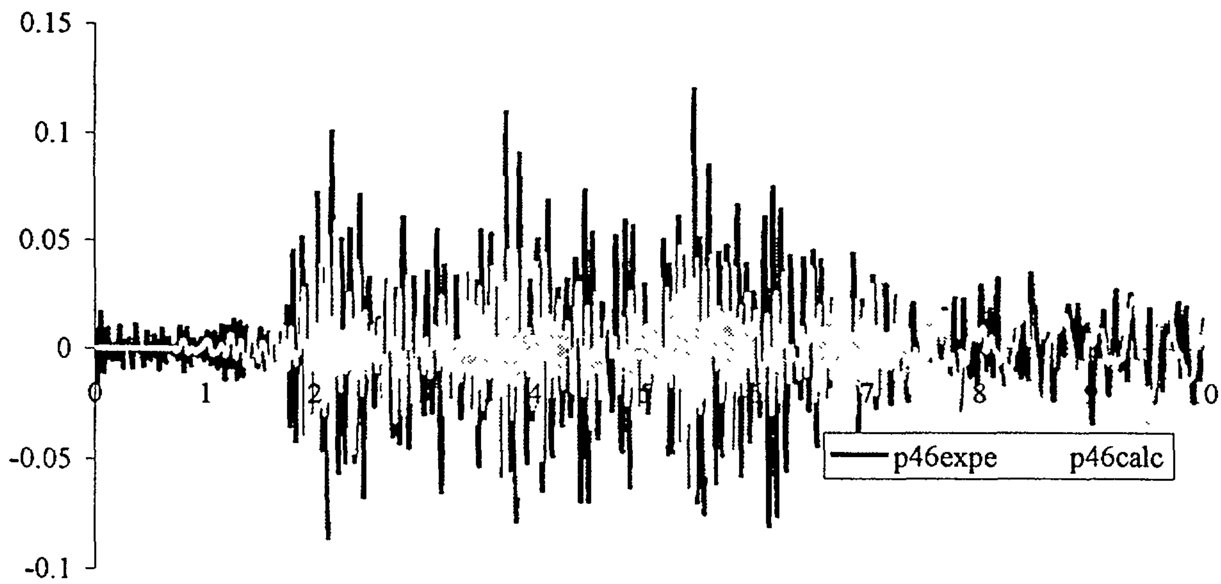


Figure 15 Calculated and measured responses for point P46

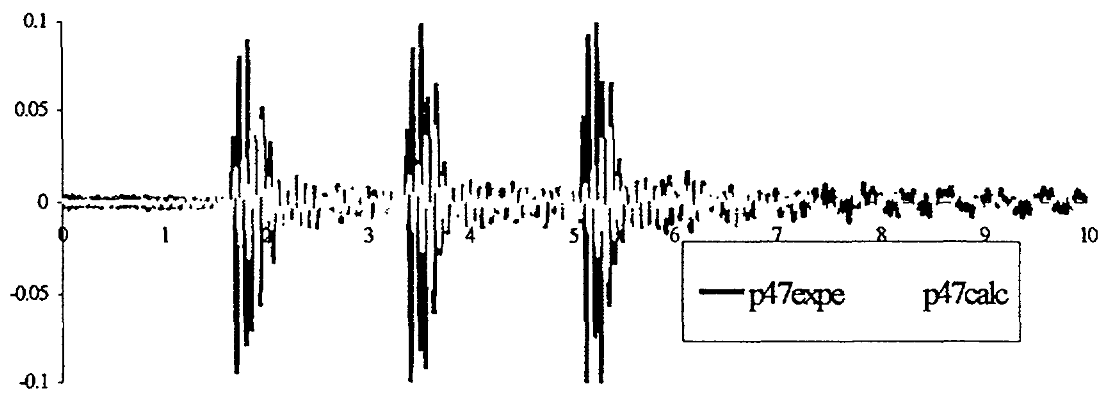


Figure 16 Calculated and measured responses for point P47

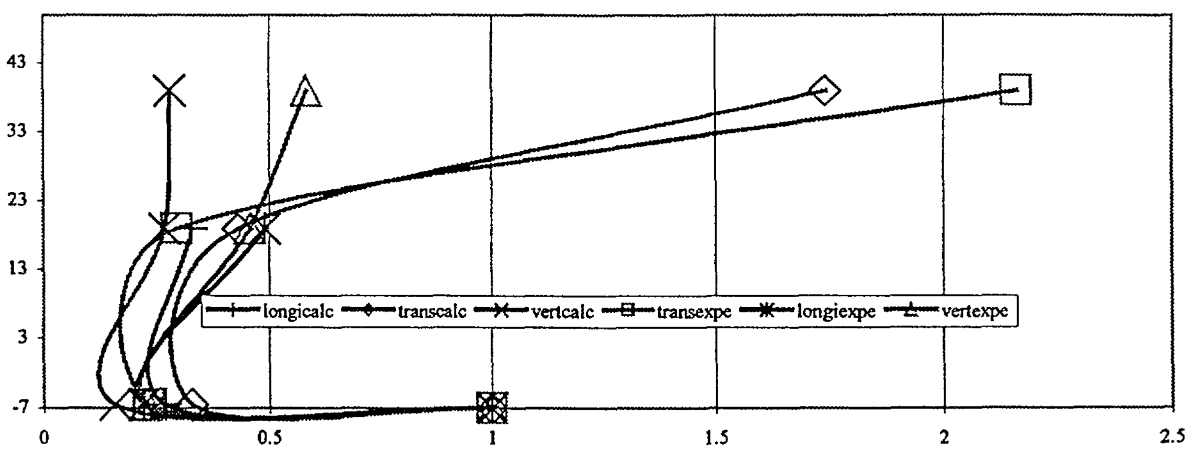


Figure 17 The of the blast responses of Paks unit 1 reactor building

10. CONCLUSION

The conclusions form Figures 8-16 can be summarized as set of curves where the calculated and measured maximum acceleration responses are plotted in vertical section of the reactor building. These curves represent the amplification and deamplification of the base acceleration in various elevations of the reactor building. Because of soft soil foundation the deamplification is strong at elevations in the stiff lower part of the building. In vertical direction there is no amplification even in the steel framed upper part of the building. The amplification of the motion is at its maximum twofold and the deamplification of the motion is at its maximum from four to fivefold.

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COMPARISON OF VIBRATION TEST RESULTS FOR ATUCHA II NPP AND LARGE SCALE CONCRETE BLOCK MODELS

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Abstract

In order to study the soil structure interaction of reactor building that could be constructed on a Quaternary soil, a comparison study of the soil structure interaction springs was performed between a full scale vibration test results of Atucha II NPP and vibration test results of large scale concrete block models constructed on Quaternary soil. This comparison study provide a case data of soil structure interaction springs on Quaternary soil with the different foundation size and stiffness.

1. VIBRATION TESTS OF ATUCHA II NPP

A full-scale vibration test results of Atucha II NPP was carried out in November of 1993 by the Commission National de Energia Atomica, Empresa Nuclear Argentina de Centrales Electricas S.A., Universidad National de Cordoba and Kajima Corporation. The main purpose of the tests was to provide experimental data on the dynamic characteristics of the main reactor building and adjacent structures of a full-scale nuclear power plant built on deep Quaternary soil deposits. Test results were intended to provide a benchmark case for control and calibration of state-of-the-art numerical techniques used for engineering design of new plants and assessment of existing facilities.

Atucha II NPP is located on the alluvial plains of Argentina on the Parana River, 100 km north of Buenos Aires. This is a low seismicity site, as results from scarce seismogenic features in the area and considerable distance to the seismically active western provinces of Argentina. Fig.1 shows general view of Atucha NPP site. The building has double spherical containment vessels, which are typical for this type of reactor, with steel inner wall (PCV) and reinforced concrete outer wall (R/B). The inner concrete structure (I/C) is encased by these vessels. The building is 60m high and the diameter of its base-mat is 60m. The supporting layer is mainly composed of Quaternary deposits of sandy clay soil, with a shear wave velocity of approximately 350m/sec and depth down to bed rock of approximately 500m. The depth of embedment is about 20m.

A total of 90 displacement components were recorded, twelve of them at foundation level of the neighboring turbine hall and at the soil surface at a distance of up to 200 m from the reactor building. Fig.2 shows measuring points on vibration test of Atucha II reactor building. Forced vibration tests were executed in November 1993 within a short period after construction was completed and before machinery installation had started. The test program included two types of dynamic excitation. The basic testing routine was a frequency sweep from 1 to 20 Hz by means of a

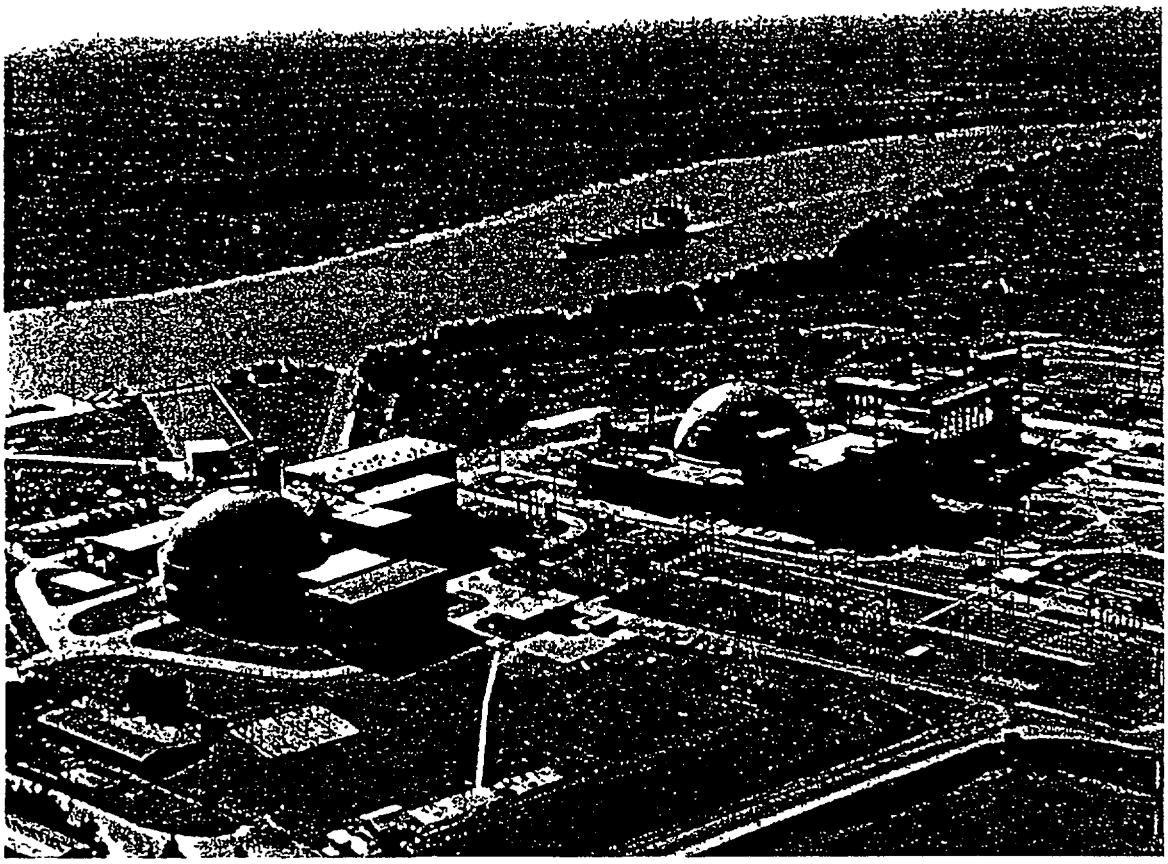


Fig.1 General view of Atucha NPP site

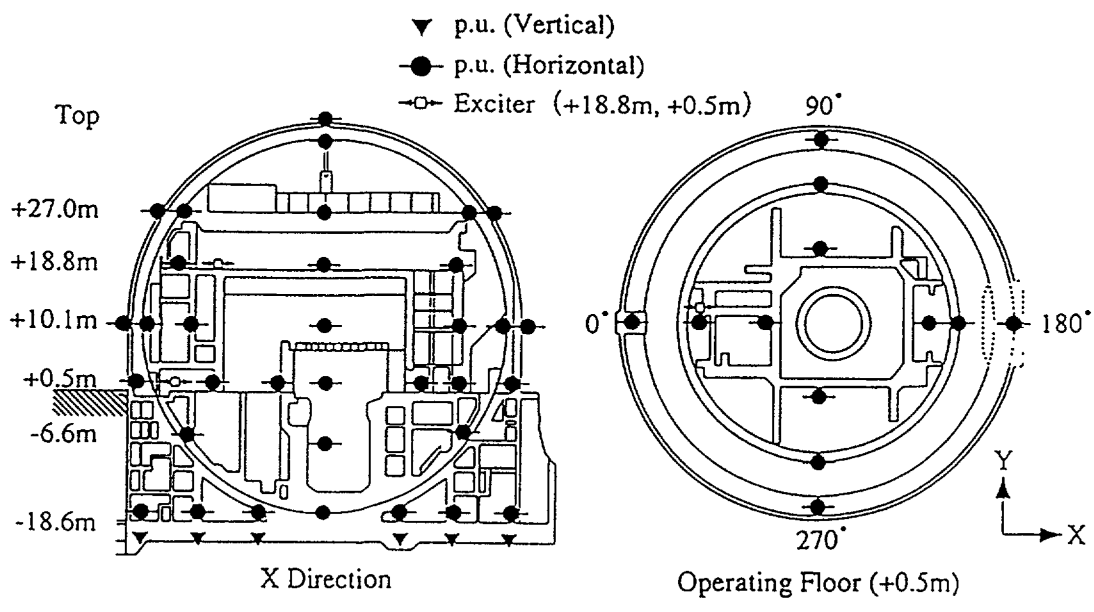


Fig.2 Measuring points in vibration test of Atucha II reactor building

mechanical exciter, with the exciter located successively in three different locations. These were provided to excite the building separately along the two main axes of the structure, and to add some degree of redundancy in the measurements. Taking the building's symmetrical shape into consideration, forces were applied along the X axis (0-180 degrees) and the Y axis (90-270 degrees), which cross at right angles on the same plane. In the X direction, an exciter was installed at two levels, GL+18.8m (at the top of the inner concrete structure) and GL+0.50m (on the operating floor), for the same measuring points. This was to observe the coupling characteristics between sway and rocking vibration. In the Y direction, the exciter was installed only at level GL+18.8m. Thus, three series of tests were executed.

Resonance and phase lag curves at the tops of the R/B, PCV and I/C for GL+18.8m excitation in the X direction are shown in Fig.3. Resonance amplitudes were normalized for an exciting force of 9.8kN. There is a small dominating peak in the range of 2.9Hz~4.5Hz, which is considered to indicate a fundamental resonance peak of the soil~structure interaction, as the phase lag curve crossing the 90 degree line. Such a wide-range low-level peak is considered to be caused by the soft soil compared to the rock and the deep embedment, which increased the radiation damping. This phenomenon is a feature of the Quaternary deposit siting. Although small peaks are observed, significant resonance peaks are observed only at 5.9Hz and 7.3Hz that are the resonance frequencies of the PCV and the R/B, respectively.

Fig.4 shows resonance and phase lag curves of Atucha II reactor building at the top of the R/B by excitation at the X+18.8m and at the Y+18.8m. Within the low frequency range, the building

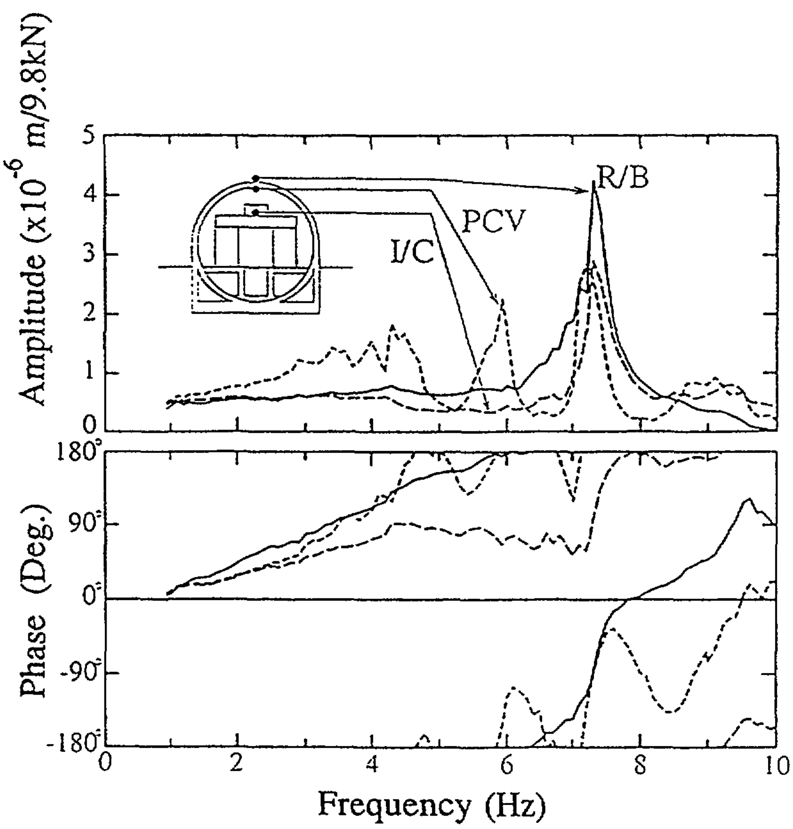


Fig.3 Resonance and phase lag curves of Atucha II reactor building by forcing at X+18.8m

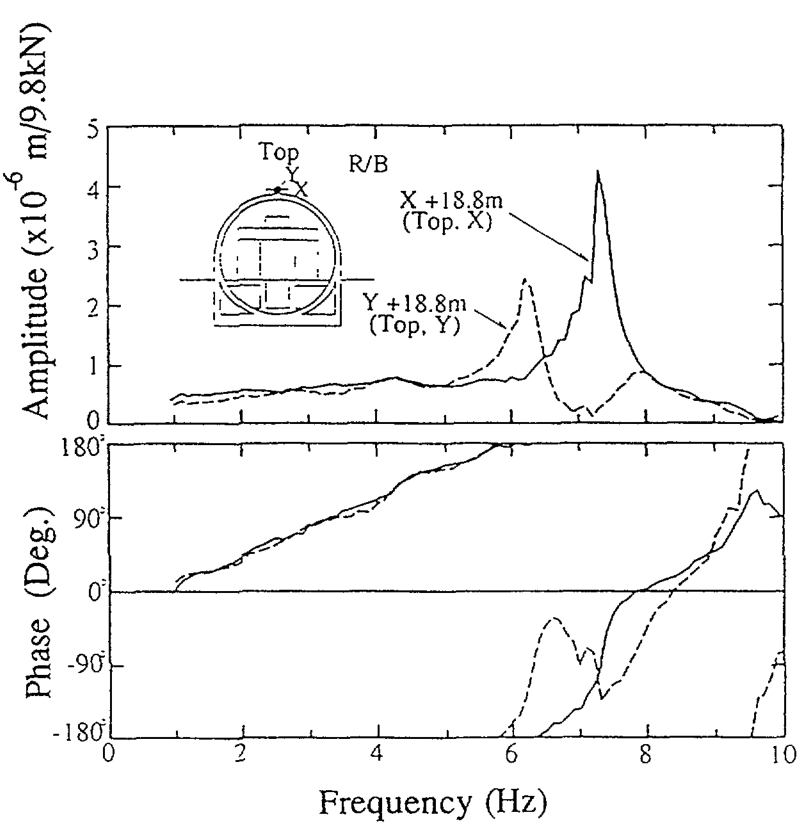


Fig.4 Resonance and phase lag curves of Atucha II reactor building by forcing at X+18.8m and at Y+18.8m

can be regarded as a rigid body, and the results show almost the same values for both amplitude and phase lag. The peak of 7.3Hz for the excitation in the X direction corresponds to the peak of 6.2Hz for the excitation in the Y direction. This difference in frequency is because the influence of the large opening at the 180 degree position. The influence of the opening is smaller for the X excitation, as it is out of the plane when the force is applied for the X excitation, while for the Y excitation, the large opening is in the plane of excitation, thus weakening the stiffness of the R/B. Fig.5 shows vibration mode shapes at 3.5Hz. Since the phase lags at measurement points of the structure at 3.5Hz are almost the same and equal to around 90 degrees, it is assumed to be the fundamental vibration mode shape of soil~structure interaction.

2. VIBRATION TESTS OF LARGE SCALE CONCRETE MODELS

The large scale field tests were performed on the grounds of Tadotsu Engineering Laboratory, Nuclear Power Engineering Center (NUPEC), Kagawa Prefecture, Japan in 1988, in order to verify the seismic stability of soil appertained to the siting technology on Quaternary deposits. For the field tests, two concrete blocks, block A and block B, were built on Quaternary gravelly soil deposits. The block A is weighing 30MN with earth contact pressure equivalent of actual reactor building, and the block B is weighing 50MN. The verification of soil-structure interaction were executed by dynamic loading tests. For the test site ground, a diluvium sand and gravel layer was

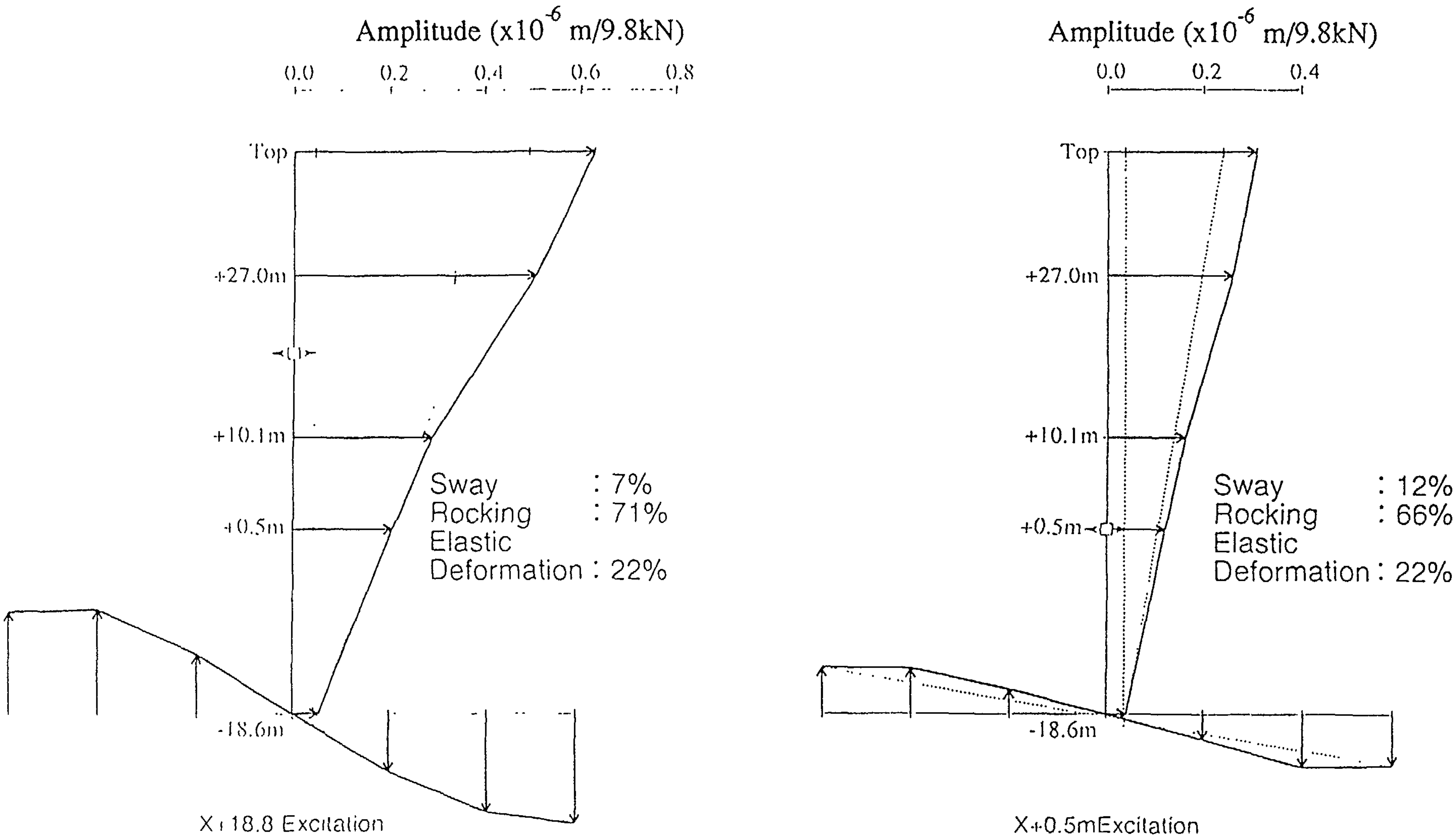


Fig.5 Vibration mode shapes of Atucha II reactor building (f=3.5Hz)

chosen, which has high possibility of being the bearing soil when building a nuclear power plant on the Quaternary deposit. There was a surface layer of about 10m thick reclamation soil on top of the selected test gravel layer, therefore, the ground was excavated to 11m below the ground surface for constructing the concrete blocks. The ground water level was lowered by using wells and controlled to hold the level of 1.5m beneath the excavated ground surface.

Fig.6 shows general view of the field test models in Tadotsu site. Fig.7 shows the relations of the test ground and the concrete blocks A and B.

The block A of 10m height was designed to have the plan dimensions of 8m×8m at the lower part and 12m×12m at the upper part, and to have the height 2m and 8m respectively, so that the

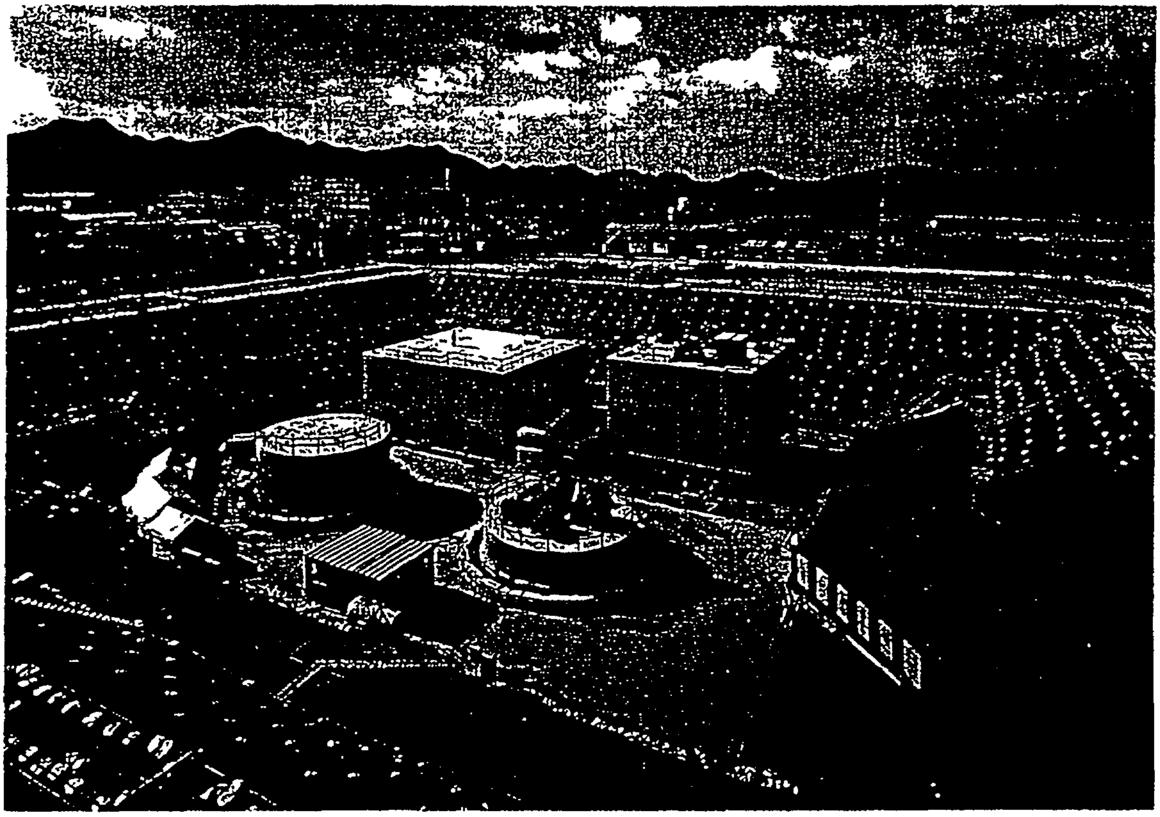


Fig.6 General view of field test models in Tadotu site

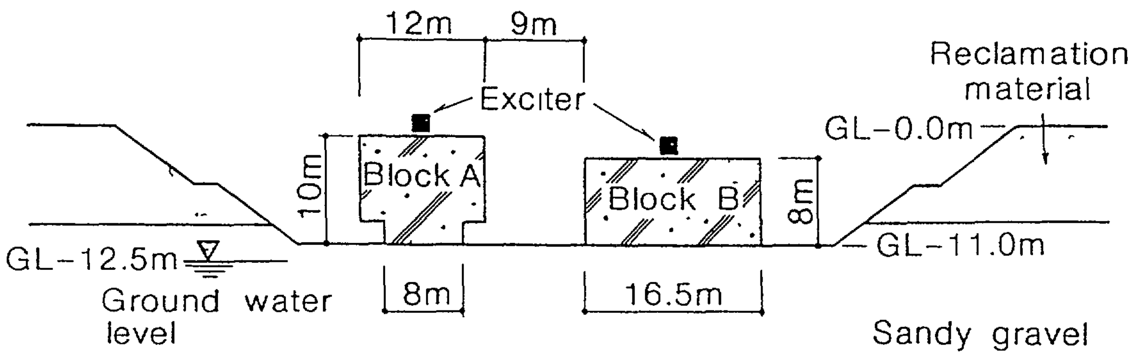


Fig.7 Concrete blocks A and B constructed on the test ground

contact pressure of approximately 470kPa could be attained. Regarding the soil-structure interaction, in order to assume the correlation with an actual building, because the non-dimensional frequency a_0 of the block A is small at 0.53, the block B was made to have the plan dimensions of 16.5m × 16.5m and 8m height, so that the non-dimensional frequency 2.09 would be the same as the actual building at approximately 2.0.

The dynamic loading test was performed by installing two sets of exciters on the top of the concrete blocks. The exciters were installed parallel to the excitations in the X direction and Y direction. Each of the selected exciter possessed the capacity; maximum eccentric moment of 6.2kN · m, maximum exciting force of 98kN, excitation frequency of 0.2Hz to 20Hz, and with plan dimensions of 2.2m × 3.7m. The excitation force was determined after confirming of its being sufficiently within the elastic limitation of the ground. Applied forces were P=19.6kN for the block A, and P=196kN for the block B. The excitations were conducted taking the procedure of increasing frequency, and were carried out by steady state tests.

Fig.8 and Fig.9 show the resonance and phase lag curves obtained at the top of the blocks A and B respectively. Each of the curve is an average of three points indicated in the figures. The

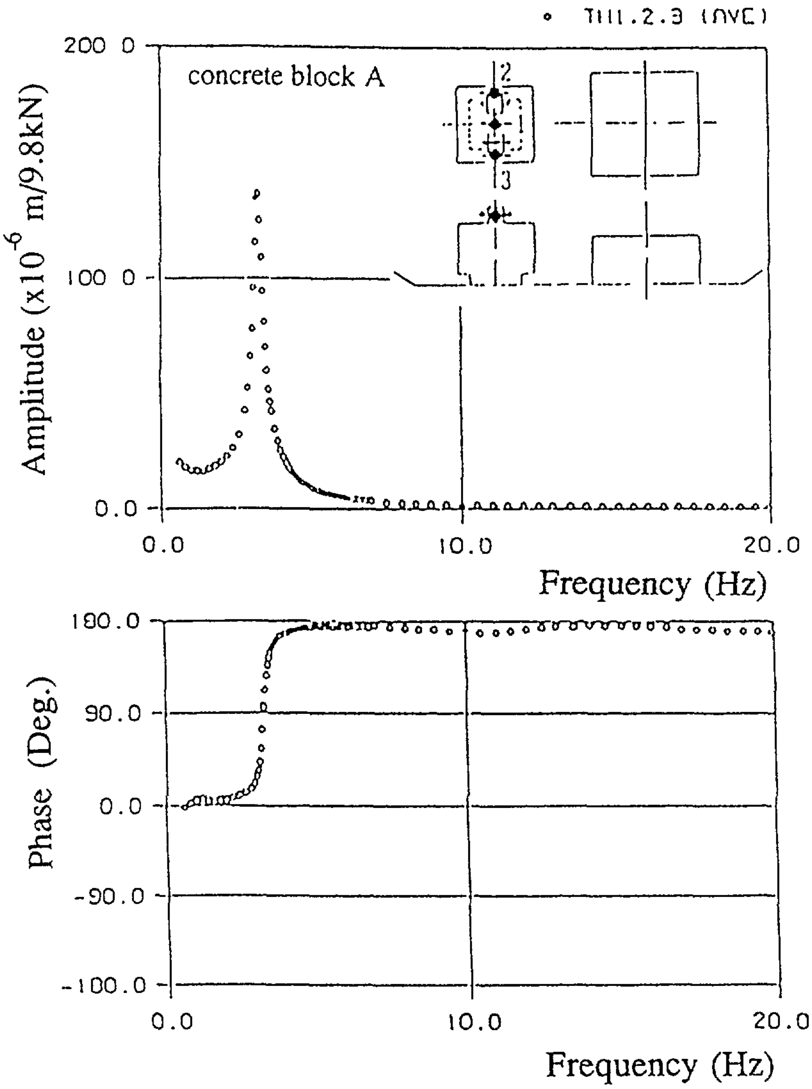


Fig.8 Resonance and phase lag curves of concrete block A (X direction excitation)

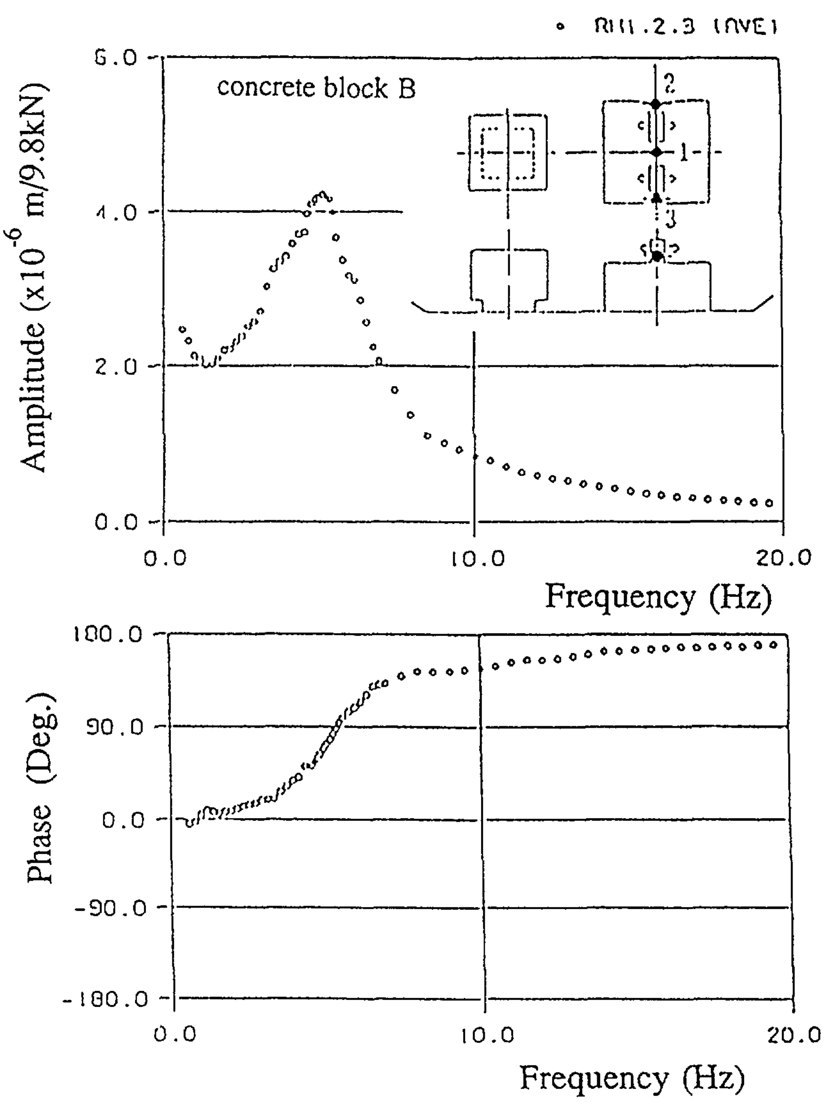


Fig.9 Resonance and phase lag curves of concrete block B
(X direction excitation)

amplitudes of the resonance curves are normalized to those corresponding to 9.8kN excitation, and the phase lag curves are indicated in term of phase lag from exciting force Regarding the resonance curves for both blocks A and B, only the fundamental resonance frequency is shown to be predominant in the range of 1.0Hz~20Hz, and the difference between X and Y directions is quite small The fundamental damping ratios obtained by power method are 5% for block A and 28% for block B in the both directions.

3. COMPARISON OF THE TESTS RESULTS

The comparison of the test results were performed in the following procedures. First, the calculation of soil springs by back fitting analyses of the test results were carried out for both of Atucha II reactor building and concrete block models. Fig.10 and Fig.11 shows soil springs concentrated at the basemat bottom of Atucha II reactor building derived by back fitting analysis in X direction and Y direction respectively. Fig.12 and Fig.13 shows soil springs of concrete block A and block B derived by back fitting analysis in Y direction forcing.

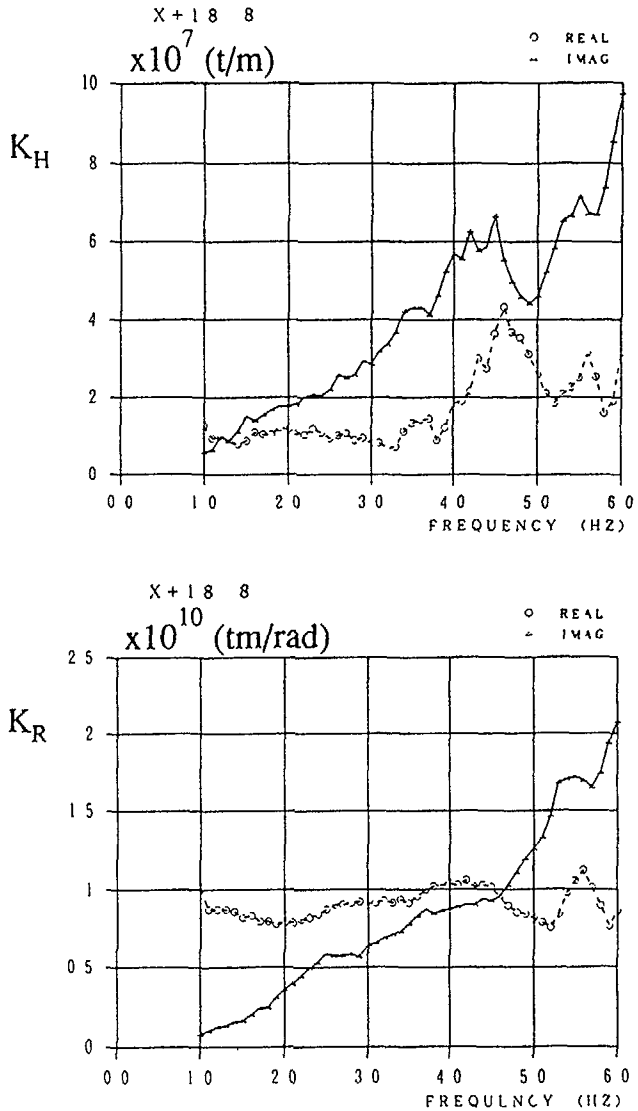


Fig 10 Soil springs concentrated at the basemat bottom of Atucha II reactor building derived by back fit analysis (X direction excitation)

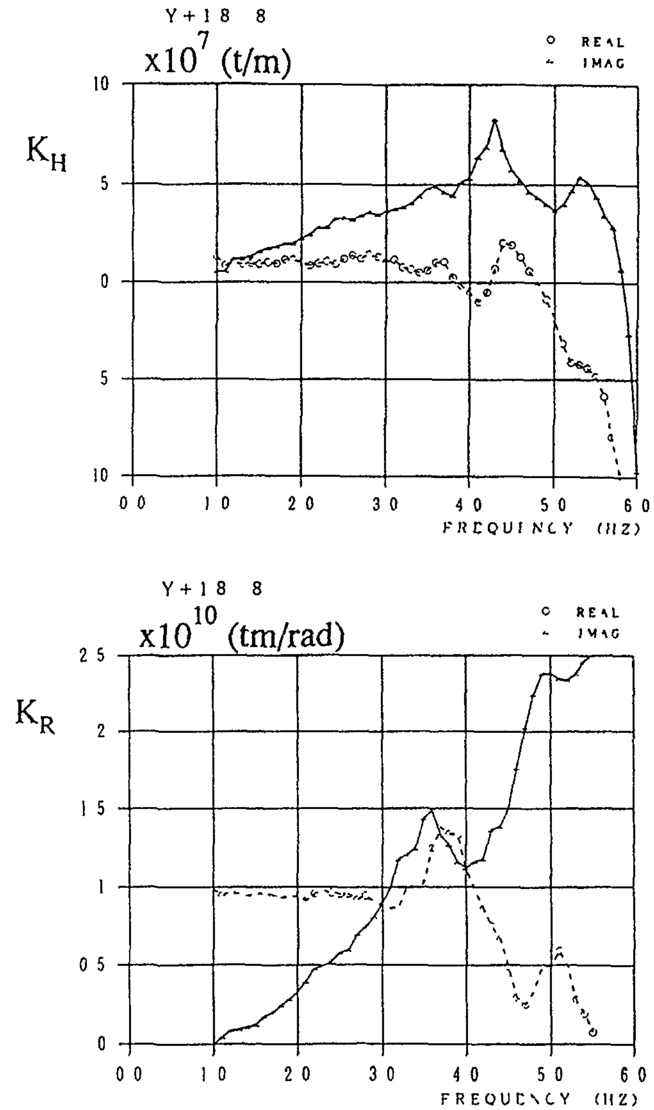
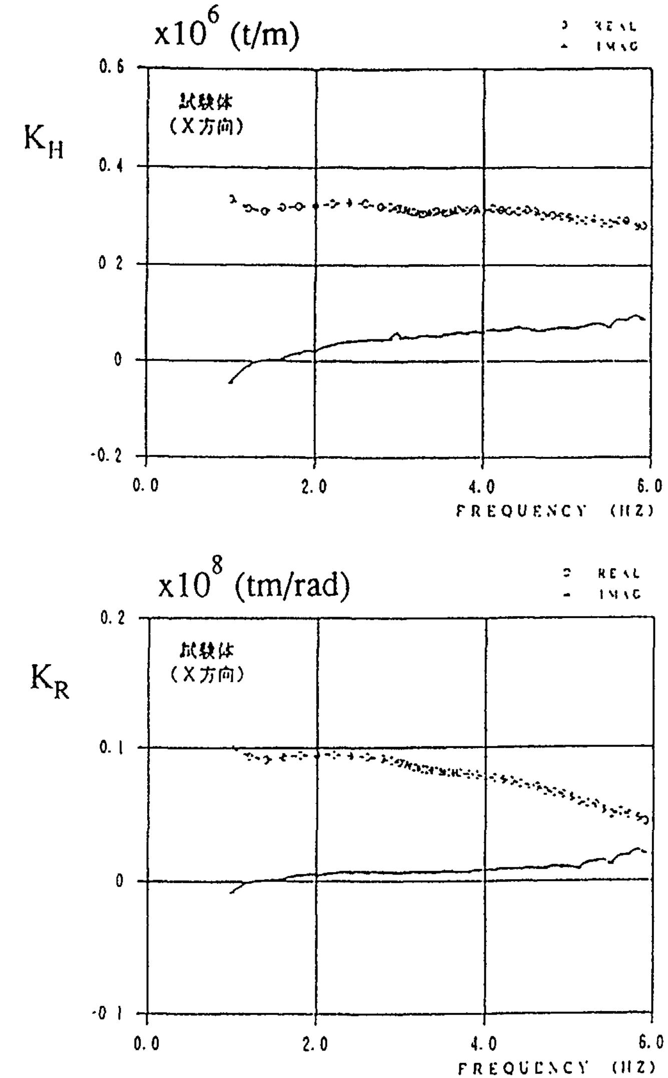
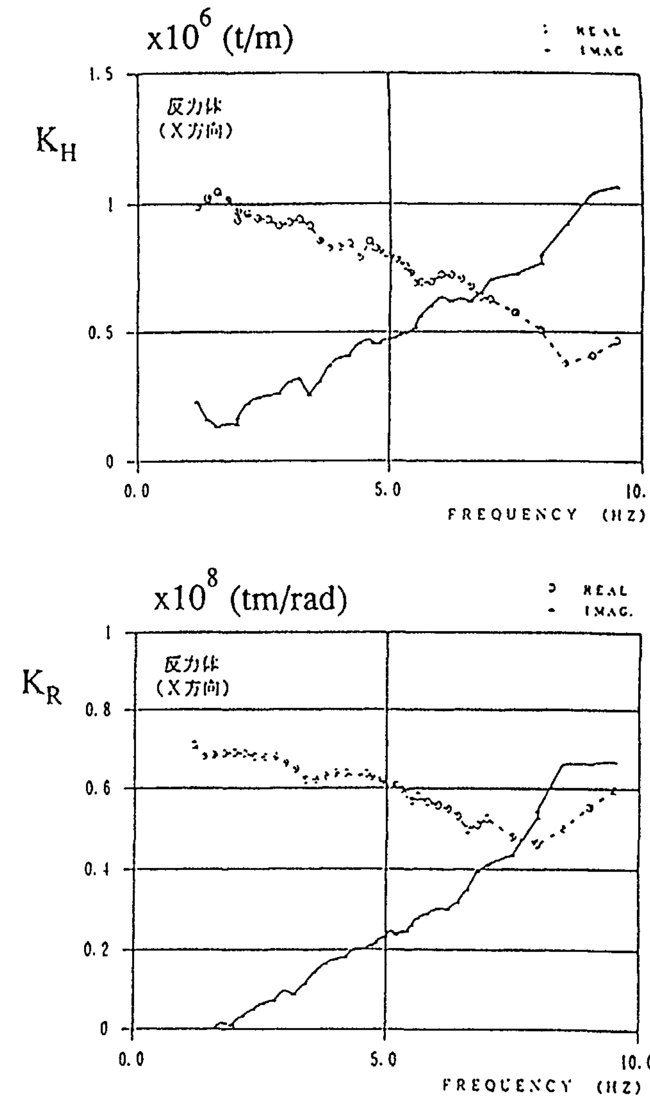


Fig.11 Soil springs concentrated at the basemat bottom of Atucha II reactor building derived by back fit analysis (Y direction excitation)



concrete block A

Fig.12 Soil springs of concrete block A derived by back fit analysis (X direction excitation)



concrete block B

Fig.13 Soil springs of concrete block B derived by back fit analysis (X direction excitation)

The soil springs of the Atucha II reactor building are included the embedment effects but the concrete blocks were not embedded. Hence, the soil springs with embedment of the Atucha II reactor building were translated to the soil springs without embedment using the coefficient ratio of soil springs with embedment and without embedment that were derived by the axisymmetric FEM analysis. Fig.14 shows the analysis result of the soil springs represented at basemat bottom of Atucha II reactor building in comparison of the with and without embedment. Fig.15 shows the coefficient ratio of the soil springs with and without embedment. The soil springs of Atucha II reactor building derived by back fitting analysis were converted to without embedment using the coefficient ratio shown in Fig.15.

Fig.16 and Fig.17 shows soil springs of Atucha II reactor building converted to without embedment, forcing at X +18.8m and forcing at Y +18.8m respectively.

Then, the damping constants were calculated using the complex soil springs obtained in order to make easy the comparison of the test results that were reflected the different structure and soil conditions. The damping constants were evaluated by the complex springs as considering that the real number portion represent the stiffness and the imaginary number portion represent the

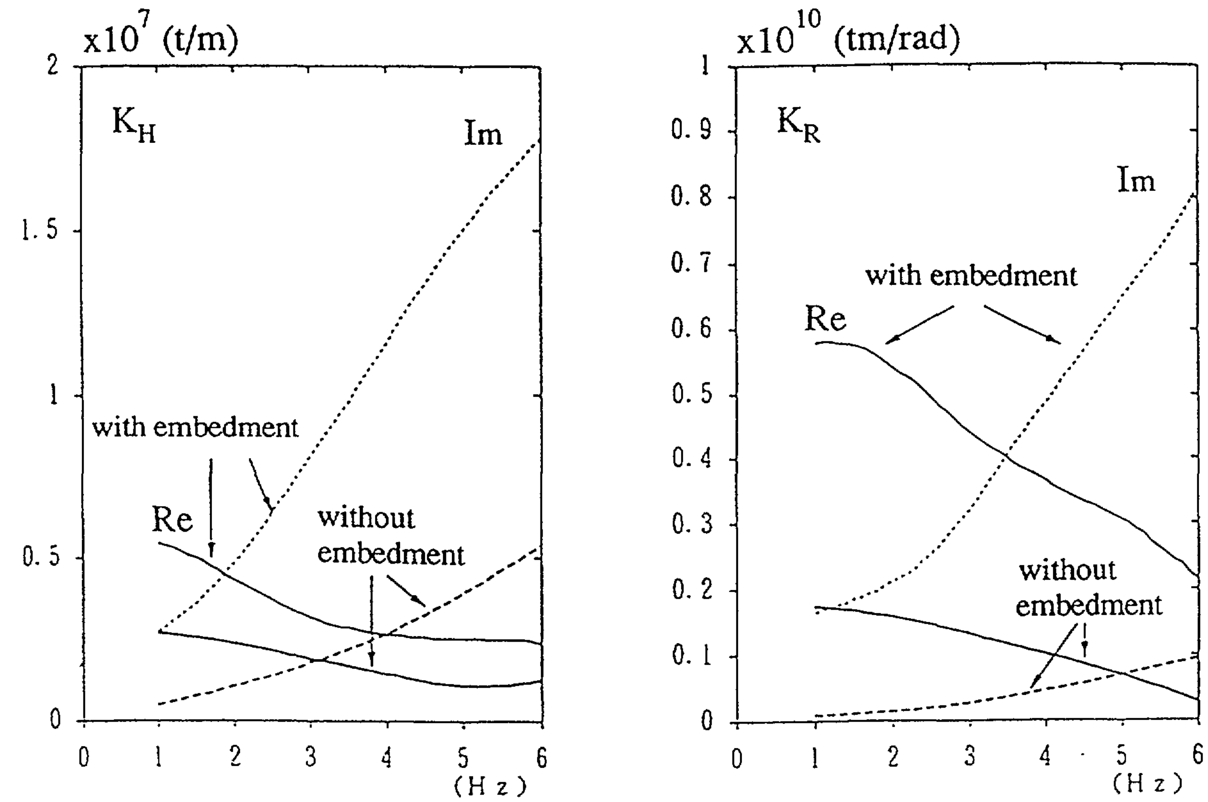


Fig.14 Comparison of the soil springs concentrated at basemat bottom as the with and without embedments derived by axisymmetric FEM analysis (Atucha II reactor building)

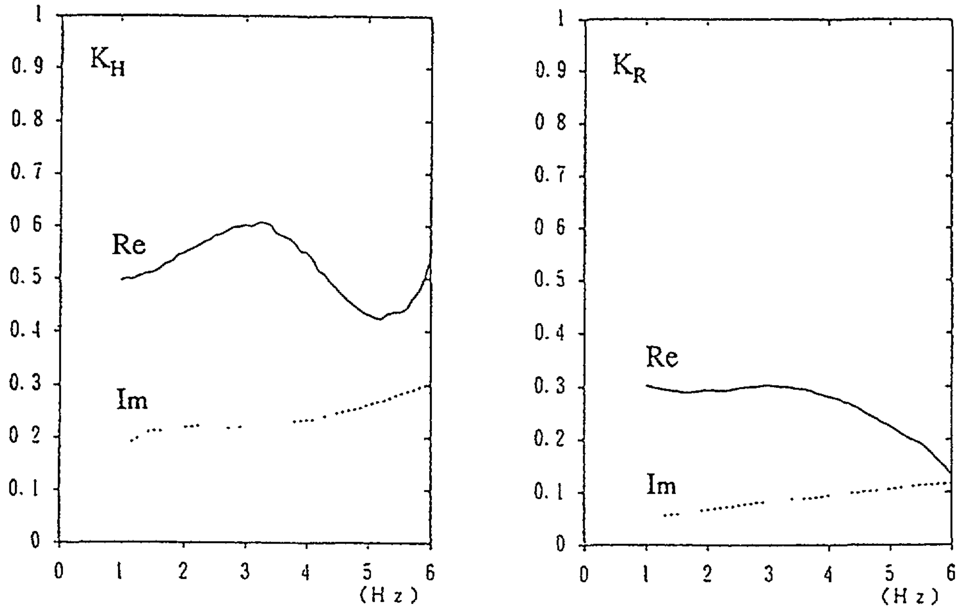


Fig.15 Ratio of soil springs of the with and without embedment
(Atucha II reactor building)

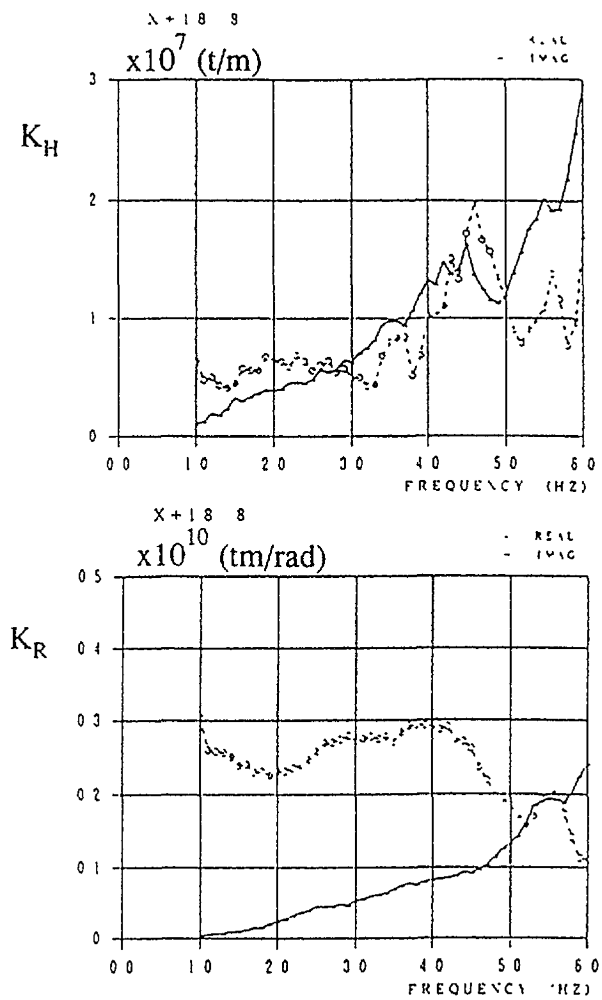


Fig.16 Soil springs of Atucha II reactor building derived
by back fit analysis converted to without embedment
(Forcing at X+18.8m)

damping. The damping constants were compared with the theoretical values for the three kinds of soil contact pressure distributions of Rigid plate, Uniform and Parabolic distributions derived by the vibration admittance theory. Fig.18 shows damping constants of concrete block A, block B and Atucha II reactor building, comparing test results and the theoretical value by vibration admittance. The damping constants of the horizontal components of the soil springs showed that the concrete block A (ao=0.53) and concrete block B (ao=2.09) showed the value like the Rigid plate distribution. The damping constants of the rotational components of the soil springs showed that the concrete block A (ao=0.53) and concrete block B (ao=1.74) showed larger value than the Rigid plate distribution and the Atucha II plant showed the value like Parabolic distribution.

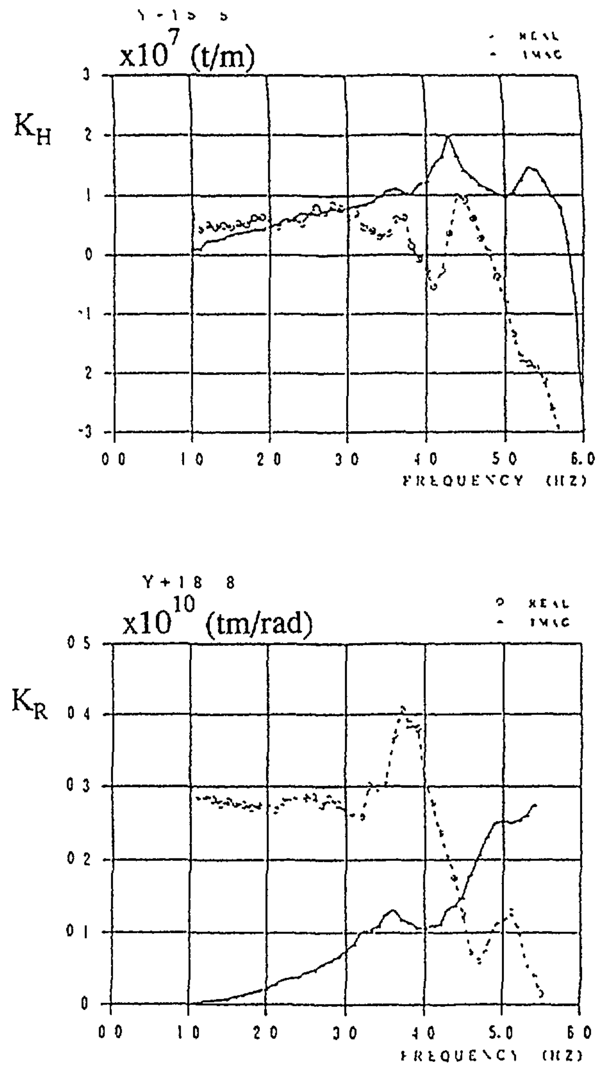
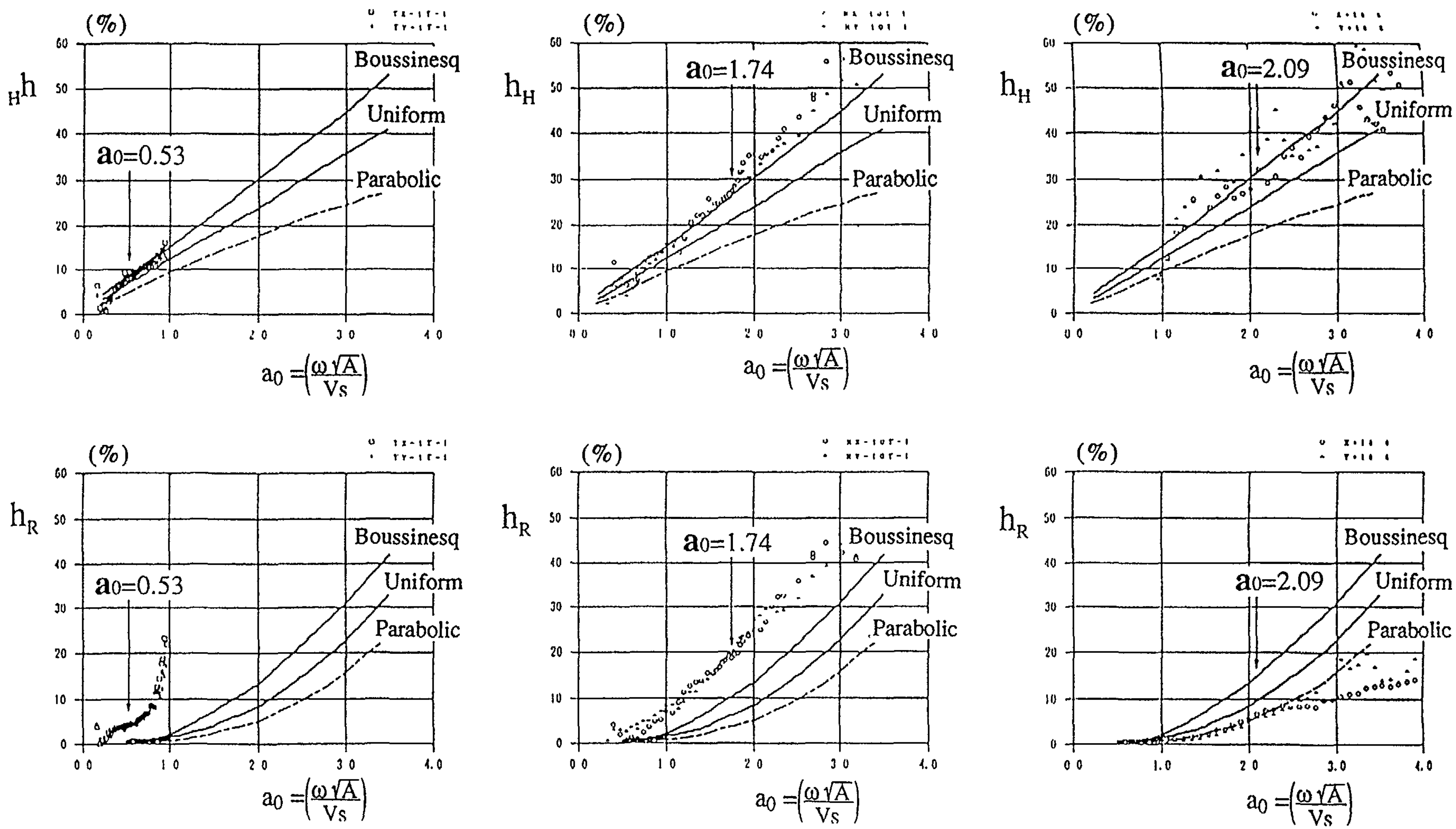


Fig.17 Soil springs of Atucha II reactor building derived by back fit analysis converted to without embedment (Forcing at Y+18.8 m)



(a) cocrete block A (b) concrete block B (c) Atucha II reactor building

Fig.18 Damping constants of concrete block A, block B and Atucha II reactor building comparing with the test results and the theoretical value by vibration admittance

4. Conclusion

From the comparison of the soil springs derived by the vibration test results for the large scale concrete blocks and the actual nuclear power plant, although, these test structures have different dimensionless frequencies as ao=0.53, 1.74 and 2.09 the soil springs characteristics observed were well correspond with the theoretical value and followings were identified.

1) To evaluate the horizontal springs of actual plants, the stiffness of the foundation can be considered as rigid plate and the distribution of the bearing soil pressure can be estimated by Boussinesq's formula.
2) To evaluate the rotational springs of actual plants, the stiffness of the foundation is mainly considered as elastic plate and the distribution of the bearing soil pressure would be varied between Uniform and Parabolic distributions.

ACKNOWLEDGEMENT

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ANALYSIS OF THE DYNAMIC BEHAVIOUR OF THE LOW PRESSURE EMERGENCY CORE COOLING SYSTEM TANK AT PAKS NPP

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Abstract

The low pressure emergency core cooling system tanks (LP ECCS) at VVER-440/V213 units have unique worm-shaped geometry Analytical and experimental investigations were performed to make an adequate basis for seismic assessment of the worm-shaped tank The full scale dynamic tests results are presented in comparison with shaking table model experiments and analytical studies

1 INTRODUCTION

In the late 1980s it was recognised that the Paks site seismic hazard may be much higher than that assumed in the design The Paks NPP launched a comprehensive programme for seismic assessment and upgrading of the plant which is due to be implemented on all of the units by the year 2002

The basic safety requirements to be maintained are to ensure safe shutdown, to cool down and remove any decay heat, and to limit radioactive release In order to achieve these goals the seismic capacity reassessment and the upgrading may be performed by applying specific methods based on the possibilities and limitations of the present operating plant Reactor shut down and the stable subcriticality could be maintained by the reactor control and protection system together with the boron system, cooling down of the reactor could be made by secondary side bleed and feed It would be possible to ensure decay heat removal by the low pressure emergency core cooling system (LP ECCS) heat exchanger after some modification Consequently, the low pressure emergency core cooling system has to be re-qualified for the new DBE level The seismic margin of systems and structures classified should be evaluated If necessary the systems should be re-qualified for the actual seismic level

In the LP ECCS there are large worm-shaped boron tanks (see Fig 1) The overall dimensions of the tanks are 6 37 m * 15 97 m * 3 86 m These are flat bottom tank without anchorage The tanks were constructed in situ from stainless steel plates having the shape of segments of a cylindrical tank The top and bottom plates are stiffened by bracing The long side walls are stiffened by three columns on each side which are by pairs connected inside the tank by tie rods The tank is filled at about 80 % of height

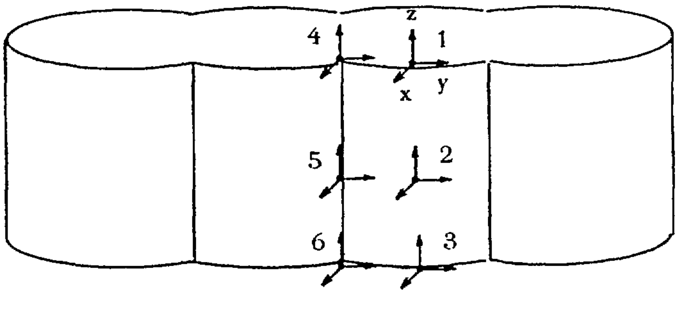


Figure 1 Location of the acceleration sensors on the worm-tank

The seismic assessment of the LP ECCS tanks having unique shape is not a trivial exercise There are two phenomena defining the seismic behaviour of the worm-shaped tanks the fluid-structure interaction and the sloshing phenomena The cylindrical as well as the rectangular tanks are well studied from these two aspects The worm-shaped tanks showing certain similarity with both rectangular and cylindrical flat bottom tanks have not been studied previously The need of experimental and analytical investigations was recognised

In frame of the IAEA Co-ordinated Research Programme on Benchmark Study for Seismic Analysis and Testing of VVER type Nuclear Power Plants the VVER-440/V213 building response has been studied by means of full-scale blast tests at Paks NPP Three series of large (up to 500 kg of TNT) explosions were carried out and the acceleration responses at characteristic points of the building structures and also the response of some large components, e g the LP ECCS tank were recorded and analysed

In 1993 the sloshing phenomenon was investigated on the shaking table in an 1 14 and 1 5 scale models of the worm-shaped tanks at the National Research Institute for Earth Sciences and Disaster Prevention (NIED), Science and Technology Agency of Japan Theoretical investigation of the modelled cases was carried out at Argonne National Laboratory Finally in1996, a very detailed experimental investigation of an 1 3 scale model of the worm-shaped tank was performed on the shaking table at NIED The full scale as well as the model experiments have been already reported at IAEA Co-ordinated Research Programme Meetings

Here, the full scale dynamic test and its result are briefly presented The full scale test results are discussed and compared with the results and finding of the shaking table experiment The comparison of the results of these two tests as well as dynamic calculation of the tank led to the better understanding of the dynamic behaviour and the seismic capacity of the LP ECCS tanks

2 SEISMIC ASSESSMENT OBJECTIVES

The objectives of the seismic assessment of the worm shaped large tanks, named by the experts simply as worm-tanks, are rather complex The phenomena determining the worm-tank behaviour are the fluid-structure interaction and the sloshing In the paper [1-3] all aspects of the fluid-structure interaction and sloshing phenomena were considered

The questions of the worm-tank seismic evaluation important directly from practical point of view are as follows

1 Whether the bottom plate and the side walls have sufficient strength especially at the corner of the bottom and side, whether buckling of the side wall could occur?

2 Whether or not an anchorage has to be added to avoid sliding or uplift of the tank?

3 Whether the strength of the connecting pipe nozzles (especially if the tank will slide or lift up) is sufficient?

4 Whether the sloshing may cause a damaging impact on the top plate and a consequent overflow?

The tasks to be investigated and the method of investigation in relation with the above mentioned questions are as summarised in the Table 1

It is obvious that the full scale dynamic test is only limited use because the low energy of the blast excitation The most powerful methods of investigation are the shaking table tests in combination with FEM analysis Nevertheless the full scale test gives the scale effect and the influence of the as-built conditions on the dynamic characteristics of the tank

Table 1

TASK	DATA	METHOD		
		FEM analysis	full scale test	shaking table test
Seismic response	eigenfrequencies, modes	Y	Y	Y
Strength of the side wall and bottom plate	dynamic pressure, overturning moment	Y	N	Y
Sliding rocking up lift	base shear force friction between bottom plate and floor overturning moment	Y	N	Y
pipe nozzles	reaction forces of piping	Y	N	Y
sloshing. dynamic impact, overflow	natural frequencies wave height	Y	N	Y

Y - used and appropriate method
N - not an appropriate method

3. THE FULL SCALE TEST

3.1 Description of the tests

The acceleration response of the worm-tank to the blast excitation has been measured at six different locations as follows (see Figure 1):

- points No 1 and 4 at the joint of the top plate and side wall,
- point No 2 at the side wall of the tank on a U shape plate,
- point No 3 on the base frame of the tank,
- points No 5 and 6 on the side wall column

Data were recorded as follows

Sampling frequency. 100 Hz
Length of a record· 37 4 s
Range 14 bit

More details on the blast experiments at Paks NPP one can find in [4]

3.2. Data evaluation

For the data evaluation a software for dynamic modal analysis (STAR - Structural Measurement Systems Inc)) was applied The eigenfrequencies and the corresponding mode shapes were obtained using experimental modal analysis method for the No 1 test data The eigenfrequencies and modes obtained in a such manner were checked comparing the results obtained from No 3 test, and looking the phase behaviour of the cross-spectra calculated for different points (in-phase and out-of phase motion at different point corresponding to the mode shape)

The complex spectra have been calculated from the time signal and the transfer function between point No 3 and other point have been obtained. The results have been checked comparing the transfer functions with those obtained between point No 6 and other points. The signals measured at points No 3 and 6 characterise the excitation itself. The eigenfrequencies were obtained using polynom-fit or peak-fit technique The polynom-fit technique provides the damping value too As a cross-check. the same procedure was applied for the test No 3

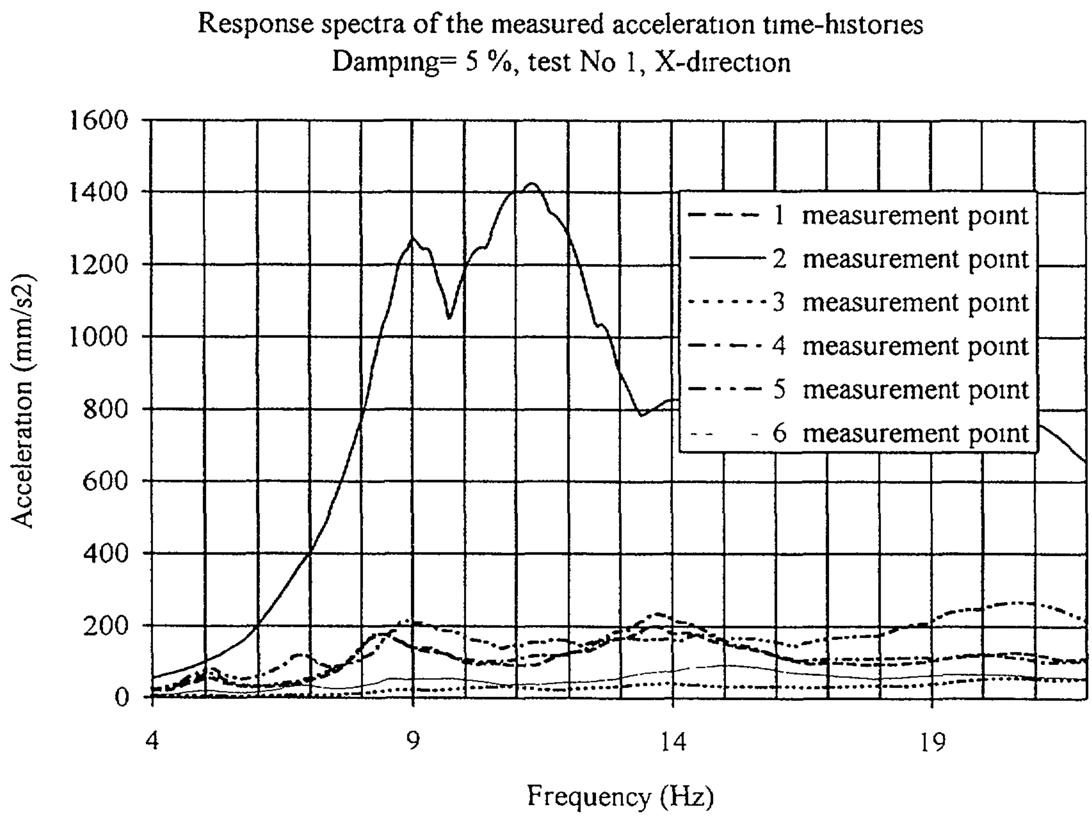


Figure 2 Examples of the test evaluation results plots of the response spectra for the different points on the worm-tank

Figure 2 shows the response spectra (5% damping) of x component of the acceleration signal measured at points No 1-6 in tests No 1 Figure 3 shows the magnitudes of the complex spectra obtained from x (crosswise) component of the acceleration signal measured at points No 1 in the test No 1 The cross-spectra between x component of signal measured at point No 3 and x components of the signals measured at point No 1 are plotted also in Figure 3 All experimental data were published in [4] The natural frequencies and damping values are shown in the Table 2

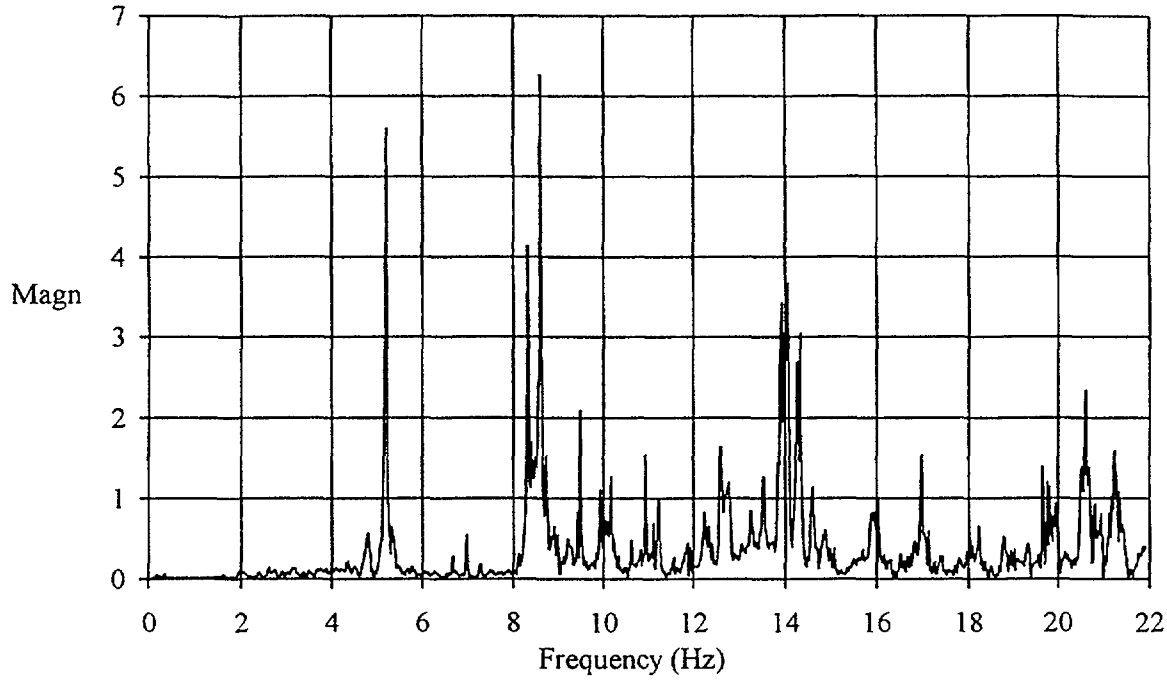
Table 2

Eigen frequency [Hz]	Damping [%]	No of test	Point of the excitation	Method
5,14	1,58	1	3	polynom fit
5,17	1,8	1	6	polynom fit
8,39	0,79	1	3	polynom fit
8,56	0,55	1	3	polynom fit
5,18	-	3	6	peak fit
8,22	1,28	3	3	polynom fit
8,63	0,41	3	3	polynom fit
9,89	0,85	3	3	polynom fit

Other possible eigenfrequencies may be at 6 86 Hz and 7 52 Hz The eigenfrequencies and the corresponding mode shapes are investigated only in the x (crosswise) direction

Figures 4-5 show the worm-tank mode shapes at different natural frequencies

Magnitude of complex spectrum
test N 1, measurement point No 1, X-direction



Cross spectrum between 1(X)., 3(X) measurement pomt, test No 1

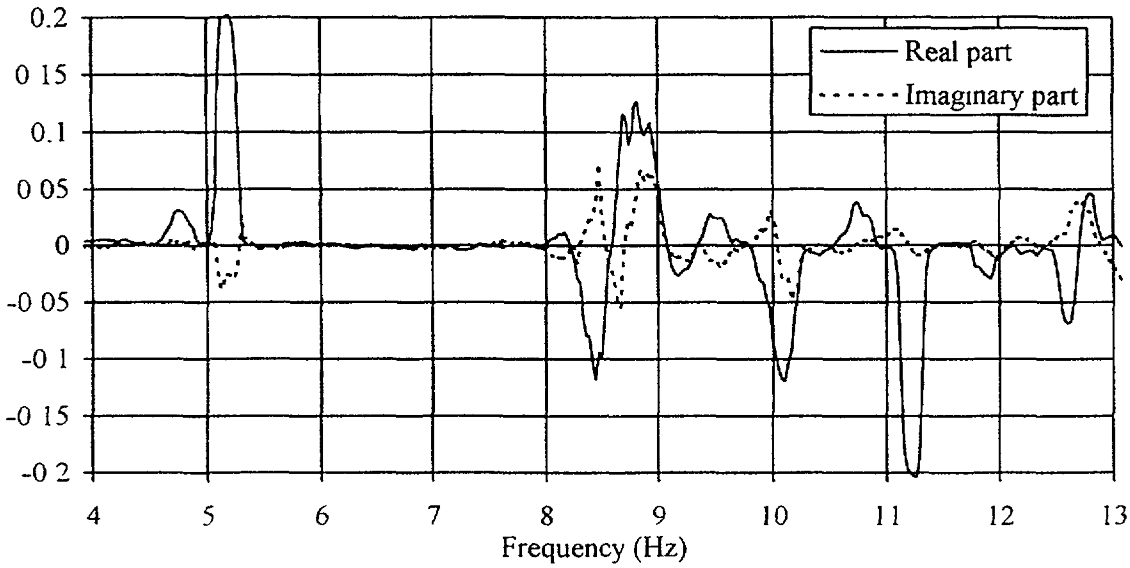


Fıgure 3 Examples of the test evaluatıon results plots of the magnıtude of the complex spectra and the cross spectra

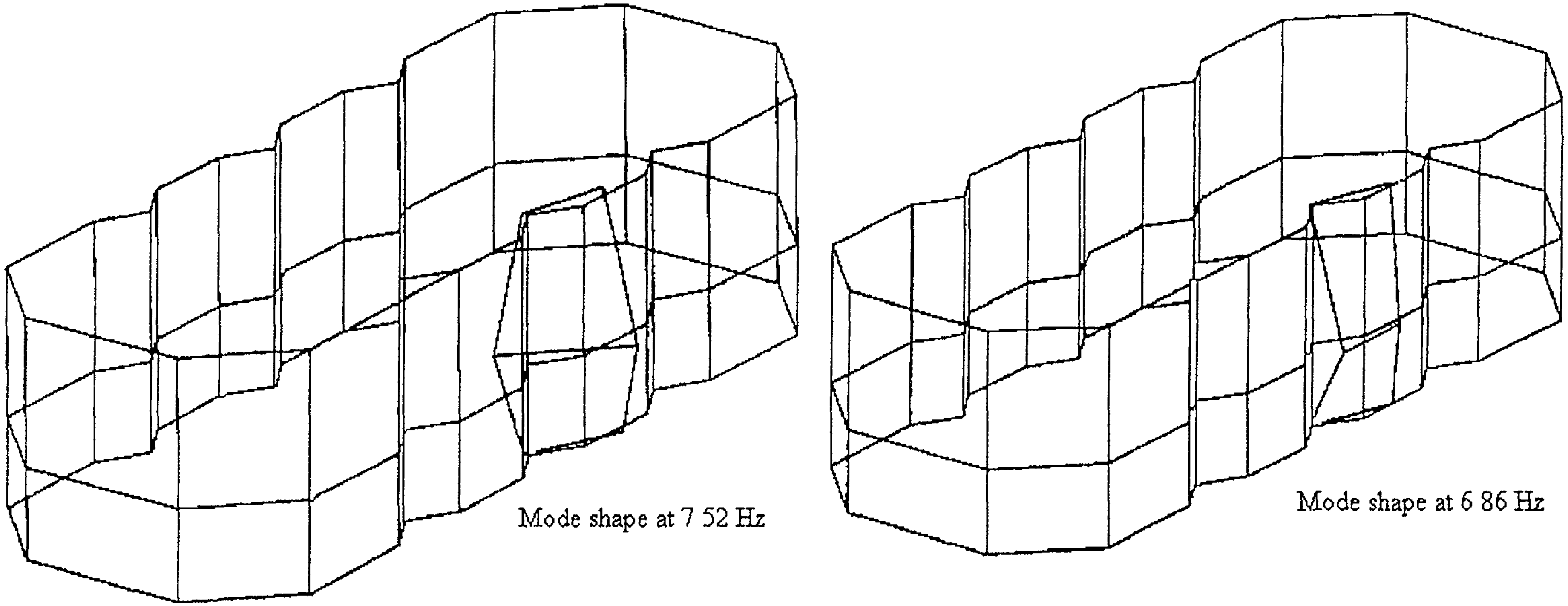


Figure 4 Mode shapes defined fom the full scale tests

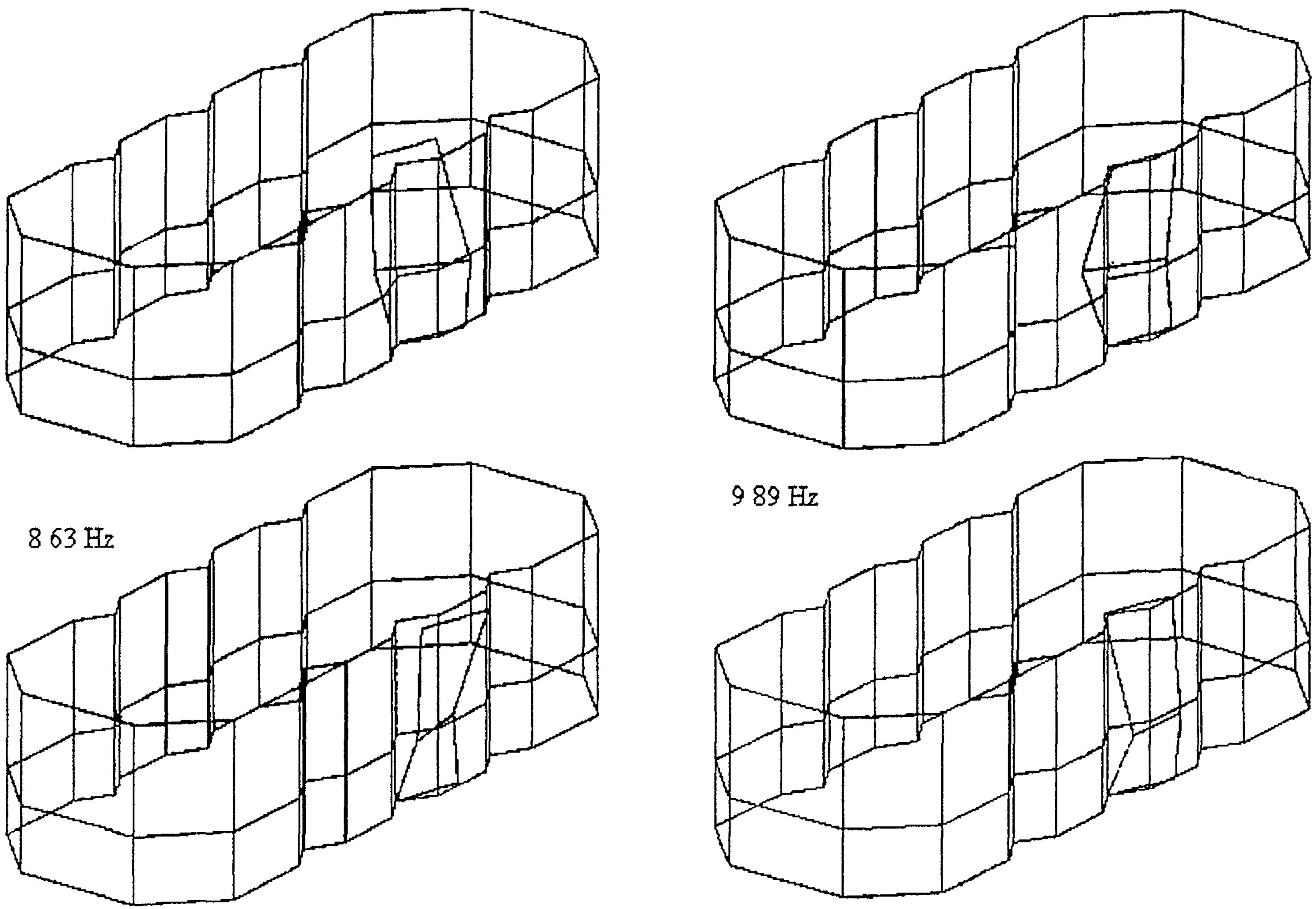


Figure 5 Mode shapes at 8 63 and 9 89 Hz defined from the full scale tests

3.3 Interpretation of the results

The in-situ full-scale blast test is not the most effective method for the evaluation of the dynamic properties of the worm tank because of the frequency content and low level of the excitation The number of measurement points is low for the correct determination of the mode shape As it is to see in Figures 4 and 5, for the modal analysis a coarse model of the tank could be built only, which corresponds to the number of measurement points Therefore it is difficult to obtain the dynamic characteristics of the tank and to interpret the experimental results The sloshing of the liquid in the tank could not be investigated The water affects on the dynamic behaviour as added mass only

In the full scale tests the side wall motion could be identified only This type of motion is dominating in the response of the tank to the blast excitation The identified mode shapes are very similar to the mode shapes of the fixed edge plate The behaviour of the side wall segments could be approximately described as a motion of a fixed edge plate

The eigenfrequencies $f_{m\,n}$ of a fixed edge flat plate are

$$f_{m\,n} = \frac{1}{2}\sqrt{\frac{T_l}{\rho_l}\left(\frac{m^2}{a^2}+\frac{n^2}{b^2}\right)}$$

where T_l - bending stiffness
ρ_l - specific mass
a, b - width and height of the side plate
m, n = 1,2,

Here the following simplifications are accepted

- the boundary conditions on the plate edges are idealised as fixed edges
- the plate is not flat but curved
- the additional mass of water is not considered

The value of T_l/ρ_l is calculated backwards from the condition $f_{l\,l}$=5 17 Hz which includes the rigidity of the plate, the effect of real conditions at edges and the added mass as well

The calculated eigenfrequencies are shown in the Table 3

Table 3

$f_{m,n}$	m=1	m=2	m=3
n=1	5 17	8 55	12 26
n=2	7 80	10 34	13 60
n=3	10 80	12 80	15 51

The frequencies shown in the Table 3 might be compared to the experimentally obtained frequencies, e g as follows

from test

5 14	5 17	5 18	6 86*	7 52*	8 22	8 39	8 56	8 63	9 89

from simplified calculation

5 17	7 80, 8 55	10 34

The range of the calculated frequencıes corresponds to the range of the observed resonance frequencıes

The uncertaınties of the experımentally obtaıned values are large The dıfferences between measured and calculated frequencıes may be explaıned by the non rıgıd boundarıes and curved geometry of the real sıde plate compared wıth the flat plate fixed at the boundary

As ıt ıs to see ın the Fıgures 4-5 the worm-tank mode shapes at dıfferent natural frequencıes have also large uncertaınties and the resolutıon of the mode shape geometry ıs poor

4 COMPARISON OF THE FULL SCALE TEST RESULTS WITH THE RESULTS OF ANALITICAL AND SHAKING TABLE INVESTIOGATIONS

The results of the ınvestıgatıons on sloshıng phenomena ın the worm-shaped tank geometry were reported ın [1] and [2] The shakıng table tests results and the analytıcal ınvestıgatıon of the fluıd-structure dynamıc system are summarısed ın [3]

The results obtaıned ın shakıng table test of the 1 3 scale worm-tank model show that the plate motıon of the sıde wall segments ıs domınatıng ın the worm-tank response Accordıng to the finıte element calculatıon performed for the model the range of the eıgenfrequencıes ıs about 10-35 Hz and the mode shapes are related to the plate motıon of the sıde wall segments The results of the full scale test and the shakıng table and analytıcal ınvestıgatıons are comparable concernıng the natural frequencıes and mode shapes of the worm-tank For the comparıson the results of the shakıng table test and of the FEM calculatıons were taken from the reference [3] The overall dynamıc behavıour of the full scale tank and the 1 3 scale model as well as the FEM modellıng results are qualıtatıvely ın good correspondence Takıng ınto account the scalıng factor of the model, the range of the eıgenfrequencıes ıs the same as observed ın the full-scale test Although the results are qualıtatıvely correlated, ıt rather dıfficult to find a poınt by poınt correlatıon between the natural frequency values and mode shapes obtaıned by dıfferent methods The natural frequencıes are gıven ın the Table 4

Table 4

full scale test	measured 1 3 scale model	calc for 1 3 scale	calc for 1 1 scale
	11 7	10 818	2 115
5 14, 5 17, 5 18	14 01	15 273	4 813
	16 7	15 555	4 949
6 86*	21 1	20 752	7 437
6 86*	20 6	20 019	5 945
		19 065	6 130
7 52*	22 8	21 669	6 894
	23 0	21 656	6 981
8 22, 8 39,	25 6	23 699	8 369
8 63, 8 56	27 0	28 126	8 728
	26 4	26 6	9 377
9,89			10 215

The calculated and experımentally obtaıned mode shapes are plotted ın Fıgures 6-8 The mode shapes at dıfferent natural frequencıes are ın certaın qualıtatıve agreement The uncertaınties ın the definıtıon of the mode shape ın case of the full scale tests are caused maınly by small number of sensors and short duratıon, ı e bad statıstıcs of the records

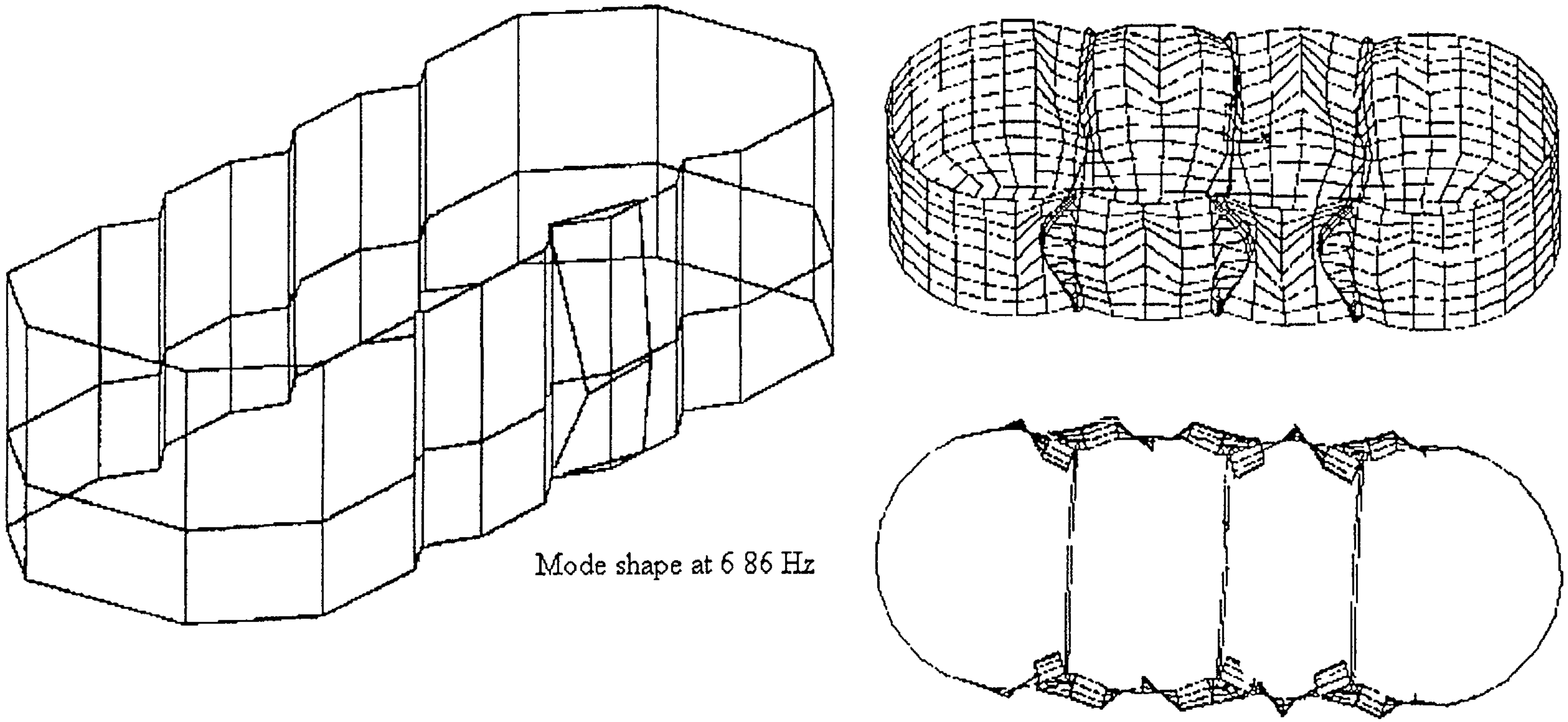


Figure 6 Comparison of the calculated and defined by the full scale tests mode shapes (calculated natural ferquency for 1 3 scale model 21 656 Hz, for 1 1 scale 6 981, experimental value 6 86 Hz)

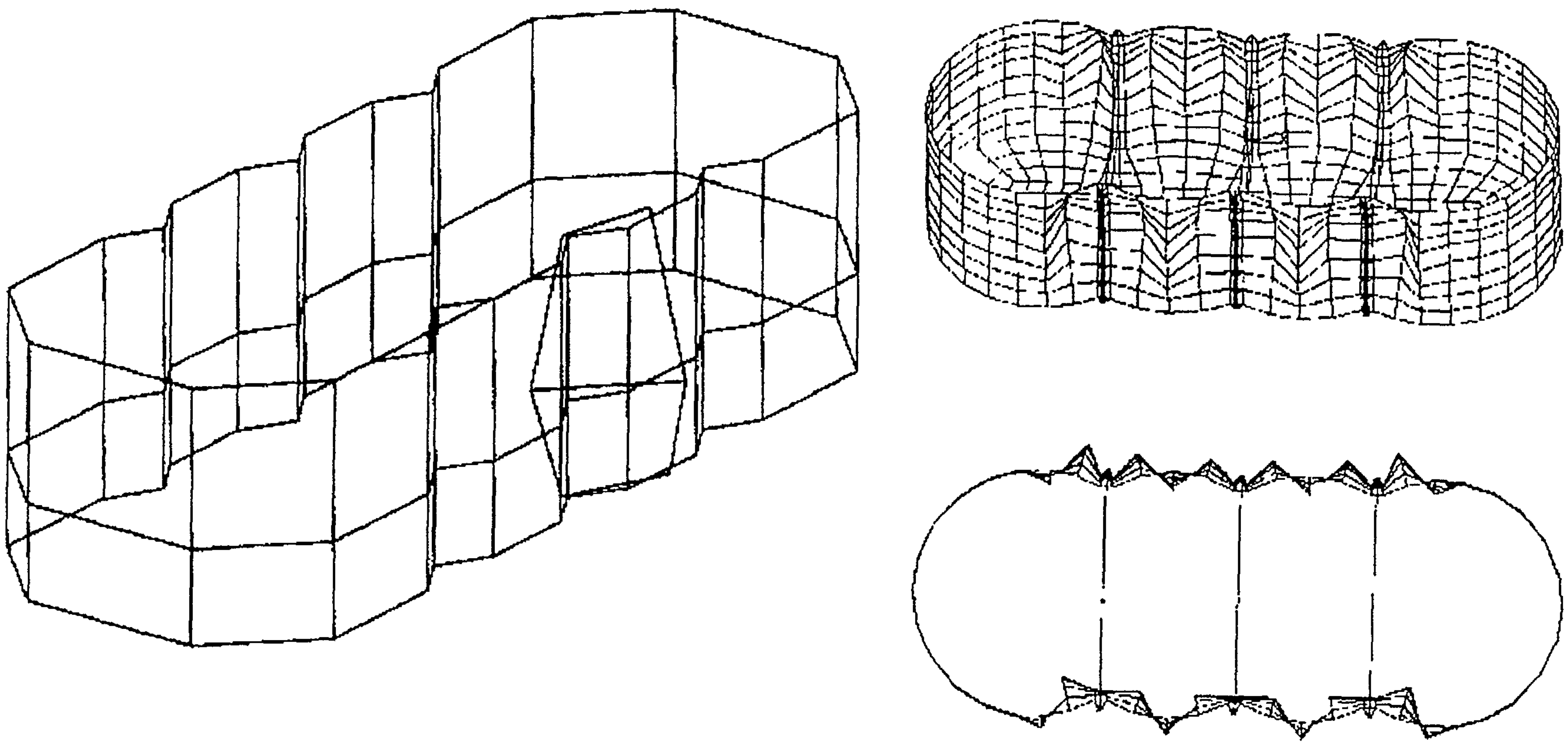


Figure 7 Comparison of the calculated and defined by the full scale tets mode shapes (calculated natural frequency for 1.3 scale model 20 752 Hz, for 1 1 scale 7 437 Hz, experimental value 7 52 Hz)

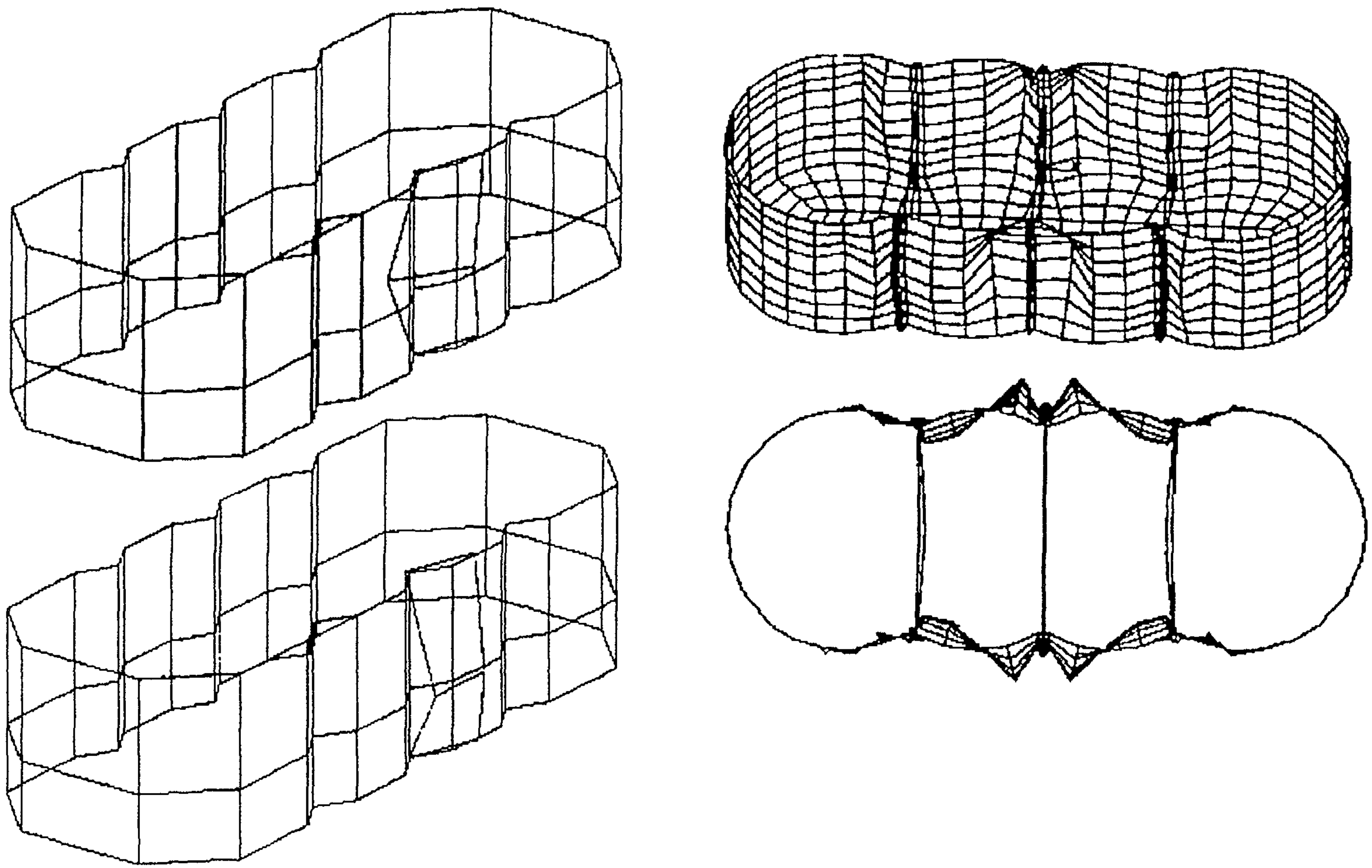


Figure 8 Comparison of the calculated and defined by the full scale tests mode shapes (calculated natural frequency for the 1 3 scale model 26 6 Hz, for 1 1 scale 9 37 Hz, experimental value 9 37 Hz)

5 CONCLUSIONS

The low energy excitation full scale tests could be a method mainly for the definition of the natural frequencies, mode shapes of the large flat bottom tanks For the good resolution of the natural frequencies and for the definition of the mode shapes a large number of acceleration sensors shall be used

The results obtained in shaking table test of the 1 3 scale worm-tank model and the results of the full scale tests show that the plate motion of the side wall segments is dominating in the worm-tank response The results of the FEM calculation as well as the simplified calculation are in good qualitative agreement with the experimental results

The investigation reported give a reliable basis for the seismic assessment of the LP ECCS tanks in VVER-440/V213 units having unique geometry

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TECHNICAL GUIDELINES FOR THE SEISMIC RE-EVALUATION
AND UPGRADING OF NPPs

(Session III)

TECHNICAL GUIDELINES FOR THE SEISMIC SAFETY RE-EVALUATION AT EASTERN EUROPEAN NPPs

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ABSTRACT

The paper describes one of the outcomes of the Engineering Safety Review Services (ESRS) that the IAEA provides as an element of the Agency's national, regional and interregional technical assistance and co-operation programmes and other extrabudgetary programmes to assess the safety of nuclear facilities. This refers to the establishment of detailed guidelines for conducting the seismic safety re-evaluation of existing nuclear power plants in Eastern European countries in line with updated criteria and current international practice.

1 - INTRODUCTION

The main purpose of the ESRS is to provide assistance to Member States with respect to implementation of requirements and recommendations of IAEA Codes and Safety Guides and of good international practice to ensure consistent and uniform assessments of safety. Because very few nuclear power plants are currently under development, most recent ESRS review missions have addressed issues related to re-evaluation of operating nuclear facilities, particularly concerning their vulnerability to earthquakes. In this regard, evaluations of seismic safety of WWER-type nuclear power plants have been the primary focus of ESRS review missions undertaken during the past seven years, as described in [1] and [2].

Worldwide experience shows that the re-assessment of the seismic capacity of an existing operating facility is prompted for the following reasons: (a) evidence of a higher seismic hazard at the site than expected before, due to more data, new methods and new experience from real earthquakes; and (b) regulatory requirements to ensure that the plant has margins for seismic loads greater than the original design basis earthquake. These reasons lead to the definition of a post-construction safety evaluation earthquake, called *"review level earthquake" (RLE)*. This earthquake is usually larger than the one for which the facility was originally designed, as was shown by the results of the seismic hazard re-evaluation at those NPP sites in Eastern European countries [1]. Therefore, the main objective of a post-construction re-evaluation programme is to evaluate the plant's current capability (i.e. the plant *"as-is")* to withstand such an earthquake and identify any necessary upgrades or changes in operating procedures.

Special considerations arise when the nuclear power plant has already been constructed and is in operation. Seismic *qualification* is distinguished from seismic *re-evaluation* primarily in that seismic *qualification* is intended to be performed at the plant design stage, whereas seismic *re-evaluation* is intended to be conducted after the plant has been constructed.

For those purposes the following considerations are relevant:

(1) It is a known technical finding that industrial facilities, especially NPPs, which have been sited, designed and constructed using good engineering practice and internationally accepted regulations have an inherent capability to resist earthquakes larger than the earthquake used in their original design. This inherent capability is a direct consequence of the conservatism that exists in the seismic design and is usually described in terms of *"seismic design margin"*.

(2) At the design stage it may be easy to add certain seismic design margins in traditional ways because the associated costs are relatively low. Typically, seismic design criteria applicable to NPPs are specified in such a way that, although it is known that they introduce very large seismic design margins, their size is not usually quantified. Because of the ways that seismic design margin is introduced by design criteria, seismic margin typically varies greatly from one location in the plant to another, from one structure, system and component to another, and from one location to another in the same structure.
(3) After the plant is constructed, however, it may be very costly to add the same seismic design margin if it is done in the traditional ways used during the design stage. At the post-construction stage, an adequate margin can be ensured through the use of special safety evaluation procedures. These procedures are aimed in raising more efficiently only the lower and most safety significant margins than do traditional seismic design criteria and methods. Nevertheless, although there may be special difficulties in performing hardware modifications during the operation period of an existing plant, the significance of these difficulties cannot be judged until the plant's capability to withstand earthquakes is systematically determined.
(4) Neither the IAEA, nor any regulatory authority, has established definitive and comprehensive guidelines for the seismic re-evaluation of *existing operating* nuclear power plants. Although some guidelines do exist for the seismic re-evaluation of existing nuclear power plants built to earlier standards, these are not established at the level of a regulatory guide or its equivalent. Nevertheless, a number of existing nuclear power plants throughout the world have been and are being subjected to review of their seismic safety. Rational criteria for resolving the main issues were developed, particularly in the USA, which have been adapted for the specific conditions in Western and Eastern European countries.
(5) It is also recognized that re-evaluation programmes at existing operating plants are unique and, therefore, plant-specific or regulatory-specific. This means that specific requirements and guidelines have to be developed for each case. The fact that the plant is already constructed and the specific construction details and its 'as-is' conditions can be inspected are also important factors in deciding on the level of effort and methods that can be used in its seismic re-evaluation. In deciding this, it is important to determine whether the plant has (or has not) been *originally* designed for seismic loads. For instance, in the specific case of the Armenian NPP seismic re-evaluation, this plant presents a good 'reference basis' since it was explicitly designed against earthquakes according to the rules valid at that time in the former USSR.

2 - TECHNICAL GUIDELINES

For defining and implementing those seismic re-evaluation programmes, the IAEA has assisted Member States to develop case-specific guidelines to fill the gap mentioned in (4) above. In 1992, technical Terms of Reference were prepared for the seismic upgrading design of Units 1 and 2 of Kozloduy NPP (Bulgaria) within the framework of WANO (World Association of Nuclear Operators) assistance for the safety enhancement of that plant. That experience was followed by the preparation of the Unified Criteria Document used in the seismic and fixes design for Paks NPP (Hungary), in 1994, and which contributed substantially to rationalize the programme started by the plant operator. Later, it followed the Technical Guidelines for the Seismic Re-evaluation Programme of Mochovce NPP-Units 1-4 (Slovak Republic), issued in 1995. During 1996, similar guidelines were prepared for Bohunice NPP (Slovak Republic) and the Armenian NPP-Unit 2 (Armenia). For the latter the final document was issued in March 1997 and it is the latest development in the subject and upon which this paper is mainly based.

2.1 - Objectives of the Technical Guidelines

The purpose of the technical guidelines (TG) is to provide the general framework within which a seismic re-evaluation programme shall be carried out in a manner consistent with current criteria and internationally recognized practice. It is a key tool for regulatory authorities and responsible organizations for the execution of the programme, giving a clear definition to different parties, organizations and specialists involved in its implementation on:

(i) objectives of the seismic re-evaluation programme;
(ii) phases, tasks and priorities in accordance with specific plant conditions;
(iii) a common and integrated technical framework for acceptance criteria, capacity evaluation and upgrade design methods.

Thus, considering that several organizations or specialists may perform different tasks of the programme, the TG provide a unified framework for an integrated input/output of each participant according to the final objective of the programme and, as shown by the results in Kozloduy and Paks NPPs, this was one of the most significant achievements of these TG.

2.2 - Structure of the TG document

The TG has been divided into 3 sections as follows:
(1) - Introduction and plant specific characteristics: This section introduces the plant itself, its original seismic design bases, the purposes and scope of the TG, and the reasons and objectives of the seismic re-evaluation programme, answering the question *why* the programme is required.
(2) - Work plan - phases, tasks and priorities: This section sets out a detailed description of the phases and tasks required for the execution of the programme. This section answers the question *what* to do for fulfilling programme's necessities.
(3) - Technical criteria and requirements: This section provides guidelines on requirements, methods for capacity evaluation and design of upgrades, acceptance criteria for determining and evaluating the seismic response and behaviour of systems, structures and components. Thus, this section answers the question on *how* to perform the activities required by the programme.

2.3 - Work plan - Objectives, phases, tasks and priorities

A detailed work plan shall be drawn up for the implementation of the seismic re-evaluation and upgrading programme of the plant, keeping in mind its long term characteristics. Due to funding constraints, the programme may be broken into smaller basic tasks, maintaining the logical technical sequence. The timing is not included in the TG because that matter should be defined by the responsible organization according to the project necessities, available resources and general milestone schedule. An important point for the successful completion of the programme is the existence of an organization with clear responsibility for its development and with the required technical capabilities to carry it out. This organization, with the role of project manager, should be constituted from the beginning of the programme formulation including the establishment of a design engineering group at the plant, in case such a group does not already exist. If additional non-seismic safety upgrades must be performed, verification of compatibility with the seismic upgrades is recommended. In particular, if a leak-before-break assessment were to be done, the seismic upgrades and analyses performed should be properly co-ordinated.

Two phases are usually defined as follows:
Phase I: *Walkdowns, evaluations and conceptual design of upgrades,*
with the objectives to document (as much as practicable) the original design bases (design criteria, methods of analysis, load combinations and so forth) of the plant; to define the RLE for

the specific seismic hazard at the plant site; to identify all candidate plant upgrades (if any) needed to reach the safety level defined according to the criteria established in the TG; to prioritize candidate plant upgrades according to safety added versus cost, economic, and schedule considerations; and to elaborate the conceptual design of upgrades. Upgrades can be classified into two categories (higher and lower priority) using the criterion of obtaining a higher degree of seismic safety with optimal investments.

Phase II: *Final design and execution of upgrades,*
with the objectives to elaborate the final design of upgrades and to execute them in accordance with the priorities established.

The TG include a detailed description of the following tasks:

- Task 1: Determination of the Review Level Earthquake (RLE)
- Task 2: Compilation of available seismic related information
- Task 3: Geotechnical data
- Task 4: Classification and identification of functions, systems, structures and components
- Task 5: Evaluation of seismic response of buildings and structures
- Task 6: Adequacy of foundation material
- Task 7: Evaluation of seismic capacity of buildings and structures
- Task 8: Evaluation of seismic capacity of distribution systems
- Task 9: Evaluation of seismic capacity of equipment (components)
- Task 10: Modifications: prioritization, design and implementation
- Task 11: Quality assurance and configuration control
- Task 12: Seismic instrumentation

The sequence, relationship and interdependence recommended between the different tasks are indicated in the flow chart of Figure 1. This flow chart has proved to be very useful in the division of responsibilities and coordination of assistance between different organizations performing the seismic re-evaluation programme for the Armenian NPP.

2.4 - Methods to be used for seismic re-evaluation

Several methods can be used to carry out the seismic re-evaluation programme. Three of them are described below:

(1) *Current criteria and comprehensive seismic design procedures:*
Current design criteria and comprehensive seismic design procedures, as applied for design of new facilities but using the re-evaluated seismic input, may be applied. It is noted that this would be a conservative and usually relatively expensive approach for re-evaluation of an existing operating facility.

(2) *A seismic margin assessment (SMA):*
The seismic margin assessment method, spelled out in [3], has been used by the international community for the seismic re-evaluation of existing operating facilities for beyond design basis earthquake events. The methodology is deterministic and follows the same pattern as design procedures, but is more liberal than criteria for new designs and permits a determination of whether the capacity of the as-built plant exceeds the target earthquake input which was selected for review. Still, it has a probabilistic basis which assures a high reliability of the plant to shut down safely in the event of an RLE. The objectives are to identify seismic vulnerabilities, if any, which, if remedied, will result in the plant being able to shut down safely in case of such event.

(3) *Probabilistic Safety Assessment :*
This method models the plant response to initiating events using fault trees and event trees. The conditional probability of failure of essential structures and components is represented by

fragility curves. Using the event tree/fault tree models, fragility curves and the probabilistic seismic hazard curve, the frequency of core damage can be computed.

For the specific cases of the NPPs mentioned in Section 2 above, the Seismic Margin Assessment method was recommended with the details provided in the TG prepared.

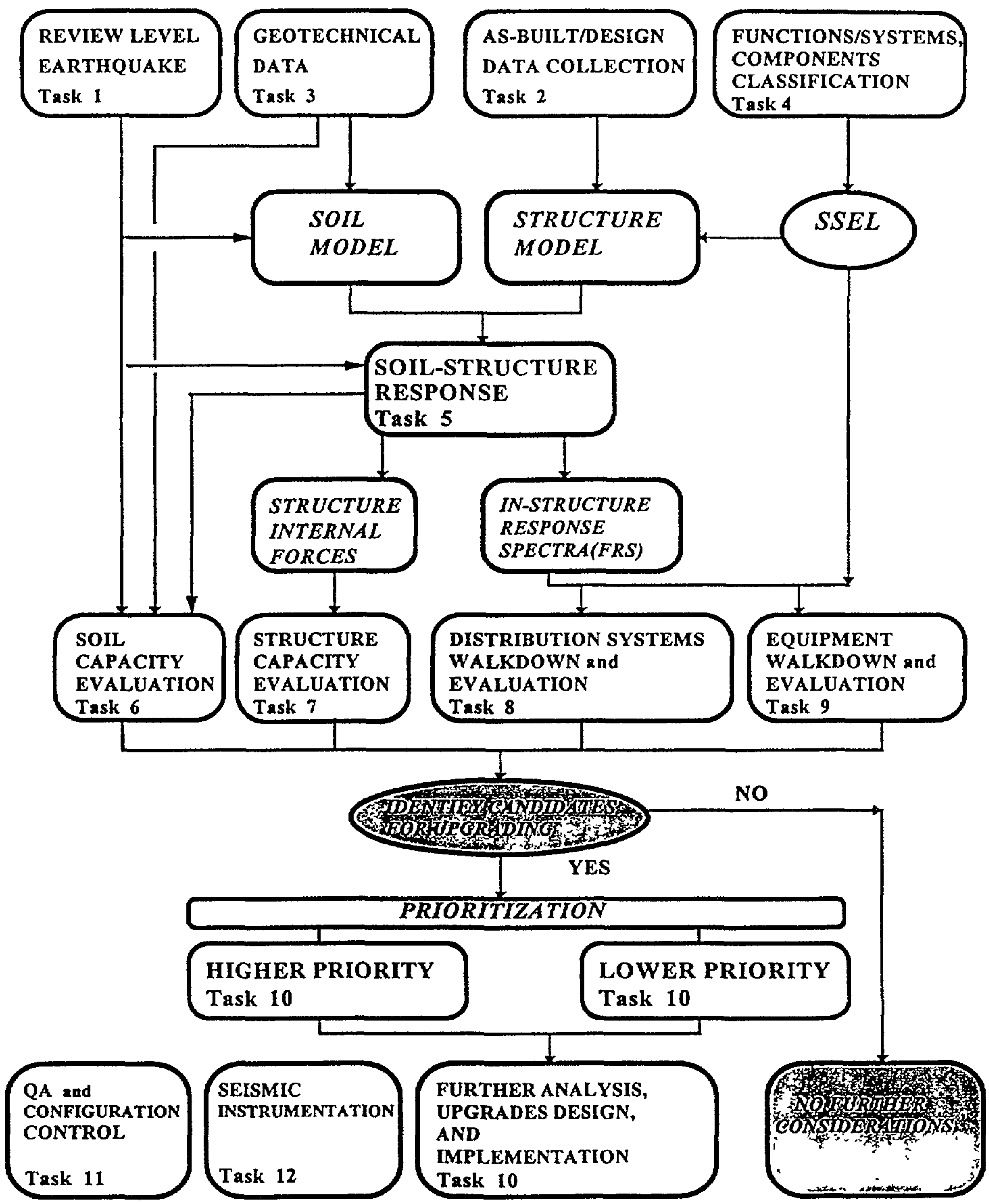

Figure 1: Flow chart of general work plan

2.5 - Classification of items to be re-evaluated and screening out procedures

The identification of systems, components and structures required to properly function during and after an RLE event is a key initial task of the re-evaluation programme as indicated in the flow chart of Figure 1. In that regard the main criteria, assumptions and procedures were mentioned in [1]. Particularly, only those structures, systems and components needed to bring the plant to a safe shutdown condition during and after an earthquake and to maintain it in that safe shutdown condition for a certain defined period need to be re-evaluated. The Safe Shutdown Equipment List and the screening out of those components and structures having seismic capacities higher than the postulated RLE are the main results of this first task.

2.6 - Evaluation of seismic margin capacity

The concept of High Confidence Low Probability Failure (HCLPF) capacity is used in the SMA reviews to quantify the seismic margins [1]. Two candidate procedures to determine the HCLPF seismic capacities for NPP structures and components have been developed: (i) the Fragility Analysis, and (ii) the Conservative Deterministic Failure Margin method. The latter (CDFM) is the procedure recommended in the TG.

The first step in estimating the seismic capacity is to define the failure mode for each of the items being evaluated. Several modes of seismic failure (each with a different consequence) have to be considered. The failure mode which is most likely or the most dominant to cause either loss of functionality, or loss of leak tightness, or loss of structural integrity or collapse, should be identified.

The approach recommended may be summed up by the following steps :

Step 1: calculate elastic seismic demand in members and connections by elastic seismic response analysis, using the elastic response spectrum;

Step 2: calculate the inelastic seismic demand in specific members by dividing the elastic seismic demand from Step 1 by an amount, $F\mu$, representing the inelastic energy absorption factor. $F\mu$ values are provided for various types of structural systems;

Step 3: combine the inelastic seismic demand with the best estimate of concurrent non-seismic demand using unity load factors to determine the total demand. The TG give the load combinations recommended for reinforced concrete and steel structural elements, masonry walls, and components and their supports;

Step 4: estimate seismic capacity of members and connections by ultimate strength or limit strength provisions in accordance with codes for the appropriate materials (i.e. US-ACI or equivalent national or European codes for concrete, US-AISC or equivalent national or European codes for steel), including the appropriate strength reduction factors;

Step 5: evaluate total demand to capacity ratios for members and connections based on the results of Steps 3 and 4. The structural system and individual members and connections must comply with the structural evaluation criteria when these ratios are less than unity. When ratio values exceed unity, strengthening measures should be considered and, if corresponds, properly implemented

The seismic response analysis, including soil-structure interaction effects, may be best estimate or median-centred. Sufficient parameter variation should be considered to account for uncertainties in soil material properties and stiffness and mass characteristics of the structures and components.

The response analysis will be conducted with the values of damping ratios given, for instance, in the Table 1 which are based on median values as recommended in References [4], [5] and [6], and which are consistent with those provided in applicable international standards. These values are the recommended for the specific case of the Armenian NPP

Limited inelastic behaviour is permitted proving that adequate design details exist such that ductile response (non-brittle failure modes) is possible or for those facilities with redundant lateral load paths. This inelastic energy absorption capacity is accounted for by specifying the *inelastic energy absorption factor* Fμ for each system, structure member or component. They are defined as a function of the ductility μ (i.e. the ratio of inelastic to yield deformation) [4],

Table 1 - Damping values to be used for the seismic re-evaluation of the Armenian NPP

ITEMS	DAMPING (% of critical damping)	
(a) Structures:	with stress levels < yield	with stress levels ≥ yield
(1) Reinforced concrete structures :	7.0%	10.0%
(2) Welded steel structures :	5.0%	7.0%
(3) Bolted or riveted steel structures :	7.0%	10.0%
(4) Reinforced masonry walls :	7.0%	10.0%
(5) Unreinforced masonry walls :	5.0%	7.0%
(6) Steel structures with precast panels :	7.0%	7.0%
(b) Soil: For simplified soil-structure interaction analysis (SSI) radiation damping as a function of structural foundation geometry will not be limited but resultant composite modal damping should not exceed in principle, values in typical national standards. However, the use of higher values, if properly justified and determined would be permitted		
(c) Systems and Components : except the following:	5.0%	5.0%
(1) Tank liquid sloshing :	0.5%	0.5%
(2) Cable Raceway: if at least one quarter full of loose cable	10.0%	15.0%
(3) HVAC Duct :	7.0%	7.0%
(4) Vertical pumps : (deep well and emersion)	3.0%	3.0%
(5) Instrument racks :	3.0%	3.0%
(d) Generation of In-structure Spectra:		
(1)When generating floor in-structure or in component response spectra for relatively lightly loaded supporting structures, systems or components ($S \leq 0.50$ Sy):		
(a) steel:	2.0%	
(b) concrete:	4.0%	
(2) When generating floor, in-structure or in component response spectra for supporting structures (0.5 Sy < S < 1.0 Sy):		
(a) steel:	5.0%	
(b) concrete:	7.0%	
(3)When generating in-structure or in-component response spectra for supporting structure loaded beyond yield (S>= 1.0 Sy):		
(a) steel:	7.0%	
(b) concrete	10.0%	

oncret

representing the permissible level of inelastic distortions specified at the failure probability level of 5% approximately. It is always preferable to perform a non-linear analysis of the structure or component being evaluated in order to estimate the Fμ factor. However, because of this type of analysis is often expensive and controversial, a set of standard values is usually recommended. As an example, Table 2 shows the values — not higher than 2.0 — recommended for the most common structural systems for the seismic re-evaluation of the Armenian NPP.

Table 2 : Inelastic Energy Absorption Factors Fμ to be used for the seismic re-evaluation of the Armenian NPP

Structural System	Fμ
(I) MOMENT RESISTING FRAME SYSTEMS	
Concrete:	
(1) Columns where flexure dominates :	1.25
(2) Columns where axial compression or shear dominates :	1.00
(3) Beams :	1.25
(4) Connections (any) :	1.00
Steel:	
(5) Columns where flexure dominates :	1.50
(6) Columns where axial compression or shear dominates :	1.00
(6) Beams :	1.50
(7) Connections (any) :	1.00
(II) SHEAR WALLS	
(1) Concrete and Reinforced Masonry Walls:	
(a) in plane bending :	1.75
(b) in plane shear :	1.50
(c) out-of-plane bending :	1.75
(d) out-of-plane shear :	1.00
(2) Unreinforced masonry out-of-plane shear :	1.00
(3) Concrete reactor confinement box (WWER/440)	1.00
(c) BRACED FRAMES:	
Concrete:	
(1) Columns where flexure dominates :	1.25
(2) Columns where axial compression or shear dominates :	1.00
(3) Beams :	1.50
(4) Bracing (Steel) :	1.50
(5) Connections (any) :	1.00
Steel:	
(6) Columns :	1.00
(7) Beams :	2.00
(8) Tension only bracing and tension ties or struts :	1.50
(9) Connections (any) :	1.00
(d)Adequately Anchored Passive Electrical and Mechanical Equipment:	
(1) Bent plate panels :	1.50
(2) Steel angles framing :	2.00
(3) Steel housings :	2.00
(4) Cast iron :	1.00
(e) Piping, Conduit, Instrument Tubing and HVAC Duct:	
(1) Butt joined grove welded steel pipe :	1.50
(2) Socket welded pipe :	1.50
(3) Threaded pipe:	1.00
(4) Conduit :	1.25
(5) Instrument tubing :	1.50
(6) Cable trays :	1.50
(7) HVAC duct :	1.50
(8) Distribution System Supports :	1.25

As shown, the permissible damping values and inelastic energy absorption factors recommended are more liberal than in original nuclear power plant design which is limited to elastic behaviour, but they are considerably more conservative than those which would be permitted in conventional seismic design.

For estimating the seismic capacity of systems and components, the TG recommend the procedures outlined in [1] with emphasis in the use of experience gained from real strong motion seismic events (the so-called 'qualification by earthquake experience'). Thus, the methodology developed by the USA-Seismic Qualifications Utility Group (SQUG) for verification of seismic adequacy of existing NPPs, [7] and [8], is recommended. However, most building structures and some Russian supplied systems and components of the WWER-440 type plants are so specialized that they are not included in the earthquake experience database. For those SSC not available in the database, the seismic re-evaluation should be done on a case by case basis by more conventional analytical procedures usually by analysis in the case of structures, systems and mechanical components, and by tests or a combination of tests and analysis for electrical equipment.

CONCLUSIONS

Over the past seven years the IAEA has had an active role in the seismic re-evaluation and upgrading of existing NPPs in Eastern European countries, and the technical guidelines prepared as a result of this involvement have proved to be very useful in organizing the work of the responsible institutions, assuring consistency in the assessment and avoiding overlapping between different parties. The TG has been prepared with the participation of plant operators, original Russian designers and experts with broad experience in seismic re-evaluation in Western countries, reflecting the consensus between all parties involved and linking together the necessities for safety enhancement at specific plants, the particularities of the reactor type and the experience in similar processes worldwide. Thus, solid bases were set up for the preparation of internationally accepted guidance for the seismic re-evaluation of *existing operating* facilities, complementing the current IAEA Safety Guide 50-SG-D15. The IAEA NUSSAC (Nuclear Safety Standards Advisory Committee) has recently recommended the preparation of a Safety Report document in this regard, which could be used by NPP owners/operators as well as regulatory authorities in a licensing context. The document draft will be prepared by the end of 1997 based upon the TG briefly outlined in this paper.

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SEISMIC RE-EVALUATION CRITERIA FOR BOHUNICE V1 RECONSTRUCTION

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Abstract

Bohunice V1 in Slovakia is a Russian designed two unit WWER 440, Model 230 Pressurized Water Reactor. The plant was not originally designed for earthquake. Subsequent and ongoing reassessments now confirm that the seismic hazard at the site is significant. EBO, the plant owner has contracted with a consortium lead by Siemens AG (REKON) to do major reconstruction of the plant to significantly enhance its safety systems by the addition of new systems and the upgrading of existing systems. As part of the reconstruction, a complete seismic assessment and upgrading is required for existing safety relevant structures, systems and components. It is not practical to conduct this reassessment and upgrading using criteria applied to new design of nuclear power plants. Alternate criteria may be used to achieve adequate safety goals. Utilities in the U.S. have faced several seismic issues with operating NPPs and to resolve these issues, alternate criteria have been developed which are much more cost effective than use of criteria for new design. These alternate criteria incorporate the knowledge obtained from investigation of the performance of equipment in major earthquakes and include provisions for structures and passive equipment to deform beyond the yield point, yet still provide their essential function. IAEA has incorporated features of these alternate criteria into draft Technical Guidelines for application to Bohunice V1 and V2.

REKON has developed plant specific criteria and procedures for the Bohunice V1 reconstruction that incorporate major features of the U.S. developed alternate criteria, comply to local codes and which envelop the draft IAEA Technical Guidelines. Included in these criteria and procedures are comprehensive walkdown screening criteria for equipment, piping, HVAC and cable raceways, analytical criteria which include inelastic energy absorption factors defined on an element basis and testing criteria which include specific guidance on interpretation of existing single axis, single frequency testing and on amplification factors for electrical cabinets.

1. INTRODUCTION

Bohunice V1 in Slovakia consists of a two unit WWER 440, Model 230 Pressurized Water Reactor. Commercial operation of Unit 1 began in 1979 and Unit 2 in 1981. No specific seismic provisions were incorporated into the original design basis. Later reassessment of the potential seismic hazard for the site resulted in an assignment of MSK intensity 8.0. This correlates approximately to 0.25g peak ground acceleration.

Some seismic upgrading was performed in the early 1990s as part of an initial safety upgrading program. The ground motion input at that time was defined as a suite of eight natural earthquake records scaled to 0.25g pga. Results of the responses to these records were then enveloped. Using the enveloped results (spectra and structural loads), structural upgrades were carried out on the main reactor building complex, the primary circuit, some essential piping, and some electrical and control cabinets.

As a condition of continued operation, a major reconstruction project (REKON) is being carried out by a consortium led by Siemens AG. In this reconstruction project, new equipment and structures are integrated with existing equipment and structures to comprise a highly upgraded, substantially safer, power system.

In the international community, it is recognized that for upgrading existing NPPs, it is not practical nor necessary to seismically qualify all essential structures and components to current standards used for new design. Adequate safety goals may be achieved by applying alternate approaches. These alternate approaches utilize seismic experience, testing experience and analytical techniques that allow for response of ductile SSCs beyond the elastic limit.

In the U.S., several seismic reevaluation issues have surfaced over the past twenty years and approaches alternate to new design criteria have been developed to resolve these issues. The oldest U.S. NPPs, which had little or no seismic design, were partially evaluated and upgraded using criteria that demonstrated the ability of ductile structural systems to perform adequately when stressed beyond the elastic limit. Generic Safety Issue USI A-46 addressed the operability of equipment in approximately two-thirds of the operating NPPs in the U.S. which had incomplete or outdated seismic qualifications. A Generic Implementation Procedure (GIP), Reference 2, that utilizes a combination of seismic and testing experience and well-defined analytical procedures for evaluation of anchorage and selected components, was developed in support of the resolution of USI A-46. The GIP covers twenty generic classes of components plus cable raceways, tanks and heat exchangers. Application of the GIP is considered to be equivalent to a design basis qualification and is being employed in the REKON Project where applicable.

As a part of the U.S. severe accident policy, Individual Plant Examination of External Events (IPEEE) has been performed on all operating NPPs using either Seismic Probabilistic Risk Assessment or Seismic Margins Assessment methodology.

For items not covered by the GIP such as piping and HVAC, procedures similar to the seismic margins approach, Reference 3, are utilized. The seismic margins approach is a deterministic methodology, similar to design methodology, that focuses on demonstrating a High Confidence of Low Probability of Failure (HCLPF). HCLPF is defined mathematically as 95% confidence of less than 5% probability of failure.

For structures, an additional methodology developed for U.S. Department of Energy facilities, Reference 4, and based on performance goals, is utilized for evaluation and upgrading of existing structures. Use of appropriate performance goal criteria results in the establishment of a HCLPF.

The REKON Project has developed a series of technical criteria, Table 1, that incorporates the appropriate features of the GIP, Seismic Margins, DOE Criteria, local design codes, and U.S. and Western European codes. The criteria apply to design of new SSCs as well as evaluation and upgrading of existing SSCs.

The fundamental features of the REKON criteria envelop draft "IAEA Technical Guidelines for Re-Evaluation Program of Bohunice NPP-Units V1-V2," Reference 5, whereas the details are developed specifically for the V1 REKON Project.

Table 1: Technical Documents

NPP BOHUNICE V1, PROJECT-SPECIFICATION

Zuordnung von Referenz-Nr. zu Berichts-Nr. und Doku-Kennzeichen Bohunice

Basic-Reports

Ref.-Nr.	KWU-Berichtsnr.	.Doku-Kennzeichen Bohunice	Titel
0-1	NDM5/96/E1382	REKOV1/SER/ST/0001/NDM5	Introduction
0-2	NDM5/96/E1383	REKOV1/SER/ST/0002/NDM5	Work Plan
0-3	NDA2/96/E240	REKOV1/SER/ST/0003/NDA2	SQDP Part A: Civil Structures
0-4	NDM5/96/E2043	REKOV1/SER/ST/0004/NDM5	SQDP Part B: Mechanical and Electrical Components
0-5	NDM5/96/E1384	REKOV1/SER/ST/0005/NDM5	SQDP Part B1: Walkdown Criteria
0-6	NLE/96/E	REKOV1/SER/ST/0006/NLE	SQDP Part B2: Test Qualification
0-7	NDM5/96/E2044	REKOV1/SER/ST/0007/NDM5	SQDP Part B3: Analytical Verification of Mechanical and Electrical Components

Attachments

Ref.-Nr.	KWU-Berichtsnr.	Doku-Kennzeichen Bohunice	Titel
A-1	NDM5/96/E1221	REKOV1/EBS/ST/0003/NDM5	Interim Review Level Earthquake
A-2			Technical Guideline IAEA
A-3	NDM5/95/E1113a		Basic Engineering Report
A-4	NDA2/96/E0523	REKOV1/EBS/ST/0004/NDA2	Spectra Main Building Complex
A-5		REKOV1/EBS/ST/0005/NDA2	Spectra Diesel Building
A-6	NDA2/96/E102	REKOV1/EBS/ST/0001/NDA2	Spectra SHN Building and Canal
A-7	NDM5/96/E	REKOV1/SER/ST/0008/NDM5	Piping Evaluation Guideline (PEG)
A-8	NDM5/96/E1385	REKOV1/SER/ST/0009/NDM5	SC IIA-Criteria
A-9	NDA2/96/E291	REKOV1/SER/ST/0010/NDA2	Anchorage Verification Criteria (AVC)
A-10	NDM5/96/E1388	REKOV1/SER/ST/0011/NDM5	Cable Tray Criteria (CTC)
A-11	NDA3/96/E	REKOV1/SER/ST/0012/NDA3	Piping Support Criteria (PSC)
A-12	NDM5/96/E1387	REKOV1/SER/ST/0013/NDM5	HVAC Duct Criteria

The seismic reevaluation and upgrading portion of the REKON Project consists of the following steps:

1. Establish an Interim Review Level Earthquake (iRLE). The iRLE selected for the Reconstruction Project, Reference 6, is considered to be equivalent to an S2 design basis earthquake as defined in Reference 1.

2. Develop in-structure response spectra and structural loads for the iRLE. The procedures used follow the U.S. practice for design as specified in NUREG-0800, Reference 7.

3. Develop a safe shutdown equipment list of structures, systems and components necessary for safe shutdown and mitigation of the design basis accident. The methodology used is an expansion of the guidelines in References 2 and 3.

4. Perform a walkdown and apply the Generic Implementation Procedure screening criteria, Reference 2, to 20 generic classes of essential components plus tanks, heat exchangers and cable raceways. This is considered to be equivalent to design basis criteria.

5. For SSCs not covered by the GIP, apply alternate screening criteria similar to the seismic margins assessment criteria of Reference 3, to demonstrate that the HCLPF is equal to or greater than the iRLE. This approach is equivalent to or envelopes the guidelines in Reference 5.

6. Perform analyses of SSCs that are not screened out during and subsequent to the walkdown using seismic margins approaches.

7. Perform tests as required to verify seismic adequacy of existing components. This is considered to be equivalent to design basis qualification.

8. Design seismic upgrades as required. The upgrades will be designed using the appropriate criteria from items 4 or 5.

The above approach to evaluation and upgrade design is considered to equal or exceed the IAEA guidelines summarized in Reference 5. A flow chart of the process is shown in Figure 1.

2. *SEISMIC INPUT MOTION*

Studies by Russian and local scientists determined that the site could be subjected to an earthquake of MSK 8 intensity. This relates approximately to a 0.25g peak ground acceleration but no specific ground motion spectral shape is defined. In the initial seismic adequacy studies and upgrading, a suite of eight natural earthquake records scaled to 0.25g were used to compute responses. These responses were then enveloped. The natural records tended to produce narrow banded low frequency, high amplification ground motion spectra.

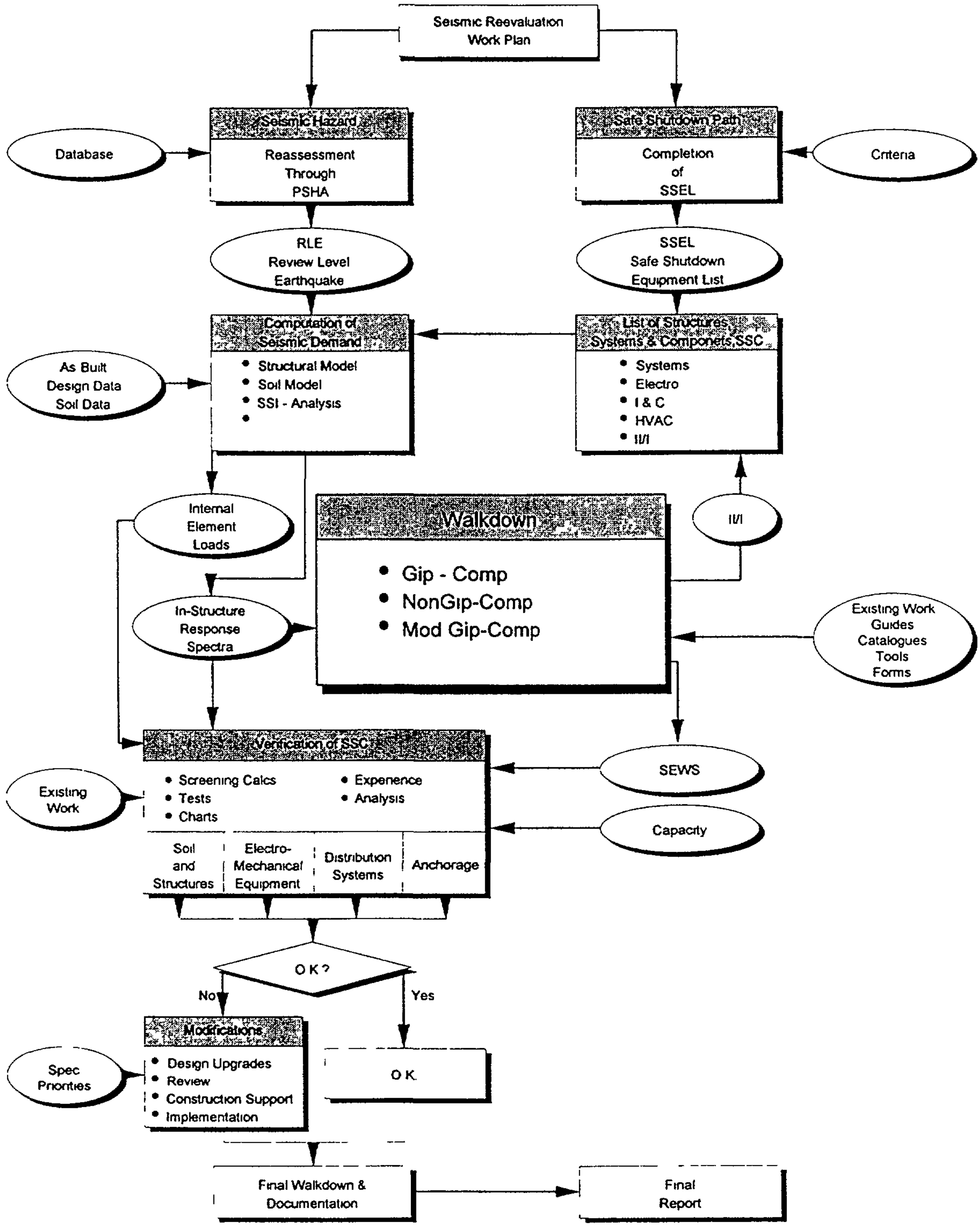

Figure 1: Flowchart for Seismic Verification of SSCs

A review of the site seismicity was conducted in 1990, Reference 8. This review included the development of an 84th percentile deterministic ground motion spectrum for the site and a probabilistic prediction of the peak ground acceleration. Results of this study indicated that the peak ground acceleration should likely be higher than 0.25g, but that the amplification of the pga at low frequency, which dominate the response of the main reactor complex, was less than resulted from the enveloping of the response to eight natural earthquake records.

In support of the long-term reconstruction of Bohunice V1 and further seismic assessment of the V2 units, the Slovakian Academy of Science Geophysical Institute, SAV, has been commissioned to conduct a detailed seismic hazard investigation for the Bohunice site. This study is ongoing. In the interim, an earthquake ground motion had to be selected as the basis for the seismic reevaluations and upgrades to be performed in the REKON Project. Using available seismotectonic data, a new study was conducted, Reference 6, to develop uniform hazard spectra for the site and to select an interim review level earthquake, iRLE, for use in the REKON Project. The goal in establishing an iRLE was to define a standard broad banded spectral shape that enveloped the best estimate of the 1E-4/yr 84th percentile site-specific spectrum.

The iRLE selected is based upon a USNRC Regulatory Guide 1.60 spectral shape anchored to 0.25g, but with the pga being further increased to 0.3g. The spectrum from 9 Hz to 33 Hz is then blended in. The resulting spectrum approximates an 84th percentile uniform hazard spectrum which may be inferred from Reference 6 and the deterministic 84th percentile spectra developed in Reference 8. This spectrum is shown in Figure 2. It is important to note that application of the GIP seismic experience-based screening criteria requires that the 5% damped spectral acceleration from about 2 to 8 Hz be enveloped by a seismic experience-based bounding spectrum which has a spectral acceleration of 0.8g in this frequency range. This is the frequency range considered to be most important in reevaluation of SSCs. As can be observed from Figure 2, the iRLE is less than 0.8g between 2 and 8 Hz, thus the GIP seismic experience-based screening criteria are applicable, providing that all other GIP criteria are met.

3. SAFE SHUTDOWN EQUIPMENT LIST

IAEA Technical Guidelines, Reference 5, for development of a Safe Shutdown Equipment List (SSEL) are patterned after the U.S. Seismic Qualification Utility Group (SQUG) GIP, Reference 2 and USNRC Individual Plant Examination for External Events (IPEEE), Reference 12, wherein a minimum set of systems and their components must be verified for seismic adequacy to achieve a safe shutdown and maintain the plant in a safe condition for up to 72 hours. Safe shutdown is defined as either hot or cold shutdown. This is achieved by:

- reactivity control
- reactor coolant system pressure control
- reactor coolant system inventory control
- reactor decay heat removal

In the U.S. IPEEE program, Reference 12, it is also required that one primary path and one redundant path be verified to achieve the above essential functions. It is also required that the capability to mitigate a small LOCA and containment isolation and cooling be verified.

In the reconstruction concept for Bohunice V1 the scope of seismic qualification follows these guidelines and in addition includes the mitigation of the design basis accident and the cooling of spent fuel. In addition, it is required to provide reliable cooling of the reactor to achieve cold shutdown.

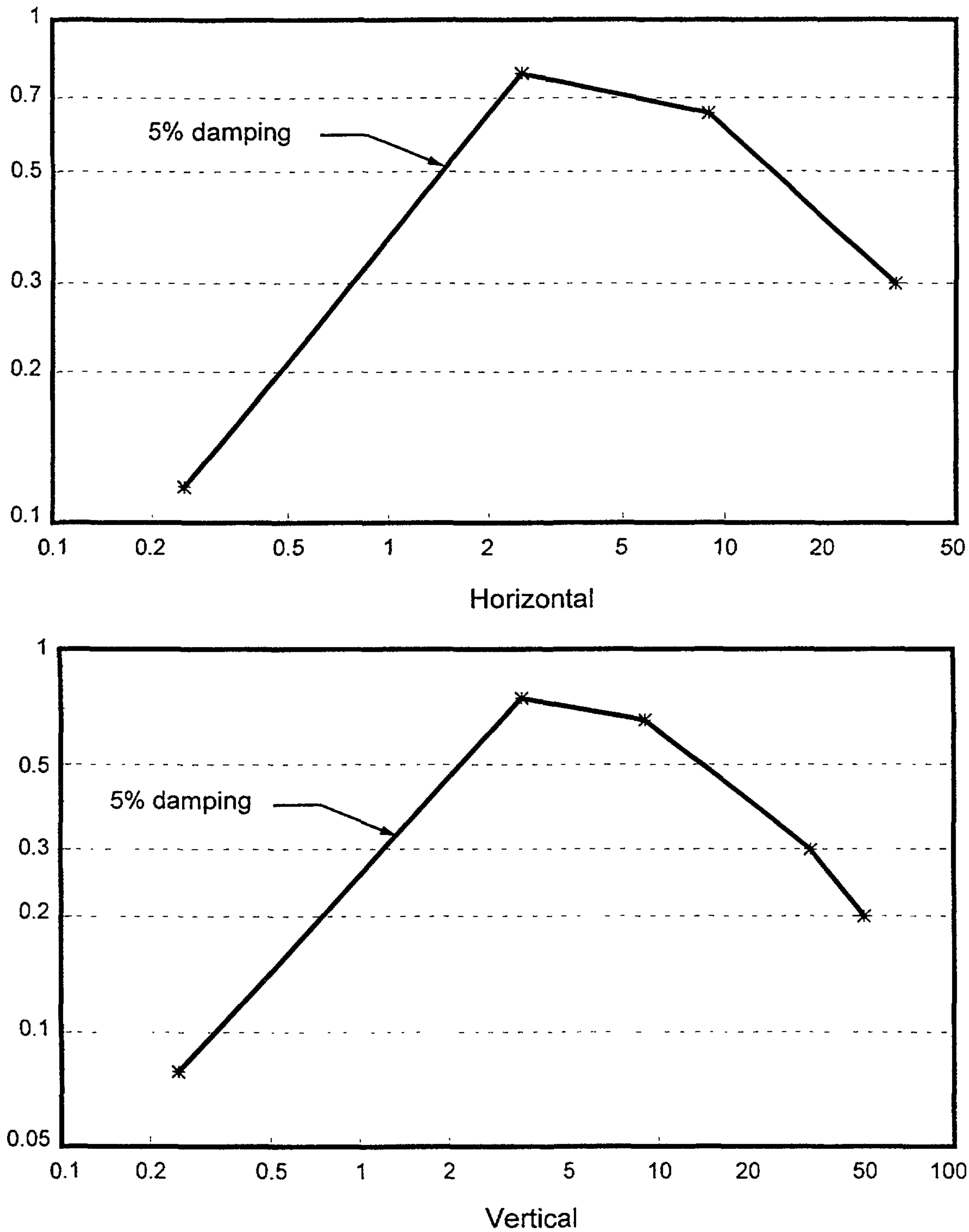

Figure 2: Interim Review Level Earthquake for Bohunice

Because of the many safety deficiencies in the WWER 440-230 design relative to Western Standards, considerable reconfiguration of the plant mechanical, electrical and I&C systems is required. In the case if emergency feedwater and service water, complete new systems are being constructed for which no essential elements are located in vulnerable areas such as the turbine hall. Many electrical systems and their cabling are being replaced or relocated to provide complete separation for fire and other hazards.

Reactivity control is provide by control rod insertion which is achieved by gravity. Control of boron concentration in the primary coolant is utilized for finer adjustment of reactivity during cool down. As is common on many Western PWRs, there is no separate emergency boration system to mitigate ATWS in the event that control rods cannot be inserted.

Pressure control in the emergency power mode is achieved by the power operated relief valves on the pressurizer. Pressurizer heaters are not essential for pressure control since the objective is to reduce pressure and not increase pressure.

Inventory control is achieved through the high pressure safety injection pumps. These pumps are also used for the early mitigation of the design basis accident which is a primary coolant break of 32mm diameter. The existing positive displacement makeup pumps are not essential for the safe shutdown scenario.

Decay heat removal to hot shutdown is achieved by release of steam to the atmosphere through new power operated steam dump valves and cooling of the steam generators by means of the new super emergency feedwater system. While it is feasible to cool the reactor by feed and bleed of the primary system by means of the pressurizer power operated relief valves and the high pressure emergency feedwater system, this is only considered as a last resort method of removing heat from the primary system.

The WWER 440-230's do not incorporate a containment that can confine the release of a large LOCA. A rectangular shaped, reinforced concrete confinement is designed to withstand the design basis accident which is a 32mm diameter primary coolant line break. The design pressure for the confinement is 1 bar.

Confinement cooling to prevent pressure of greater than 1 bar is achieved by means of the confinement spray system. Isolation of all confinement penetrations must be demonstrated through two isolation valves. In addition, the structural stability of piping past the second isolation valve must be demonstrated to an anchor point or through sufficient pipe supports that restrain all important degrees of freedom of the piping.

Spent fuel cooling is achieved via the spent fuel pool heat exchanger which is cooled by the new service water system.

The systems described above that provide the basic functions all require support systems and I&C to monitor and control the essential processes.

The safe shutdown equipment list incorporates all components, piping, cabling and HVAC ducting in the systems that perform the basic functions and in their essential support systems.

4. CIVIL STRUCTURES

Seismic category 1 (SC1) structures to be reassessed and upgraded if necessary are the main reactor building complex and the diesel generator building. Other SC1 buildings are either new additions, are being completely rebuilt or replaced. Structures that are not SC1, but whose failure could affect the functionality of SC1 structures and equipment are categorized as SC2A and include the ventilation stack, a radwaste building and a bridge connecting the auxiliary and reactor buildings. The criteria described herein are for SC1 structures which must be assessed and upgraded. Those structures that are SC2A are evaluated and upgraded per the Slovakian National Building Code, Reference 9.

Modeling: Finite element models are utilized in the evaluation and upgrade design of SC1 structures. Building models are to be constructed of beam and plate elements. The main reactor building complex consists of a concrete confinement structure and steel-framed reactor building superstructure, electrical galleries, ventilation hall and turbine building. All structures of the main building complex are interconnected, but have individual foundations. Thus, the finite element model is required to account for soil-structure interaction of these independent foundations, which in some cases are strip footings. There are many non load bearing unreinforced masonry walls and concrete panels in the main building complex which are not capable of carrying lateral structural loads arising from the iRLE, thus they must be modeled as mass only. Material properties used in the modeling are to be standard handbook properties from building codes and vendor catalogs.

The diesel generator building consists of steel framing and load-bearing unreinforced masonry walls. In this case, the stiffness of the masonry walls must be included in the model.

Input Motion: Analysis of the main reactor building complex is conducted using time history input. Artificial time histories for the three directions of input motion must result in spectra that envelop the iRLE horizontal and vertical spectra at 5% damping and must be statistically independent. Time history analysis is utilized for development of spectra and loads. For the simpler diesel generator building, in-structure response spectra are not needed and response spectrum modal analysis is sufficient for computing structural loads and for purposes of designing upgrades.

Soil-Structure-Interaction: For the reactor building complex, soil structure interaction effects must be properly accounted for using state-of-the-art methods that address embedment effects and the independent foundation input motion. Strain compatible shear modulus and damping must be employed. To account for uncertainty in soil properties, three cases are to be analyzed using the best estimate soil properties and maximum and minimum soil modulus properties. The maximum and minimum modulus are to be taken as two times and one half of the best-estimate case. For the much simpler diesel generator building, it is conservative and adequate to ignore SSI effects and use a fixed-base model of the structure above grade level and conduct response spectrum modal analysis to develop loads.

Damping: Damping to be considered in the analysis includes that due to hysteric energy losses in the structural and soil and radiation damping in the soil. Structural damping to be used in the analysis, whether for developing response spectra or loads in members, are provided in Table 2. Soil damping is determined from the soil characteristics and, in accordance with U.S. practice, Reference 7, it is not limited as long as realistic soil profiles are used or calculations are conducted in the frequency domain.

Development of Spectra: Response spectra are to be developed from the three independent soil stiffness cases and then smoothed and broadened not less than 10% so that a single spectrum for each of the three orthogonal directions envelopes the broadened individual soil stiffness cases. If the maximum and minimum soil cases produce a frequency range of greater than plus and minus 15%, no broadening is necessary.

Strength: Design code ultimate capacity equations shall be used to determine the allowable response of the structural elements. Code ultimate capacity may be determined from U.S., German or Eurocodes.

Table 2: Damping Values D in % of Critical Damping

Type of Structures	Damping Value D (%)
Welded aluminum structures	4
Welded and friction-bolted steel structures	4
Bearing-bolted steel structures	7
Reinforced concrete structures	7

Notes:

(1) These values are appropriate for linear analysis and should not be used for non-linear analysis where hysteritic energy dissipations is directly considered.

(2) Lower damping values may be appropriate for development of response spectra if the overall structural demand is less than about 1/2 of yield.

Ductility: The extent to which structures may be loaded beyond code ultimate capacity is determined on an element basis rather than a global basis. Ductility factors, $F\mu$, shown in Table 3 may be used to reduce the calculated elastic inertia response of structural elements. These factors are derived from Reference 4 and when applied in combination with code ultimate capacity equations, a HCLPF is achieved.

Load Combination: Response to the iRLE shall be combined with concurrent static and dynamic loads in accordance with:

$$1.0\,(DL + LL + T) + 1.0\,iRLE_i/F\mu + 1.0\,R_{SL} < U$$

where:

DL = Dead Load
LL = Live Load
T = Restraint of Thermal Expansion
$iRLE_i$ = Seismic Inertia Load
R_{SL} = Other earthquake induced loads such as differential motion and systems interaction effects. The design basis accident is not postulated to occur simultaneously with the iRLE.
$F\mu$ = Element ductility factor per Table 3
U = Ultimate code capacity

Non-permanent loads that counteract the effects of seismic loading shall not be included in the load combination.

Limitation factors on concurrent loads are:

0.25 for LL
0.3 for Snow load
0.0 for working loads of hoists and cranes

5. MECHANICAL AND ELECTRICAL COMPONENTS

Mechanical and electrical components may be evaluated by performing a detailed walkdown and screening of components using seismic experience-based screening criteria,

Table 3: Inelastic Energy Absorption Factors, Fμ

	Fμ Value
Steel braced frames	
- Beams	1.60
- Tension-compression diagonal braces	1.40
- Tension-only diagonal braces, chevron, V, and K bracing	1.20
- Columns	1.00
Concrete braced frames or concrete/steel frame systems	1.40
- Tension-compression diagonal braces	1.20
- Columns	1.00
Ordinary steel moment frames	
- Beams	1.50
- Columns in flexure	1.50
- Columns in axial compression or shear	1.00
Ordinary concrete moment frames	
- Beams	1.20
- Columns in flexure	1.20
- Columns in axial compression or shear	1.00
Reinforced concrete shear walls	
- In-plane flexure	1.40
- In-plane shear	1.20
- Out-of-plane flexure	1.40
- Out-of-plane shear	1.00
Connections for all structural systems	
- Assure connection stronger than members by 20%	1.00

Connections - For all structural systems, the connections are typically governed by less ductile failure modes than the attached members. As a result, connections must be capable of withstanding the lesser of (1) the strength of the connecting members; (2) the member force corresponding to Fμ of unity; or (3) the maximum forces that can be transmitted through the connection by the structural system.

analysis, testing or a combination of these methods. Components which must be verified are categorized as SC1A, SC1B, SC1C and SC2A. SC1A components are those that must function during or after the earthquake. SC1B components must survive the earthquake without loss of pressure boundary. SC1C components must only maintain their stability and SC2A components are nonessential components whose failure could impede the function of SC1 components. The emphasis for verification of seismic adequacy is on walkdown and screening where applicable. This methodology is generally applicable to all categories except the case of SC1A components which must function during the strong motion shaking. In cases where screening cannot be accomplished, selected analyses or tests must be conducted to verify seismic adequacy.

Verification by Walkdown and Screening: The GIP, Reference 2, is utilized to guide the walkdown and screening of components. The GIP covers 20 generic classes of equipment and the screening criteria contained in the GIP are based for the most part on the successful performance of equipment in strong motion seismic events. The seismic experience database that forms the basis for the GIP screening criteria is primarily for U.S. commercial grade equipment, but some Western Europe and Asian data is included. In order to apply the GIP screening criteria to equipment manufactured in Eastern Europe and the former Soviet Union, the engineers performing the walkdowns and screening must be experienced in not only the application of the GIP, but also in the background of the data which served as the basis for the GIP. This background and training are necessary as a condition for demonstrating the applicability of the GIP and for making screening judgments.

The walkdown screening per the GIP criteria requires fundamentally that:

- The equipment is represented in the data base. Note that absolute representation is not necessary but similarity of important features must be demonstrated.
- Anchorage must be verified. Alternate anchorage verification criteria have been formulated for the REKON Project to specifically account for the anchorage configurations used.
- GIP criteria for capacity vs. demand must be satisfied. This requires the floor response spectra are bounded by 1.5 times the seismic experience based SQUG bounding spectrum or that the Ground Motion Spectrum is enveloped by the SQUG bounding spectrum. For most components, the later criteria requires that the component must be demonstrated to have a natural frequency greater than 8Hz.
- Relays within the component must meet specific relay screening criteria.
- Components must be free of seismic induced interactions (falling of objects onto the component, impact, spray, etc.)

All components which do not meet the screening are outliers and require alternate methods to demonstrate seismic adequacy. Note that in almost all cases, the relay screening is not applicable and relays must be addressed separately even if the enclosures meet all of the screens.

Verification by Analysis: For the most part, the seismic adequacy verification does not require detailed analysis and the analytical verification is focused on anchorage capacity. The GIP provides detailed guidance on evaluation of anchorage with emphasis on expansion anchors. In general, the original anchorage of mechanical and electrical equipment at Bohunice did not include expansion anchors. Many components were unanchored and must be anchored, in which case expansion anchors are often used and the GIP is used as guidance in sizing expansion anchors. Often details of the existing anchorage cannot be verified and new anchorage must be designed.

Outliers that do not meet the GIP walkdown screening criteria may be resolved by analysis. Analysis is generally performed to assess a strength issue or define a displacement. In general, analysis cannot be reliably conducted to verify function of electro-mechanical devices subjected to dynamic load. If analysis is performed a variety of methods may be used which, in order of increasing complexity, include:

- Static analysis with the coefficient being defined as the peak of the response spectrum at the attachment point for flexible systems or the zero period acceleration for rigid systems.

- Response spectrum modal analysis.
- Linear time history analysis
- Non-linear time history analysis.

Damping values applicable to seismic response of component are listed in Table 4.

If components are evaluated by analysis, applicable code equations are utilized to assess capacity. Codes of different countries and for different types of components differ slightly, but, in general, for the same failure modes in components and supports the allowable stresses are similar. U.S. ASME and German KTA standards are applied where applicable. Ductility factors, $F\mu$, may be applied to ductile components where structural capacity is the failure mode of concern. Ductility factors may generally not be applied to components if deformations are critical to function.

Table 4:
Damping Values for Mechanical and Electrical Components
(percentages of the critical damping)

Structures and Components	Damping Value
Welded steel structures (support structures)	4.0%
Bolted steel structures (support structures)	7.0%
Tanks, vessels, heat exchangers	4.0%
Pumps	3.0%
Valves	3.0%
Instrument cabinets and racks	3.0%
Piping	4.0%
HVAC ducts	7.0%
Cable trays <50% loaded	10.0%
Cable trays ≥50% loaded	15.0%
Sloshing liquid	0.5%

Ductility factors, $F\mu$, for components are listed in Table 5.

Load combinations applicable to components vary with the type of component or support. Seismic inertia loads are combined with normal operating loads such as dead weight and internal pressure. Any operating load that is present more than 2% of the time must also be included in the load combination.

For pressure boundary components, the applicable load combination is:

$$1.0\ DL + 1.0\ LL + 1.0\ P + iRLE/F\mu < \text{ASME Level D or KTA allowable stress}$$

For component supports the applicable load combination is:

$$1.0\ DL+1.0\ LL+1.0\ T+(iRLEi^2 + IRLEm^2)^{1/2}/F\mu < \text{ASME Level D or equiv. KTA}$$

where:

$iRLE_m$ is seismic anchor motion loading and the other terms are as defined in Section 4.

Note that for non-ductile elements of supports, $F\mu$ is 1.0.

For non-pressure components such as electrical enclosures, the structural criteria of Section 4 is to be used.

Testing: In general testing is limited to electro-mechanical devices such as relays, motor contactors and breakers. Testing should comply to recognized national standards such as KTA 2201.4, Reference 10, or IEEE 344, Reference 11. In general, multi-axial, multi-frequency testing should be conducted, but single-axis, single-frequency tests are acceptable under certain qualifying conditions.

Existing test data are to be evaluated relative to criteria in current standards and the requirements derived for the specific location. Major requirements for seismic verification by test are:

Test Seismic Input Motion: The seismic input is defined by a Test Response Spectrum (TRS) applicable at the location of the specimen mounting (in-cabinet, floor, wall, etc.). The TRS must envelop the Required Response Spectrum (RRS) defined at the

Table 5: Ductility Energy Absorption Ratio Values, $F\mu$

Components and Supports	Structural Ductility Energy Absorption Ratio $F\mu$
Passive mechanical and electrical components:	
Ductile material	1.50
Non-ductile material	1.00
Passive mechanical and electrical component supports:	
Steel frames	2.00
Steel skirts, saddles, etc.	1.50
Active mechanical and electrical components including supports	1.00
Instrument cabinets and racks including supports	1.00
Pipes fabricated of steel:	
Butt welded	1.50
Socket welded or bolted flange	1.50
Threaded	1.00
Piping supports made of steel:	
Rigid supports (hangers, columns)	1.25
Framework	2.00
HVAC ducts, including supports	1.50
Cable trays including supports	1.50
Steel substructures (welded or bolted)	
Columns	1.00
Beam members	2.00
Connection members	1.00

mounting location. The actual input motion of the shake table must result in a response spectrum that is equal to or exceeds the RRS in all frequency ranges of interest. If the input motion is single frequency, the envelope of the spectra resulting from the single frequency inputs must envelop the RRS for all frequencies above the fundamental frequency. See Figure 3 as an example of enveloping of single frequency spectra. If the response investigation demonstrates that the dynamic response is primarily in a single mode, then single-frequency testing is acceptable without a penalty. If response is multimode, the envelope of the single frequency TRS must be reduced by $\sqrt{2}$ before comparing to the RRS.

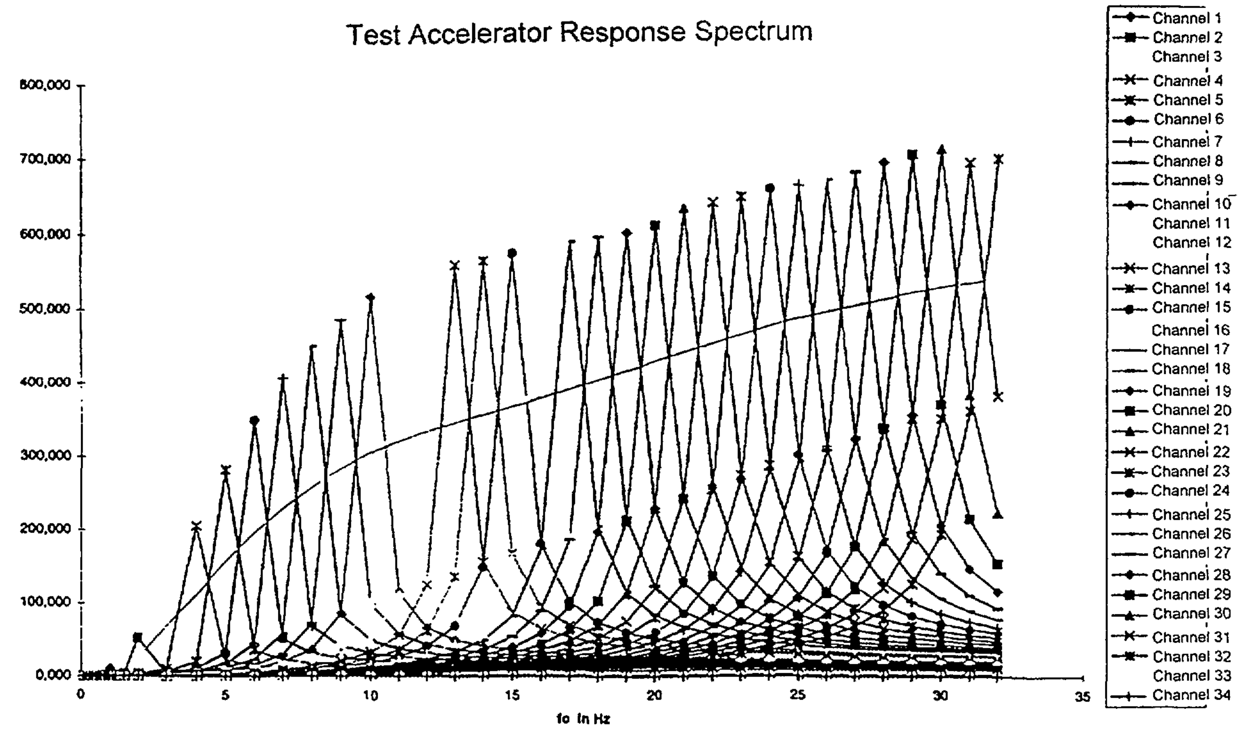

Figure 3: Enveloping of Single Frequency TRS

If testing is single axis, and it can be reasonable demonstrated by response investigations or by geometric arguments that there is very limited coupling between directional responses or that the component function is only sensitive to a single direction response, then the single axis test TRS is acceptable for comparison to the RRS for each axis. Otherwise, the single direction response must be reduced before comparing to the RRS. The reduction should be either $\sqrt{2}$ if the response is sensitive to two directional input or $\sqrt{3}$ if sensitive to three directional input.

Monitoring: If electrical functions are critical during the shaking, then the essential functions must be monitored during the test. In the case of devices such as relays that may be in different states during normal operation, each state must be tested and monitored. If there are no active functions to be performed during the time frame of the earthquake, then it is only necessary to verify that the component is functional after the earthquake.

Combinations of Analysis and Test: In many instances it is necessary to test devices such a relays for function, but it may not be necessary to test the entire cabinet enclosure. Typically, transmissibilities will be developed from the floor to the device by analysis, in-situ testing or by use of generic amplifications derived from experience as provided in the GIP. In the case of generic amplifications, some modifications have been made to the GIP guidance to reflect stiffening that has been conducted on flexible panels to which relays are mounted. Panels have been stiffened to a point that amplifications greater than 3.0 are not postulated.

6. DISTRIBUTION SYSTEMS

For distribution systems like piping, cable trays and HVAC ducts a combination of detailed walkdown using experience-based screening criteria and selected analysis is applied for evaluation. Selecting of the appropriate procedure depends mainly on the complexity of the system. Screening criteria have been developed in previous seismic reevaluation projects in various European plants and were modified for the specific issues at Bohunice.

Piping and Piping Supports: It is common practice that piping systems are designed or reevaluated either by detailed finite element analysis or by simplified methods like support span charts. In new design the use of simplified methods is usually restricted to small bore piping while the rest is verified by analysis, which leads to an extensive amount of computer calculations.

With the general experience that piping systems are very rugged under seismic loads, even if they have not specifically been designed to seismic criteria, it is judged to be acceptable to increase the scope of piping systems to be evaluated by simplified method.

Due to experience gained in various reevaluation projects in Germany and Western Europe, the use of detailed analysis for Bohunice piping systems is limited to:

- Reactor coolant system
- High pressure systems with design temperature > 100 C
- Selected system sections with extremely complex layout

All other piping, including all systems or system sections which are classified as SC2A, will be evaluated with simplified screening criteria.

Verification by Analysis: For those systems, where analysis is required the response spectrum modal analysis method is to be used with applicable code equations and stress values. Modeling, decoupling and system properties are done in accordance with international standards like U.S. ASME and German KTA. Stress intensification factors are used in accordance with the same codes, provided that they represent the geometric configuration. A damping value of 4% is specified for all diameters whereas higher values may be accepted if they are justified.

The load combination for piping analysis is specified as:

Pressure + deadload + earthquake loads < ASME or KTA Level D allowable stress

Earthquake load is defined as the inertia loading from the iRLE.

Where structural capacity will exceed allowables a ductility factor of 1.5 may be applied to represent non-linear behavior of the system, however, in accordance with Table 5, a factor of 1.0 is applied to nonductile joints.

Verification by Walkdown and Screening: The method which is used for most of the piping systems is a combination of seismic experience and a variety of generic calculations representing typical piping layouts and arrangements.

The screening criteria are focused to satisfy the three major aspects for verification of seismic adequacy of piping systems which are:

- Vertical ad horizontal piping support spacing
- Flexibility check and expansion length for anchor movements (thermal and seismic)
- Support loads for verification of substructures and anchoring

The screening criteria for the above parameters are set up in tables and nomographs for easy use in walkdown screening

The engineers performing the walkdown must be trained not only in the application of these criteria but also must be able to verify the construction quality of the piping arrangement. In cases of non-applicability of the criteria, the criteria may be modified according to the individual situation or supplemented by some simple analysis.

The walkdown staff further has to have extensive knowledge in system operation parameters such as normal and transient conditions to assure that all credible load combinations are addressed and that seismic upgrades will not cause adverse affects to the operational requirements.

Verification of Piping Supports: To assure seismic resistance of piping systems, piping supports and their anchoring is more relevant than the pressure piping itself. Experience gained in various seismic evaluation tasks show, that for piping supports, the most critical parts are either the anchoring, certain features of some of the standard support items or the welds, but in a very few cases the steel structures. The focus is on the non-ductile portions of the supports.

The acceptance of capacity of piping systems for seismic loads depends mainly on the supports performing the right function and having a continuous load path.

Most existing piping supports are standard designs and construction and the development of screening and walkdown criteria take into account the specific detail. To verify integrity of piping supports by walkdown and screening a set of criteria were set up which address the:

- Functional performance of the pipe support substructure
- Capacity of the most critical items
- Type and quality of welding

Due to similarity of piping supports and standard configurations not every support has to be evaluated, only a sampling of typical construction.

Guidance for individual modification of the screening criteria, especially for quality discrepancies in welding, are provided. A simple assessment procedure was established taking into account geometric aspects as well as fabrication parameters like welding undercut, holes or gaps.

For piping supports for which the simplified procedure is not applicable, due to complex geometry or loading, static analyses are performed to verify seismic adequacy. The analyses are conducted in accordance with current international standards like ASME or German DIN or Eurocode and are performed with standard computer tools.

Where necessary and applicable, ductility factors Fu, as listed in Table 5, may be used.

Assessment of HVAC Ducts: Essential HVAC ducting at Bohunice V1 is constructed of either folded seam or welded seam sheet metal. In both cases, longitudinal connections are made by bolted flanges. The folded seam ducting is thinner, thus potentially more vulnerable to seismic inertia loading, plus has a much less robust design of the connection flanges.

Failure modes to be considered in the assessment are buckling of the ducting, opening of the folded seams, attachment of the ducting to the flanges and the pressure integrity of the bolted flange joint.

Evaluation of the failure modes by analysis is difficult and uncertain. Fortunately, Siemens has a large database of HVAC ducting tests which includes data for ducting which is very similar in design and construction. From these test data it was determined that the critical failure mode for both cases is opening of the bolted flange joints, thus compromising the pressure retention capability of the ducting. Opening of the flanges does not, however, result in instability or collapses of the ducting systems.

Test results provide the bending moment capacity of the ducting flange joint. Given this moment capacity, span spacing for vertical and horizontal supports can be determined. In determining the allowable span spacing the peak of the 7% damped floor spectrum is used as the seismic demand. Seven percent damping is the acceptable value from the REKON standard and is obtained from KTA standards for NPP design. By using the peak of the spectrum for demand, the eigenfrequency of the HVAC and support system does not have to be calculated or controlled.

Support loads from seismic inertia are defined as the peak 7% damped spectral acceleration times the tributary weight of the ducting. For vertical support spacing, dead load is added to the tributary seismic load.

Existing supports are generally inadequate and new supports must be added. The new supports are standardized designs sized to carry the tributary loading derived from the governing span spacing. In almost all cases, the supports are welded to existing structural elements. Where support attachment to the structures is by expansion anchors, manufacturers allowable loading is used to size the expansion anchors.

The span spacing and support screening criteria are summarized in tables for easy use in walkdown screening and placement of new supports.

Cable Raceways: Existing cable raceways are in general evaluated in accordance with the criteria in the GIP with some modifications to accommodate unique features of cable raceways and their supports at Bohunice. Most existing raceways do not pass the GIP screening, thus the walkdown screening of cable raceways is focused primarily on modifying the raceway support system to comply with the GIP. In many cases, new supports and their anchorage are designed to standard strength criteria utilizing the damping values in Table 4 and ductility factors in Table 5.

Many new routings of cables necessitate complete new design of raceway systems. These new system raceway designs have been done to existing KTA standards and do not utilize the more liberal seismic experience based GIP criteria.

7. *SYSTEMS INTERACTIONS*

Seismic-induced systems interactions may be spatial or systematic. Spatial interactions result from a failure or deflection of a SC2 item which may impair the function of an SC1 item by falling on it, impacting it or spraying it. These types of interactions are usually identified during the walkdown and screening phase. The most common of these interaction sources at Bohunice are the numerous unreinforced masonry walls and concrete panels which are weakly attached to the steel structural members. Systematic interactions might consist of failures of a pipe or heat exchanger that is not safety related, but is not isolated from an essential system. This can also occur in electrical systems where a short in a non safety circuit is not isolated from an essential circuit. These types of interactions are usually identified from reviews of flow diagrams and circuit diagrams.

The criteria for evaluation and upgrading of potential systems interactions sources is flexible to the extent that it must only be demonstrated that the interaction cannot occur. Use of the Slovakian Building code, Reference 9, for structural type interaction sources is acceptable and consistent with international guidelines, Reference 1. Other methods, such as energy methods suggested in Reference 5 are acceptable as is engineering judgment based on seismic experience. Often it is convenient and not a cost penalty to upgrade anchorage for potential systems interaction sources using the SCI criteria summarized in this paper. In other cases, such as for a large structure which is an interaction source, use of the local building code for upgrades is the most prudent approach. The engineering effort vs. the hardware cost must be considered in making upgrades to alleviate potential interactions.

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A REGULATORY VIEW OF THE SEISMIC ASSESSMENT OF EXISTING NUCLEAR STRUCTURES IN THE UNITED KINGDOM

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Abstract

The paper describes the background to the seismic assessment of existing nuclear structures in the United Kingdom. Nuclear installations in this country were not designed specifically to resist earthquakes until the nineteen-seventies, although older plants were robustly constructed. The seismic capability of these older installations is now being evaluated as part of the periodic safety reviews which nuclear licensees are required to carry out. The regulatory requirements which set the framework for these studies are explained. The licensees' processes of hazard appraisal and examination of the response of the structure are briefly summarized. Regulatory views on some of the criteria used to judge the adequacy of safety are discussed. Finally the paper provides some comments on future initiatives and possible areas of development.

1. INTRODUCTION

The first electricity-generating nuclear power station to be constructed in the United Kingdom was Calder Hall, a Magnox type gas-cooled reactor which began operating in 1956 and is still in operation 40 years later. In those early days of the UK nuclear programme the installations were not designed specifically to resist earthquakes, indeed seismotectonics was in its infancy. However, as the potentially damaging effects of earthquakes even in low seismicity areas came to be recognized, modern standards were developed which considered earthquake forces. The first power reactors for which seismic loading was considered in the UK were the Heysham Stage 2 and Torness Advanced Gas-cooled Reactors (AGRs), which were designed in the 1970s and received consent to begin construction in 1980.

This paper described the background and the present position for seismic assessment of existing nuclear installations. This is reported within the context of the overall arrangements for the regulation of nuclear safety in the UK. A strategy has been adopted of reviewing the safety of all nuclear installations for their long-term operation, this programme is known as the periodic safety review (PSR). The evaluation of the seismic capability of a plant forms part of the investigation. The paper explains the regulatory view on the criteria used to assess the adequacy of the performance of the plant.

HM Nuclear Installations Inspectorate has now had several years' experience of assessing seismic safety cases for existing nuclear plant. At the same time, we are aware of similar initiatives in other countries and have, in fact, had a number of contacts on this subject, either bilaterally or through conferences, seminars and other meetings, such as those arranged by the International Atomic Energy Agency (IAEA). It is hoped that both we and the licensees can gain from these exchanges in terms of identifying potential improvements and areas of confidence in a plant's performance, as well as learning new techniques which can be adapted for assessment of UK plant.

2. REGULATION

In the UK, the main legislation governing the safety of nuclear installations is the Health and Safety at Work etc. Act 1974 and the associated relevant statutory provisions of the Nuclear Installations Act 1965. Under the Nuclear Installation Act, no site may be used for the purpose of installing or operating any commercial nuclear installation unless a nuclear site licence has been granted by the Health and Safety Executive (HSE) and is for the time being in force. HM Nuclear Installations Inspectorate (NII) is that part of HSE responsible for administering this licensing function.

The Health and Safety at Work etc. Act requires the provision and maintenance of plant and systems of work that, so far as is reasonably practicable, are safe and without risks to health. This means that risks must be reduced to as low as is reasonably practicable, this is the "ALARP principle'. The legislation places the primary responsibility for safety on the licensee of each installation. It is the duty of NII to see that appropriate standards are developed, achieved and maintained by licensees, to ensure that any necessary safety precautions are taken, and to monitor and regulate the safety of plant by means of its powers under the licence and relevant regulations. This is a non-prescriptive licensing regime chosen so that responsibility for safety is left with the licensee.

The Nuclear Installations Act gives the NII, on behalf of HSE, the power to attach conditions to each site licence in the interests of safety. There are 35 standard licence conditions which are applied to most sites. Before granting a licence, the NII requires a written demonstration of safety, the safety case, and the licensee must make adequate arrangements for keeping the safety case up to date (Licence Condition 14). Also, the licence requires the licensee to carry out a periodic and systematic review and reassessment of safety cases (Licence Condition 15). Long Term Safety Reviews (LTSR) carried out for the Magnox gas-cooled reactors after they had been operating for 20 years were the earliest form of continuing performance review in the UK. Periodic safety reviews (PSR) are now required every ten years, and these have already been carried out for some Magnox stations operating beyond 30 years and are currently under way for the AGRs. Seismic assessment of the installation is one of the topics covered by the LTSRs and PSRs.

3. SAFETY ASSESSMENT PRINCIPLES

The regulatory system in the UK is non-prescriptive so the licensees are free to develop arrangements to give appropriate levels of safety to their plant. To help judge the adequacy of the licensees individual safety cases, HSE has published the safety assessment principles (SAPs, Ref. 1). These are used by the NII Inspectors in their assessments; they are not mandatory. The SAPs are fairly general in nature providing a broad view in most instances. They have been developed drawing on past experience, best practice and international standards (IAEA). The guidance provided by the SAPs was initially intended for new plant, i.e. plant yet to be constructed. In many cases, however, it is equally applicable to older plant. As stated in the text of the SAPs, for older plant the age of the plant and its projected life are important factors to be considered when making an assessment.

The SAPs can be split into five groups:

1. Fundamental principles

2. Siting principles

3. Safety analysis principles

4. Engineering principles

5. Life cycle requirements

When making a seismic assessment of an existing structure, principles from most groups are relevant, but Principles P119 to P125 and P128 to P131 are directly applicable. Further discussion of the application of the SAPs to seismic design can be found in Reference 2.

In response to Sir Frank Layfield's recommendation at the Sizewell B Inquiry to formulate guidelines on the tolerable levels of individual and social risk to workers and the public from nuclear power stations, HSE produced its Tolerability of Risk (TOR) Report (Ref. 3). TOR effectively defines a number of high level criteria which modern nuclear power stations must meet to comply with the levels of risk which, in HSE's judgement, society and public are prepared to tolerate. A plant would not be licensable if the risk was intolerable. As the risk decreases, it enters the tolerable region and the plant is in principle licensable, but UK law requires the risk to be pushed down to 'as low a level as is reasonably practicable' (ALARP). Further reductions in risk would bring it into the broadly acceptable range where NII would not normally push for further improvement, though the law still requires the licensee to provide such improvements as are reasonably practicable.

As pointed out in TOR, there are additional uncertainties in quantifying the risks from some older designs since the plants, although often robust, do not have designs and construction governed by modern standards of quality assurance or quantitative risk estimation. On the other hand, HSE does not consider it reasonable to expect such older plants to demonstrate that they meet all the safety requirements that would be required for modern plants.

TOR and SAPs, therefore, give us some basic numerical guidelines, but the essence of our seismic assessments is that we are looking for the licensees to provide a demonstration that the risks from their plant in the event of an earthquake are both tolerable and have been reduced to as low as is reasonably practicable.

4. SEISMIC HAZARD

The UK is situated in an intra-plate tectonic region of north-western Europe which has low seismicity. Since the late 70's, techniques for the determination of seismic hazard have developed significantly and various approaches are now available for calculating the site specific hazard. SAP P129 requires that a design basis earthquake should be determined so that conservatively it has a predicted frequency of being exceeded no more than once in 10,000 years. At the Sizewell 'B' and Hinkley 'C' Inquiries (Refs. 4 & 5), the site specific seismic hazard was reviewed, techniques are now well established. Principia Mechanica Ltd (PML), developed a piece-wise linear spectrum from southern European and US data using the Newmark Hall methodology. This spectrum has been used extensively for the design of new plants and the assessment of existing structures. More recently, strong motion data from intra-plate areas has been collected to produce uniform hazard spectra (UHS). This latter approach is believed to be more appropriate by some researchers. However, there are no strong motion UK records and there is debate over the validity of the low frequency section of the UHS generated. The record of historical earthquakes for the UK is only essentially complete above magnitude 4 for the last 200 years. There is still considerable uncertainty in defining the seismic hazard.

5. SEISMIC EVALUATION OF EXISTING NUCLEAR PLANT

5.1 Overview of the Programme of Reviews

British Nuclear Fuels plc (BNFL), and the pre-privatization Scottish Nuclear (SNL) and Nuclear Electric (NE) have all carried out reviews of older plants. The seismic capability of each of the Magnox reactors was assessed in its Long Term Safety Review (LTSR). The purpose of the LTSR programme was to demonstrate that the plants would be adequately safe for at least 30 years' operation. For the Chapelcross and Calder Hall reactors, BNFL used techniques which were developed during the Seismic Damage Assessment (SDA) of reprocessing plant at Sellafield (see below). From the experience gained in both the LTSRs and SDAs, ways are being developed by the licensees to enhance the methodology of seismic evaluation. Magnox Electric plc (MEP) is now carrying out studies to show that its Magnox reactors are fit for continued operation beyond 30 years and BNFL is doing the same for operation beyond 40 years for Calder Hall and Chapelcross. Nuclear Electric Limited (NEL) and Scottish Nuclear Limited (SNL) have also begun Periodic Safety Reviews (PSRs) of their AGR reactors. All these reviews include seismic evaluation.

5.2 Seismic Safety Strategy

As a result of the non-prescriptive nature of the British regulatory system, the approach to achieving an acceptable level of safety at existing nuclear installations varies between the licensees and the different types of plant involved. The review carried out to date by licensees have compared the performance of each structure to various seismic input reference levels. This approach has been used to indicate the level of hazard which would cause failure of the system as the complexity of the structure and the definition of failure often make the calculation of ultimate seismic capability very difficult.

5.2.1 NE and SNL's Magnox Long Term Safety Reviews

The LTSR assessments used a ground motion defined by a 0.1g horizontal pga and the PML response spectrum. A consideration in choosing this level was undoubtedly that the IAEA guidance for the siting of new nuclear power plants (Ref. 6) recommends that, regardless of any lower apparent exposure to seismic hazard, all plants should adopt a minimum value of 0.1g peak ground acceleration. The intention was to establish that the major structures and the plant used to shut down the reactor, remove decay heat and maintain negative reactivity could survive this motion, and to use this information as a basis for deciding whether the stations were acceptably safe. The assessment should also have identified any improvements which were reasonably practicable. Plant improvements have indeed resulted from these reviews, including such things as better restraint of electrical equipment and the installation of tertiary boiler feed systems for decay heat removal.

5.2.2 BNFL's Seismic Damage Assessment

BNFL have carried out a seismic damage assessment (SDA) for the chemical plant at Sellafield, firstly to identify the potential for improvements to the robustness of the installations, and secondly to allow preparation of emergency plans for coping with the consequences of an earthquake. The SDA predicted the likely plant performance at 0.125g, 0.25g, and 0.35g pga (PML spectrum). The 'walkdown' methodology developed in the USA was also used. Many of the techniques in the Electric Power Research Institute (EPRI) methodology for the conservative deterministic failure margin (CDFM) (Ref. 7) were employed. The SDA aimed, however, to provide only a slightly conservative, best estimate of the plant performance and therefore did not actually comply with all the CDFM criteria. The SDA techniques are now being developed to provide a methodology for periodic safety reviews. For their reactors, BNFL adopted a two-stage process. All safety-related plant was shown to be capable of surviving a 0.125g pga event (PML spectrum) and a subset of 'plant essential to safety' one of 0.2g pga.

5.2.3 AGR Periodic Safety Reviews

NEL and SN have proposed the following policy for the integrity of protection in the periodic safety review programme for AGRs:

(a) For any infrequent initiating event (more frequent than 10^{-3} per annum), there should normally be at least two lines of protection to perform any essential function, with diversity between each line;

(b) For any infrequent initiating event (less than or equal to 10^{-3} per annum) there should be at least one line of protection to perform any essential function, and that line should be provided with redundancy.

NEL and SN have stated that, for the seismic safety case, the magnitude of the infrequent initiating event should correspond to a severity consistent with a return frequency of 10^{-4} per annum at the site. In some plant reviews, e.g. Hinkley Point B, Hunterston B, due to the urgency of the work, the spectrum for the assessment of the 'bottom line' plant has been pragmatically agreed as the PML spectrum anchored at 0.14g pga. This is thought to be a sufficiently adequate surrogate of the 10^{-4} per annum UHS. For other AGRs, NEL intend to provide a site specific UHS at the expected confidence level, with a probability of exceedance of 1 in 10,000 per year. The appraisal will examine all essential structures and a single line of protection (including redundancy) to trip, shutdown and cool the reactor. The systems involved have been designated 'the bottom line plant'.

The plant which will provide a diverse means of achieving trip, shutdown and post trip cooling against frequent events, is called the 'second line plant'. The ground motion specification for the frequent initiating event is 0.1g pga and the PML response spectrum appropriate to the site condition. This choice of input motion allows continuity with the methods used in the assessments of the Magnox stations for the LTSRs.

Plant whose failure could threaten the defined lines of protection is known as 'related plant'. It will be assessed to the same level as the plant which it could threaten.

Building response to the input ground motions will be determined using established modelling techniques and soil structure interaction. Two approaches will be used for plant assessment: analysis and 'walkdown'. The 'walkdown' will make use of the SQUG Generic Implementation Procedure (Ref. 8) and its associated caveats when using earthquake experience data. Analysis will be used whenever the walkdown approach is not applicable or fails to demonstrate that the item can withstand the earthquake. The capacity of the structure and plant items will be determined using design code allowable stresses, strains and deflections in the first instance. Should the determined capacity be inadequate for the proposed functional requirement, more detailed calculations may be carried out allowing limited but tolerable damage or inelasticity.

By reviewing against two levels of seismic input motion which can be related to frequency of occurrence and past experience, a judgement can be made on the acceptability of the plant. NEL and SN intend to declare the margins above assessment levels in order to provide confidence in the methodology and to help establish that the ALARP principle has been satisfied.

5.2.4 MEP, Magnox Periodic Safety Reviews

To establish that the MEP Magnox plants can continue operating safely beyond 30 years, Magnox PSRs are being carried out as a development of the LTSR programme. NII has requested that the licensees' PSRs should show, where possible, that the 'bottom line plant' has a safety margin beyond the capacity which was demonstrated in the LTSR against an earthquake ground motion defined by the PML response spectrum anchored to 0.1g pga.

Discussions with MEP are still ongoing however, but MEP has proposed to demonstrate that a single line of protection exists against the 10^{-4} per annum seismic event in an essentially similar but perhaps simplified manner to that for the AGR PSR.

6. REGULATORY VIEW OF SEISMIC EVALUATION IN PSRs

6.1 Objectives

The general objectives established for the Magnox LTSRs for assessors to review the licensees' safety cases against are also an appropriate guide for periodic reviews on AGRs, chemical plants and other existing nuclear installations. They are:

(1) To confirm that the plant is adequately safe for continued operation.

(2) To identify any life limiting features.

(3) To compare the existing plant's safety against modern standards and to instigate any reasonable practicable improvements.

The findings from NII's review of the Magnox LTSRs (Ref. 9) have been published. The general approach was further considered in the 'Submission to the Nuclear Review' from the HSC (Ref. 10) and as such the programme represents a basis upon which further development can be made.

6.2 Seismic Safety Case Considerations

A Safety Case is the written justification of a plant's safety and shows that the risks are tolerable and ALARP. The adequacy of the Safety Case is judged using TOR, SAPs, the debate from public inquiries, international discussion and experience.

The Safety Case should clearly establish the strategy for showing an adequate level of seismic safety for a structure. It should explain how tolerability and ALARP requirements have been met. This could be achieved by producing a generic top tier document that describes an appropriate methodology providing a coherent philosophy from input motion to performance criteria. This top tier document should link safety significance to performance criteria. The SAPs in the safety analysis principles discuss the use of both probabilistic and deterministic techniques. In practice, deterministic methods have been used exclusively in the seismic safety cases so far submitted. The lower bound of demonstration of seismic safety has generally been established in reviews as the PML spectrum anchored at 0.1g pga. Judgements on the satisfaction of the ALARP principle may be aided by considering the numerical margin provided by the structure above the hazard reference level and carrying out a comparison against modern standards. The size of the nuclear inventory should be considered in assessing risk. Research initiatives are progressing on seismic probabilistic risk assessment (PRA), particularly investigation into structural fragility curves. It is hoped that a more quantitative method of the assessment of risk will soon be adopted to be used in conjunction with present techniques.

The Safety Case should describe the safety significance of a structure, particularly with reference to overall plant safety and the consequences of failure. A comprehensive plant hazard identification procedure is needed to establish adequate lines of protection. The present reviews have used a combination of desk top studies and 'walkdowns'. The state of the structure that would constitute failure, and the consequences of that failure, should be identified. For multi-plant sites, the allocation of the risk between facilities should be studied as the earthquake will affect the whole site simultaneously. Safety categorization of the structure may help in determining the level of examination required. Once the safety significance of a structure has been established, the appropriate level of seismic performance can be identified.

For older plant, the condition of the structure may have considerable impact on its seismic performance. The Safety Case should identify the current state of the structure and elements that have been subject to maintenance or repair. Any life limiting features should be determined. The Safety Case should take account of the structure's past history where relevant and justify the monitoring and inspection regime. Monitoring may be required to ensure that the structures material properties as used in seismic performance calculations are maintained for its remaining life.

Difficulties are sometimes experienced in obtaining information, e.g. drawings, material properties about the structure. Various techniques can be used to confirm structural layout and material properties, e.g. NDT, video inspection. Any ductility factor used to modify the seismic forces due to inelastic behaviour should be justified, in particular, there should be appropriate structural detailing.

The licensee is expected to make arrangements for the safety case to be peer reviewed to provide an independent overview of its adequacy.

6.3 Approaches Presently Adopted by Licensees

The licensees' continued development of the methodology for the seismic evaluation of existing nuclear structures is welcomed The multi-level seismic input approach, for example, 'bottom line plant' backed up by 'second line plant' qualified at different hazard levels, enables a judgement to be made that the risks from the plant in the event of an earthquake are tolerable. Any sensitivity analyses should provide additional confidence. An appraisal of the margins that exist in the seismic capability of the plant assessed against these events should permit an argument to be developed that the risks have been reduced to as low as is reasonably practicable. Various reasonably practicable options for improvements to the plant and structure should be considered, particularly those that enhance the seismic performance to modern standards. Any weak links in items of plant or structure which might cause failure to provide their functional requirements during an earthquake, should be identified by this process. If numerical margins are determined, these plant items may then be ranked so as to identify areas where strengthening would decrease risk most effectively. Care must be taken that comparisons between margins are meaningful, e.g. the calculations should be made on the same basis. One of the objectives of this technique is to create a balanced design.

At present, tolerability and the ALARP principle are demonstrated by deterministic engineering analysis only. Additional confidence could be obtained if seismic probabilistic risk assessments were undertaken. The risk from the seismic hazard could then be found quantitatively and compared to the risk from other classes of hazard. More effective strengthening of the safety case could be carried out as necessary.

Inevitably, the assessment of older plant requires an element of judgement, e.g. quality of information, making of the ALARP argument. When this is necessary, it is useful for the areas of judgement to be clearly indicated for the full reasoning behind the judgement made to be identified and for details of the sensitivity of the structure's performance to the judgement to be provided. From our experience in reviewing licensees' Safety Case, NII has found that discussion at an early stage in the development of the seismic safety strategy helps reduce the number of plant specific issues that often develop at a later stage.

7. FUTURE DEVELOPMENTS

The Nuclear Industry Group on Seismic Methodology established by BNFL, MEP, NEL, SN and the UK Atomic Energy Authority, reviews current issues in seismic assessment. The group considers generic seismic hazard or design matters, and part of its current programme involves the assessment of existing plant. NII keeps in regular contact with the group.

Some of the UK's nuclear research is co-ordinated under the auspices of the Health and Safety Commission and is managed by the nuclear industry, in consultation with NII, through a series of technical working groups (TWGs). The programme provides for safety issues to be raised by NII and for research to be contracted out to consultants and research establishments by the licensees. Relevant current issues include: probabilistic seismic hazard assessment, seismic performance of masonry panels, soil-structure interaction, fragility studies, inelastic seismic design, uncertainty and conservatism in seismic design.

The NII monitors developments internationally in seismic assessment. In May 1995, two American consultants were invited to the UK to bring ourselves and the major licensees up to date on progress in the USA. They visited briefly a number of nuclear installations and provided information on 'walkdowns', seismic assessment approaches and simplified seismic PRA. In August 1995, NII took an active role at the post SMiRT 13 Conference Seminar organized by the IAEA on seismic re-evaluation, where seismic assessment of existing structures was discussed. This may eventually lead to IAEA guidance on the subject. NII also participates from time to time in other international collaborations. One such project in which we are currently involved is the Seismic Shear Wall ISP (International Standard Problem) being co-ordinated by the OECD's Nuclear Energy Agency. The research involves comparison of the results from full-scale shake table tests on concrete shear walls with computer models. NII is also a supporter of the Center for Nuclear Power Plant Structures, Equipment and Piping, North Carolina State University, USA, which promotes seismic issues relevant to the operation of nuclear plant.

Internationally, a considerable amount of assessment concerning seismic performance appears to have been carried out on existing nuclear power plants using seismic PRA. For example, in the US some plants in the IPEEE programme have been assessed using seismic PRA, similar studies have also been undertaken on selected plants in Spain, Switzerland and Eastern Europe. However, the reactors are not gas-cooled and the overseas experience may not be directly applicable to the UK. In the Sizewell B PRA the plant's seismic response was included. Seismic PRA for existing plants has been found to be beneficial overseas in providing a systematic process of hazard consideration, a quantitative estimation of risk, a means of identifying weak links, and a balanced protection system. The challenge is to see if it can be usefully adapted in the UK.

8. CONCLUSION

(1) As a result of the non-prescriptive nature of the UK regulatory system, the licensees have been able to adopt a number of approaches to demonstrate the seismic safety of their existing structures. This has accommodated the wide range of types of nuclear installation which have different safety and protection requirements.

(2) The licencees are further developing their methodologies for seismic evaluation of existing plant. The Safety Cases need to clearly indicate how tolerability and ALARP requirements are met.

(3) A multi-level seismic input approach has been used by some licensees, which has facilitated judgement on the tolerability of risk and aids the extent to which risks can be considered to be reduced to as low as reasonably practicable (ALARP).

(4) There is scope for further improvement by more robustly demonstrating that the ALARP principle has been satisfied. From international studies there is justification for believing that seismic PRA could provide some assistance in this area. Also, seismic PRA may be helpful in better determining where the weak links are in a seismic safety case and providing an estimate of risk.

(5) Benefit has been gained from techniques and systems for evaluation adapted from US practice. By reviewing international practice, further initiatives may be developed.

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UPGRADING OF SEISMIC DESIGN AND INVESTIGATION ON AGEING ISSUE OF NUCLEAR POWER PLANTS IN JAPAN

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Abstract

In Japan, seismic design methodology of nuclear power plant (NPP) has been established [1]-[3]. And yet efforts have been continued to date to upgrade the methodology, because of conservative nature given to the methodology in regard to unknown phenomena and technically-limited modeling involved in design analyses. The conservative nature tends to produce excessive safety margins, and inevitably send NPP construction cost up. Moreover, excessive seismic design can increase the burden on normal plant operation, though not necessarily contributing to overall plant safety. Therefore, seismic engineering has put to many tests and simulation analyses in hopes to rationalize seismic design and enhance reliability of seismic safety of NPPs. In this paper, we firstly describe some studies on structural seismic design of NPP underway as part of Japan's effort to upgrade existing seismic design methodology. Secondly we introduce a summary of an investigation performed in Japan to investigate the effect of aging of NPP structures and equipment on the seismic safety of an NPP.

Most studies described here are carried out under the sponsorship of MITI (the Ministry of International Trade and Industry Japan), though, similar studies with the same motive are also carrying out by nuclear industries such as utilities, NPP equipment and system manufacturers and building constructors.

This paper consists of three sections, each introducing studies relating to NPP structural seismic design, upgrading of the methodology of structural design analyses and investigation to establish an evaluation methodology of aging effect on seismic safety of an NPP..

1. STUDIES ON STRUCTURAL SEISMIC DESIGN

In this section, following four studies are introduced as typical examples of ongoing studies on upgrading of NPP structures ;

(1) Model Test of Dynamic Cross Interaction Effects of Adjacent Structures,

(2) Model Test of Multi-axes Loading of RC (reinforced concrete) Shear Walls,

(3) Seismic Proving Test of Concrete Containment Vessels,

(4) Application Study on Seismic Base Isolation System to An NPP Building.

1.1. MODEL TEST OF DYNAMIC CROSS INTERACTION EFFECTS OF ADJACENT STRUCTURES

The objective of this test is to clarify the effects of structures built adjacent to an NPP reactor building, i.e., a turbine building etc., on the dynamic characteristics of the reactor building because, in

building, i.e., a turbine building etc., on the dynamic characteristics of the reactor building because, in the current seismic design analysis, the reactor building is modeled as single independent structure. Actually, many massive and heavy buildings are constructed close to a reactor building i.e., another reactor building, turbine building etc. This effect is categorized as the structure-structure interaction, and it is pointed out that this effect won't be negligible in the case buildings are massive and heavy. In order to clarify and estimate the effect, NUPEC under the authpiece of MITI, started this project in April 1994 [4],[5]. The project consists of two sub-tests, i.e., field and laboratory tests. The field test is carried out in the Higashidori site in Aomori Prefecture, located in the northern part of Honshu island of Japan. In the test, two types of adjacent structure models scaled down by about 1/10 are used. One is to examine dynamic cross interaction of the same two structures i.e., reactor buildings, and the other is to examine the interaction of different structures i.e., reactor building and turbine building. Also a single reactor building model with the same scale is constructed separately for comparison purpose.
The schematic drawing of the test site and building models is shown in Fig.1 and a picture of test models (the adjacent reactor building models) is given in Fig.2. The field test consists of shaker test and earthquake observation. These tests are carried out under two different conditions, with and

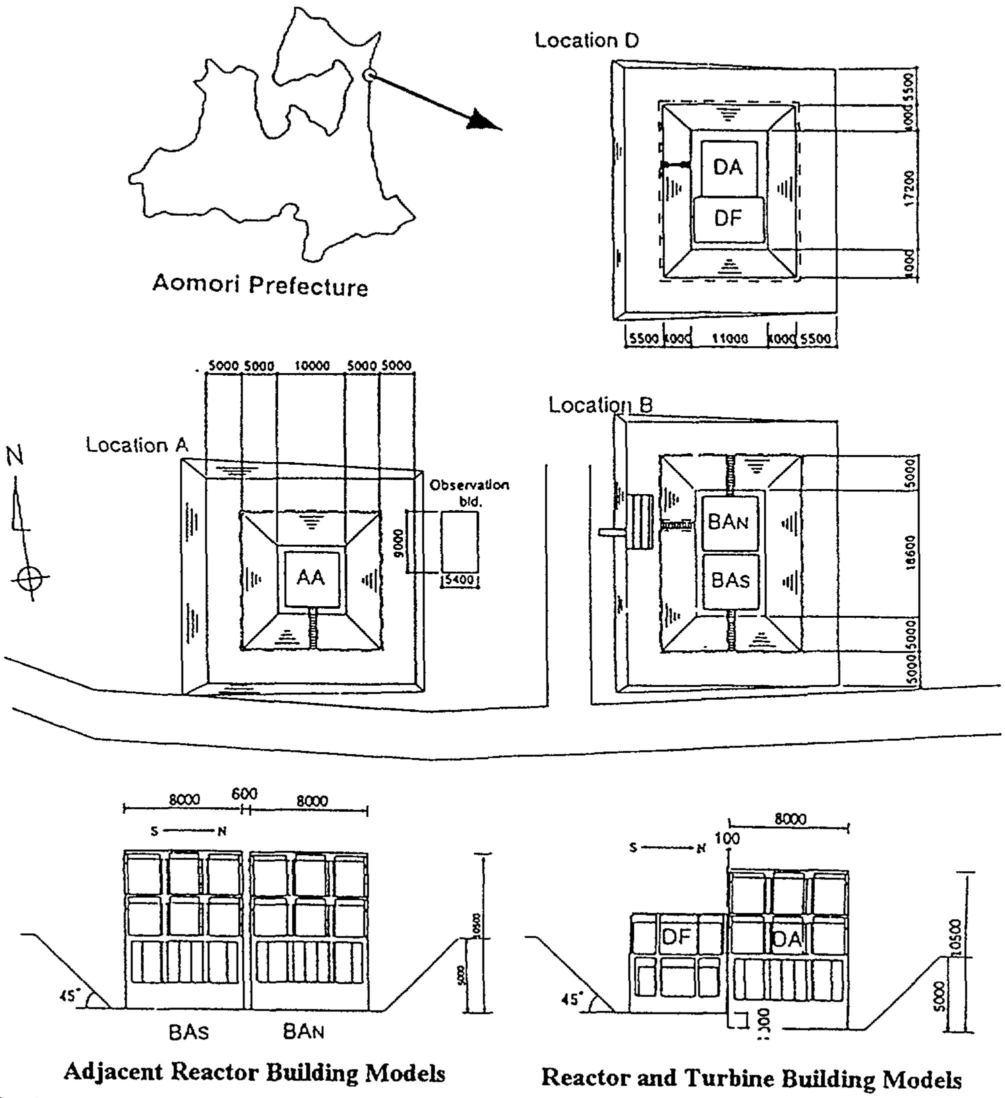


Fig.1
Test Site (Higashidori, Aomori Prefecture) and Building Models for the Field Test.

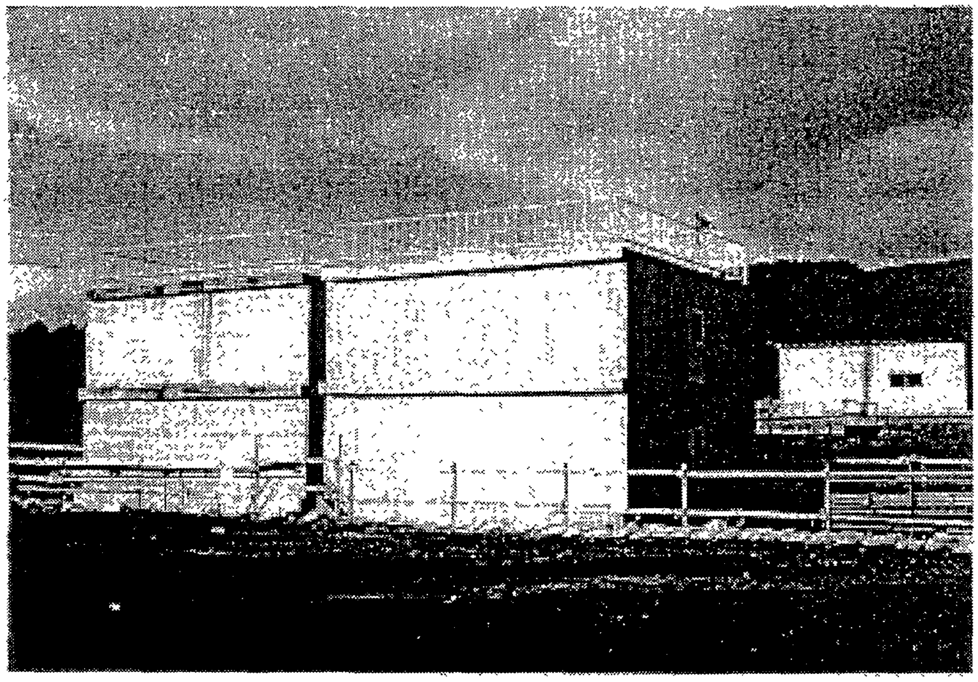


Fig.2 North-East View of the Adjacent Reactor Building Models

without embodiment of buildings. Laboratory test is planned to compensate a case of limited field test. The model consists of several small reactor building models made of aluminum, which have a 30centi-meters square cross-section in dimension and 38cm in height (1/260 scale), turbine building models with the same scale and a soil model which is made of silicon rubber (2.8m-in diameter and 1.0m-in height). The model is mounted on a shaking table and dynamic motions, including simulated design earthquake motions, are applied to study the effect of adjacent structures on earthquake response characteristics of a reactor building. The test will be completed by the end of March 2002.

1.2. MODEL TEST OF MULTI-AXES LOADING OF RC SHEAR WALLS

The cbjective of this project is to study dynamic response characteristics of NPP reactor building under three dimensional loading conditions which occur during a major earthquake. The current design analysis deals with three directional earthquake components independently. Then only the in-plane force on RC shear wall is evaluated as earthquake load. In order to know the ultimate strength of the RC shear walls, the effect of out-of-plane force on the strength should properly be evaluated as well. The motive of the study is to learn the ultimate strength of RC shear wall under three dimensional earthquake loads. From this standpoint, NUPEC started this project under the sponsorship of MITI in April 1994. The project consists of two sub-tests, i.e., static-cyclic and dynamic tests [4],[6]. The static-cyclic test includes following four sub-tests ; (1) an element test in which both shear-force and normal-force are applied to RC plates, (2) an element test in which simultaneous in-plane and out-of-plane loads are applied, (3) lateral diagonal loading test of box type shear walls and (4) simultaneous multi-axes loading test which is performed by applying simultaneous orthogonal horizontal and vertical loads and/or simultaneous orthogonal horizontal loads. The schematic drawings of the test concept are shown in Fig.3 and typical test view of the simultaneous horizontal and vertical loading test is shown in Fig.4. The dynamic loading test will be started from 1998. The test will be carried out to confirm restoring force characteristics of RC shear wall obtained by the static-cyclic loading test can stand on The test will be completed by the end of March 2004.

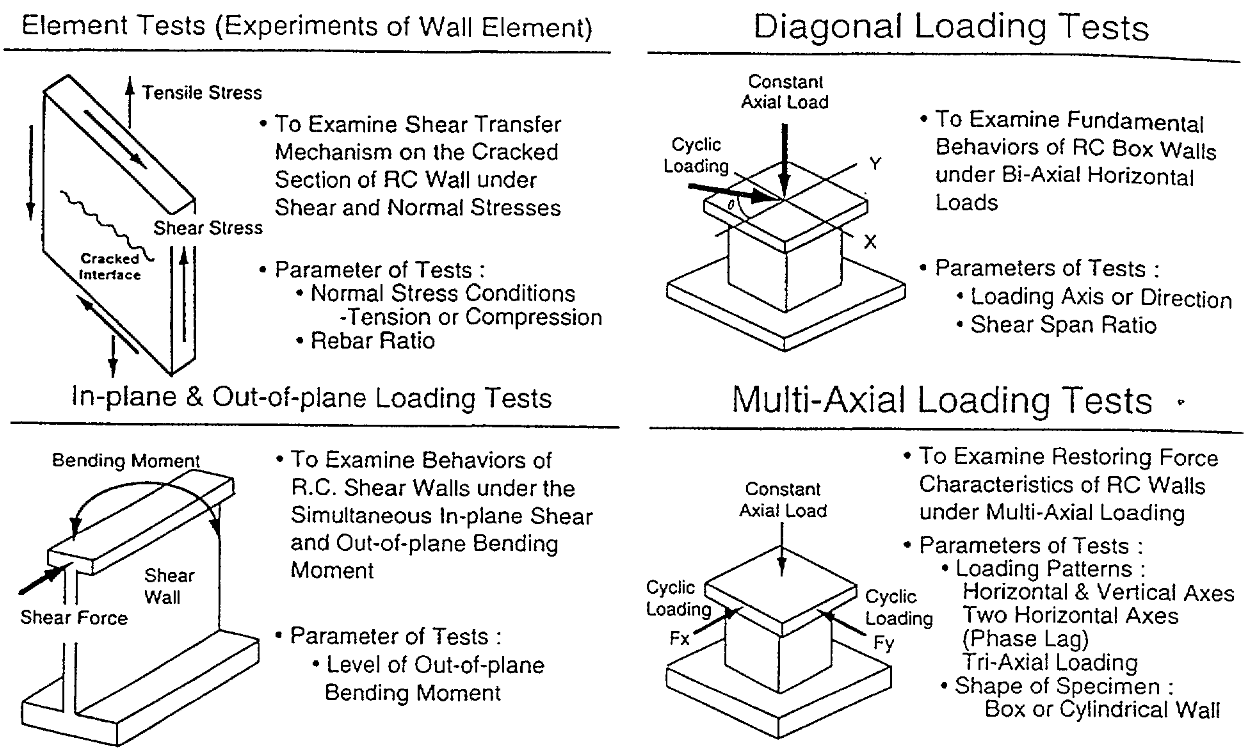


Fig.3 Basic Concept of the Static-Cyclic Test.

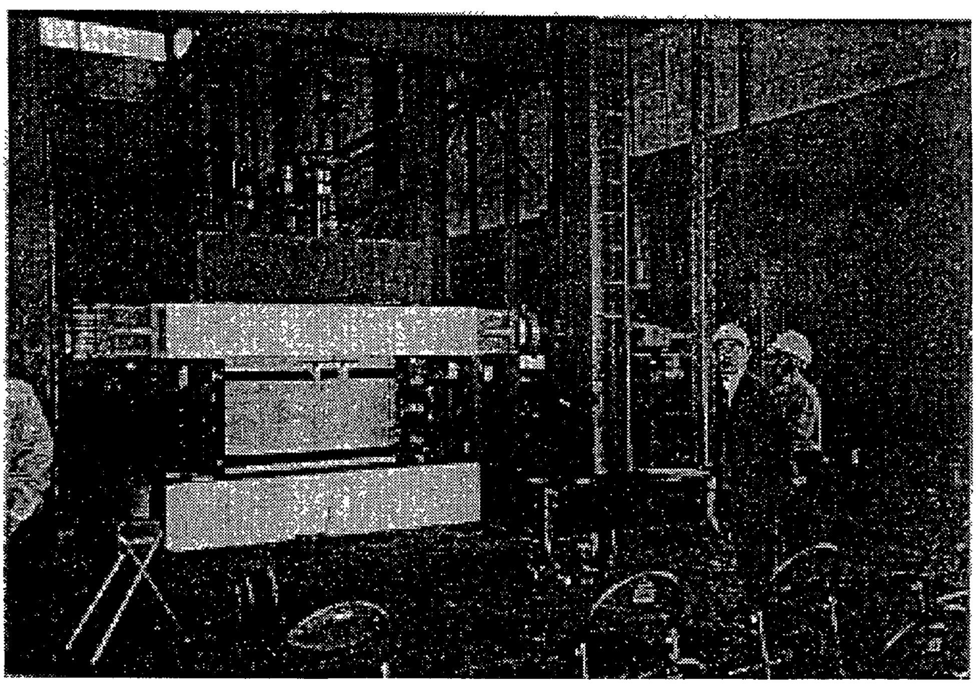


Fig.4
A Typical Test View of the Simultaneous Horizontal and Vertical Loading Test.

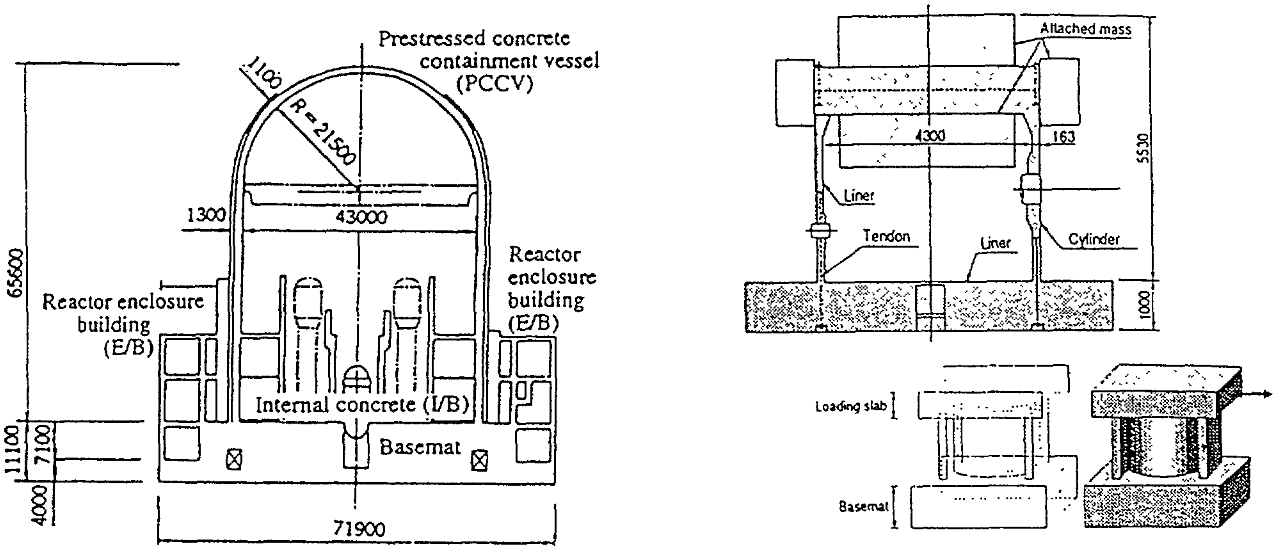
1.3. SEISMIC PROVING TEST OF CONCRETE REACTOR CONTAINMENT VESSELS

This project is planned as part of the seismic proving tests of NPP facilities which has been carried out by NUPEC using the large-scale, high-performance shaking table at Tadotsu Engineering Laboratory [7]. The project consists of two tests on concrete containment vessels, i.e., a PWR Prestressed Concrete Containment Vessel (PCCV) and a BWR Reinforced Concrete Containment Vessel (RCCV). The tests are to prove structural and functional integrity of a PCCV and a RCCV for the design earthquake S1 combined with design pressure, and for the design earthquake S2 unpressurized. In addition, the ultimate capacities of a PCCV and a RCCV to withstand earthquakes will be investigated.

1.3.1 OUTLINE OF PCCV TEST PLAN

The test model is determined through a detailed investigation given to a concrete cylinder shown in Fig.5 [8]. The scale of the PCCV model is 1/10 of 1,100MWe PWR plant in Japan. In this model concrete dome is omitted because the seismic load on this portion is not critical. Thus it is replaced by a flat concrete slab, therefore the inverted U-shaped tendons which are applied to actual PWR plant are replaced by one through vertical tendons. A total of 434tons of additional lead masses are attached above, below and circumference of the top slab, to lower the natural frequency of the model and to compensate the seismic load reduction due to application of a scale model. The main test body measures 4.63m in outer diameter and is 6.53m in height including the basemat and additional lead mass. The model has a steel liner plate inside. The liner is 1.6mm-thick which is 1/4 scale to an actual liner plate, and this thickness is the minimum fabrication limit with proper tolerance. Thus the scale of the pitch of the liner anchors is also determined as 1/4. However, the depth of liner anchor is determined as 1/8 because of short intervals of rebars and/or tendons of PCCV. The design pressure for the test model is 4.0kg/cm^2, which is the same as that of an actual PCCV. The scales of the test model are summarized in Table 1. Total weight of the test model is 760tons. A picture of the test model is shown in Fig.6 [9]. The horizontal input earthquake motions of S_1 and S_2 to be used for the test were generated to fitting the design response spectra which are determined by enveloping the design earthquake ground motions of an actual PWR plant and the results of the preceding study on the design earthquake ground motions performed under the sponsorship of MITI. The vertical earthquake ground motions for the proving test is generated by fitting vertical design response spectra which are determined by multiplying the horizontal spectra by the factor of 0.5. The factor is the maximum value of the horizontal-to-vertical spectral component conversion coefficients which are proposed as the result of the project performed by NUPEC.

The test is carryng out from February to June in 1997 which consists of following four sub-tests; (1) preliminary test to investigate the dynamic characteristics of the model at a low level of acceleration, (2) verification test on design analysis method to check for the basic response characteristics by applying sinusoidal and/or simulated earthquake ground motions, (3) proving test which confirm the structural and functional integrity for design earthquakes of S_1 and S_2, and (4) seismic margin test which is carried out to comprehend seismic margins of the CCV to the design earthquakes.



Section of Reactor Building of a PWR

Test Model

Fig.5 Outline of Actual PCCV and Scaled Test Model.

Table 1 Scale of The Test Model

	Concrete	Liner		
		Plate Thickness	Anchor Pitch	Anchor Depth
Scale	1/10	1/4	1/4	1/8

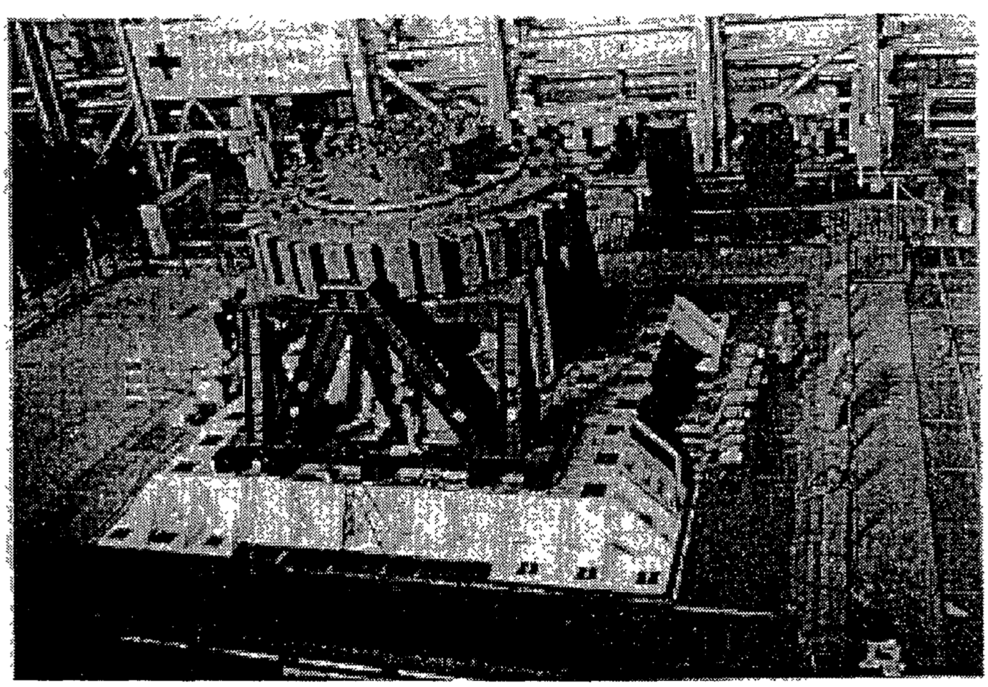
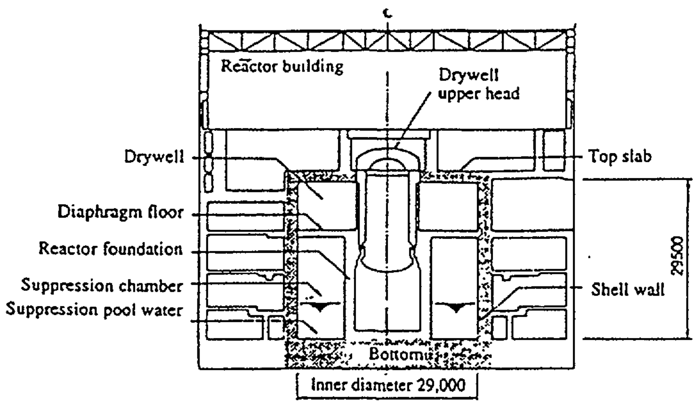


Fig.6
A Bird's-Eye View of the PCCV Test Model Installed on the Shaking Table at Tadotsu Engineering Laboratory, NUPEC.

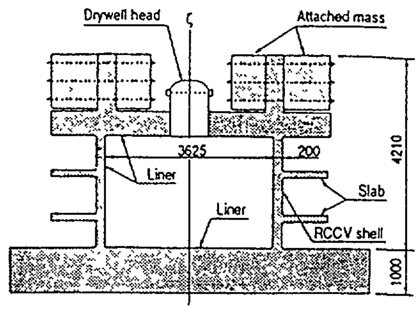
1.3.2 OUTLINE OF RCCV TEST PLAN

The aim of the RCCV proving test is to confirm dynamic characteristics, structural integrity and functional toughness against leakage of contained gaseous materials during and after the design earthquakes [9]. After the proving test, the seismic margin test will be carried out to learn dynamic behavior and seismic design margin to the design earthquakes by applying the large earthquake motions which exceed the design earthquake motions of S_2, as large as possible up to the limit of shaking table performance. The RCCV test model consists of a cylindrical shell wall, a topslab, a bottom slab, steel liner and additional lead masses. A drawing of the designed test model is shown in Fig.7 which is scaled 1/8 of an actual RCCV of a 1,350 MWe ABWR plant. As for dimensions, the test model is 5.63m in diameter (main test body) and 5.2m-high including basemat and additional lead masses. The RCCV shell wall is 20cm-thick whose scale is 1/10 of the actual RCCV. The scale of the pitch and depth of the liner anchors are determined 1/4 of those of the actual RCCV. Total weight of the model is 580tons including additional lead masses of 280tons. The additional lead masses lower the natural frequency of the model and compensate the seismic load as described in the previous section of PCCV. Fabrication of the model will be completed by the end of March 1998. Then the test will be started in April 1998 and completed by the end of July 1998. Detailed test plan is currently under examination.



Note: The shaded portion indicates the boundary of the reactor containment vessel.

Section of Reactor Building of an ABWR



Test Model

Fig.7 An Outline of Actual RCCV and a Scaled Test Model in Planning.

1.4. APPLICATION STUDY ON SEISMIC BASE ISOLATION TO NPP BUILDING

In recent years, there has been growing hopes to apply the seismic base isolation system to an NPP. Particularly, its application is expected for FBR (Fast Breeder Reactors) because reactor coolant temperature of an operating FBR is designed over 500℃ so that excessive seismic design for equipment and piping makes their thermal design difficult and that deteriorates economics of overall plant construction. Thus the particular effort to realize seismically isolated NPP was made in the feasibility study on a commercial demonstration plant of FBR [4]. Although seismically isolated buildings have been already constructed by many private construction companies in Japan, further verification studies are required with respect to the safety and integrity of the seismic isolation technology when it is applied to an NPP. From this standpoint, two major studies on seismic base isolation technology are carried out. One is a large scale project called "Verification Tests on FBR Seismic Isolation Systems". It was a MITI-founded project entrusted to CRIEPI (Central Research Institute of Electrical Power Industry) and was carried out from 1987 to 1994. In this study, the following items were investigated ;

(1) assessment on characteristics of large seismic isolation elements,

(2) assessment on dynamic vibration characteristics of seismic isolation system,

(3) assessment and study on appropriate seismic isolation structures,

(4) setting of design basis earthquake ground motions for seismic isolation NPP building,

(5) assessment on reliability of seismic isolation systems,

(6) development of seismic isolation design procedures.

As the result of this study, a draft guideline entitled "Design and Technical Guidelines on Seismic Isolation" was proposed [4]. Authorization of the draft is currently under discussion. Also the modification of the current standard earthquake design spectrum so called "The Ohsaki Spectrum" is investigated to raise the lower frequency component less than 2.0Hz by using fault rupture models of various kinds. Because the natural frequency of seismically isolated building tends to be designed around 0.5Hz. From the viewpoint of the frequency components lower than 2.0Hz, it is pointed out that the standard design spectrum proves smaller than that of recorded obtained by actual major earthquake such as Hyogoken Nanbu earthquake as shown in Fig.8 [10].

The other study is a FBR research common to electric power companies in Japan. In the study, a conceptual design of FBR as shown in Fig.9 [11] as well as an estimation of ultimate behavior of seismically isolated buildings and seismic fragility of the isolation systems are made by three-dimensional seismic response analyses designed to consider the rupture phenomenon of the isolation system. Figure 10 shows typical fragility curves and a typical rupture strength distribution of a base isolation system consisted of 367 isolators [12]. The results were obtained by 3-D (3dimensional) and/or 2-D earthquake response analyses. The study result demonstrated appreciable seismic safety margin against the design-base earthquake S_2 and proved that the rocking response of the building has a significant influence on the ultimate behavior rather than on the torsional response of the isolation systems.

Like FBR, application studies of the seismic isolation system to light water reactors have been carried out. The studies are focused on the plant construction cost reduction and the standardization of plant seismic design [13].

2. STUDIES ON UPGRADING OF SEISMIC DESIGN ANALYSES

An upgraded modeling technique usable in seismic design analyses of NPP structures is essential in streamlining seismic design. Recent remarkable progress in computer performance enables us to use sophisticated complex nonlinear analysis models for reasonable cost in curring in simulation of earthquake response of NPP structures when their site is hit by a strong earthquake. From this standpoint, many analytical studies are under way to find rational modeling for important structures. In this section, following two studies are described as example of this category;

(1) Comprehensive Applicability Studies of Test Data to Actual Plant Model,

(2) Seismic Design in Consideration of Vertical Seismic Ground Motion.

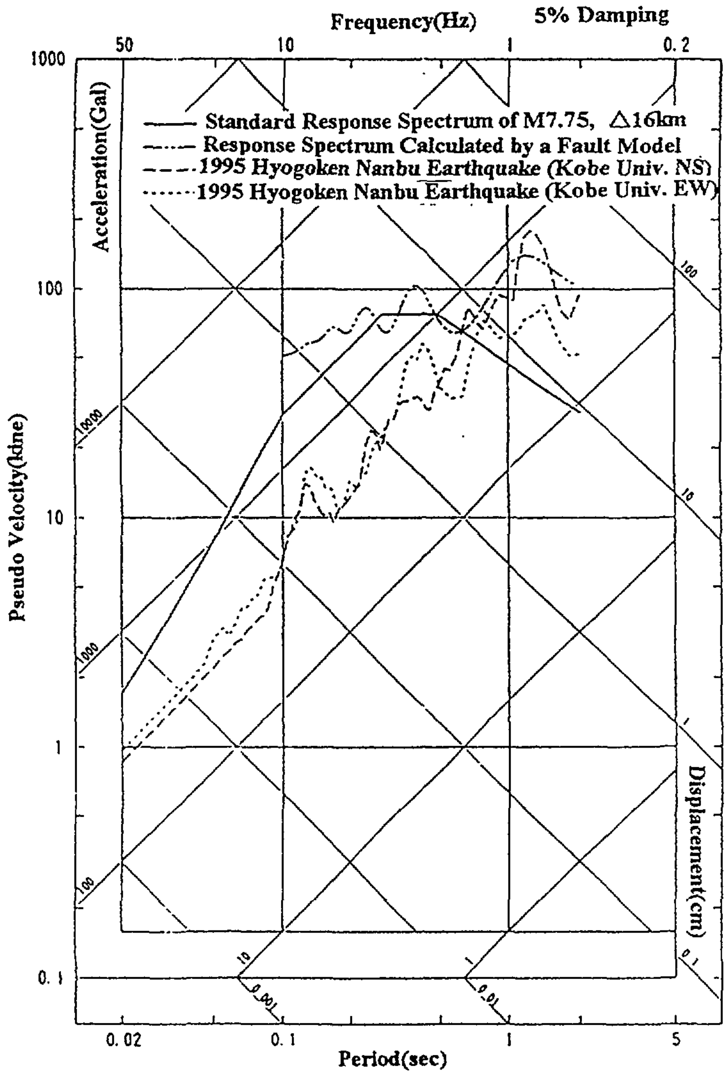
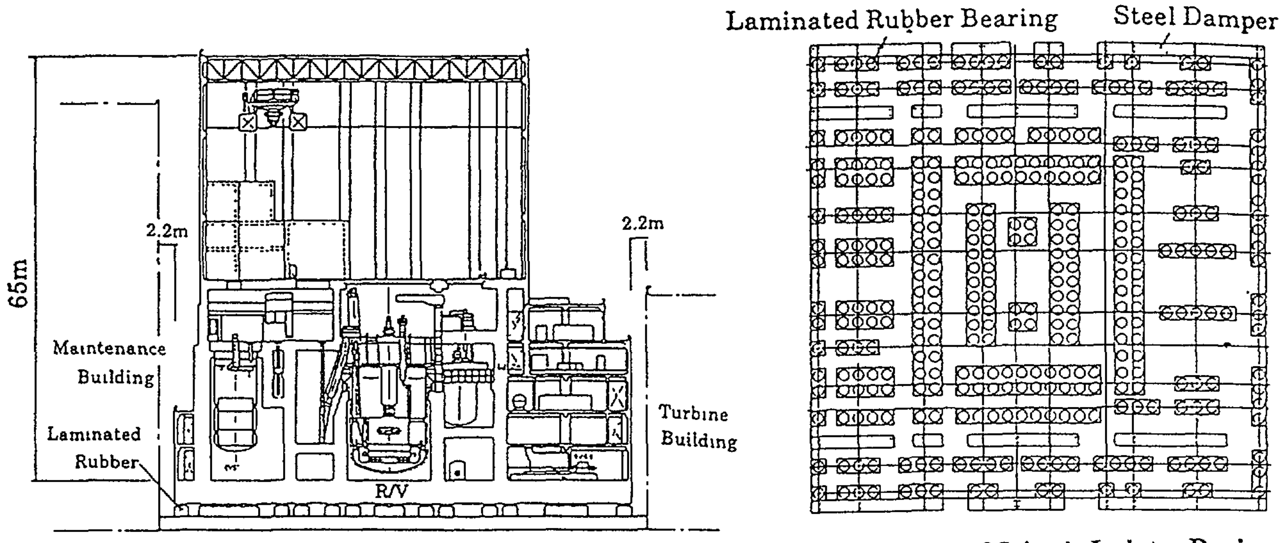


Fig.8 Comparison of a Standard Design Response Spectrum with the Response Spectra of Observed Earthquake Records and the Response Spectrum calculated using a Fault Model.

2.1.COMPREHENSIVE APPLICABILITY STUDIES OF TEST DATA TO ACTUAL PLANT MODEL

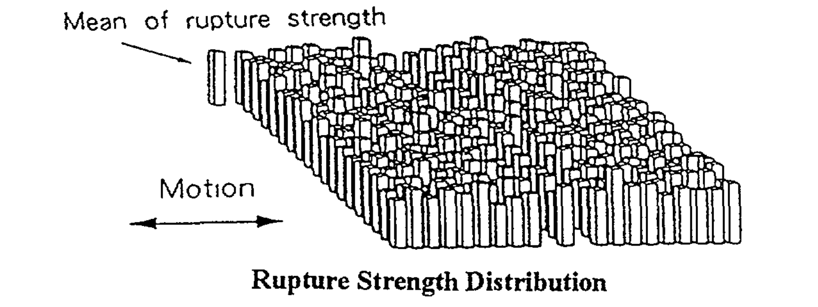
The project is designed for a comprehensive review and compilation of the test data and results obtained from preceding six tests to improve seismic safety analysis codes usable in an actual plant analytical model for the seismic design [14]. These tests are (1) model test on restoring force characteristics of reactor building, (2) model test on dynamic interaction between reactor building and soil, (3) model test on basemat uplift of reactor building, (4) model test on enbedment effect on reactor building, (5) evaluation test on inelastic seismic response of reactor buildings and (6) experimental evaluation of floor response spectra [15]. Conceptual drawings of these tests are shown in Fig.11. Also an outline of this program is shown in Fig.12. Result of this study is expected to produce a sophisticated analytical model (one of the best estimate models for the present) for seismic design analysis which contribute to upgrading of seismic design analyses. Also the test data obtained in preceding test projects will be compiled, updated and stored so that they can be referred to in the future effort to furthen upgrade plant seismic design analyses.



Section of FBR Reactor Building

Arrangement of Seismic Isolator Devices

Fig.9
Seismic Base-Isolated FBR Reactor Building and Arrangement of Seismic Isolator Devices. (Reproduced from Ref.[9])



Fragility Curves

Fig.10
Typical Fragility Curves and Rupture Strength Distribution of Seismic Isolator Devices. (Reproduced from Ref.[10])

2.2. SEISMIC DESIGN IN CONSIDERATION OF VERTICAL SEISMIC GROUND MOTION

In the current technical guideline on seismic designs of NPP, vertical seismic load is statically dealt with. However, to upgrade the seismic design methodology, a design in which dynamic seismic loads in both horizontal and vertical directions are considered simultaneously could be required first. For this purpose, the design earthquake ground motions in the vertical direction must be determined.

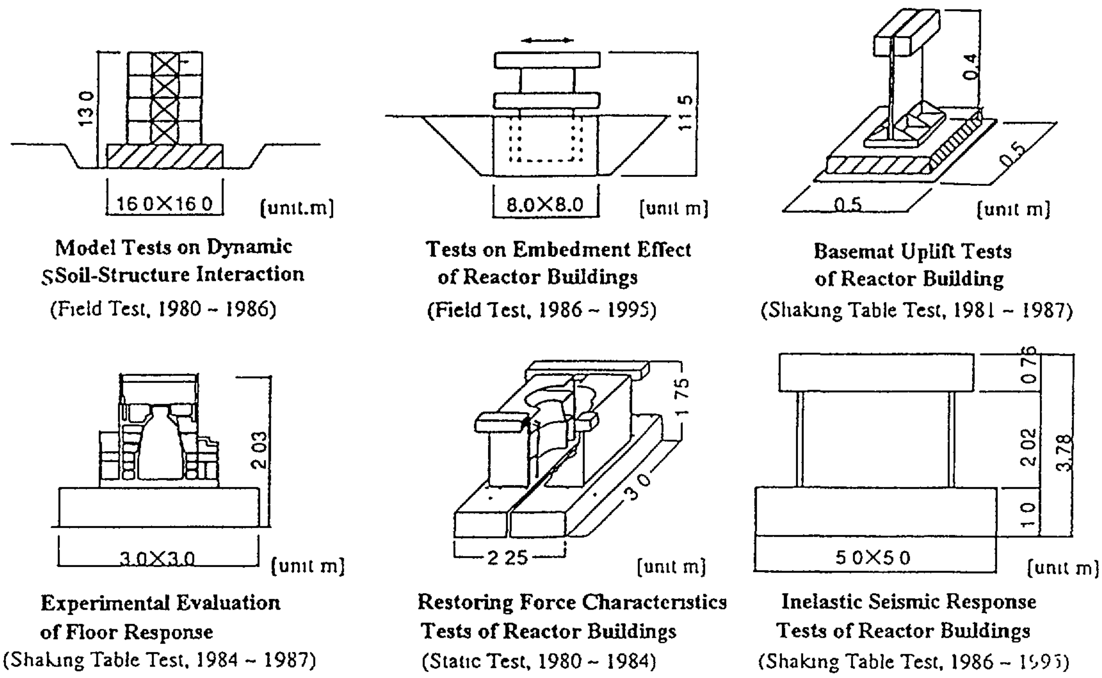


Fig.11
Basic Concept of the Preceding Tests Performed to Improve Seismic Safety Analysis Codes.

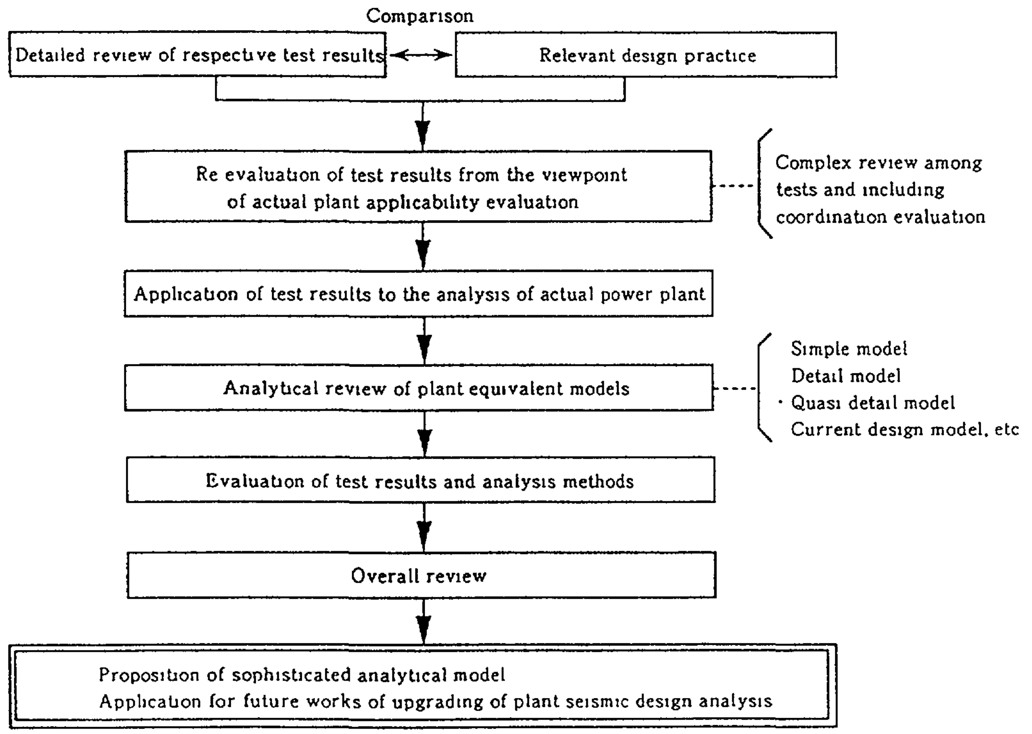


Fig.12
Outline of Comprehensive Applicability Studies of Test Data to Actual Plant Model.

The vertical design earthquake ground motion is defined in the form of response spectrum as it defined in the horizontal direction. In this project, the vertical-to-horizontal response spectral ratios (see Fig.13) of major earthquake records observed on rock field were investigated. By using the results, a conversion coefficient of horizontal-to-vertical component of design earthquake response

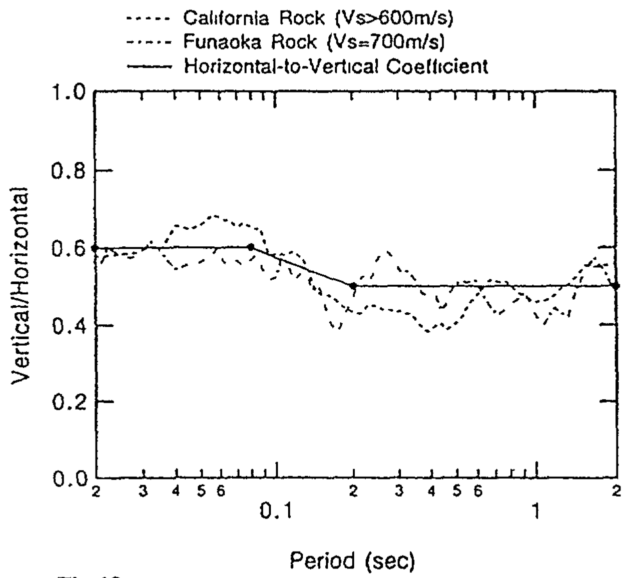


Fig.13
Comparison of the Proposed Vertical-to-Horizontal Response Spectral Ratios with those of Recorded Major Earthquakes.

spectrum was proposed [16]. At the same time, a rational synthesizing method of design earthquake motions was proposed [17]. Then to demonstrate this method, typical five design earthquake ground motions which have both horizontal and vertical components were generated. Applicability of the conversion coefficient of horizontal-to-vertical response spectral component to the earthquake record of the 1995 Hyogo-ken Nanbu Earthquake (Kobe Earthquake) and its aftershocks were investigated and confirmed enough applicability which indicates the reliability of the conversion coefficient.

The methodology and models of earthquake response analysis of structures which can simultaneously deal with both horizontal and vertical components of input earthquake ground motion were also studied. Based on the study results, prototypical analytical reactor building models for PWR and BWR plants were proposed. Figure 14 shows an example of BWR reactor building model. Then simultaneous horizontal and vertical earthquake response analyses were demonstrated by applying the design earthquake ground motions synthesized in this project.

The project will be expanded to develop the methodology which can deal with equipment earthquake response in the horizontal and vertical directions simultaneously.

Then a rational seismic design in which ground motions of both horizontal and vertical directions are considered will be proposed.

3. REEVALUATION OF EXISTING PLANTS FOR SEISMIC LOADING

The latest Japanese Examination Guide for Seismic Design of Nuclear Power Facilities was issued by NSC (Nuclear Safety Commission) in 1978 [18] and was partly revised in 1981. At that time, a total of 28 NPPs were already constructed and/or permitted to be constructed. Although those NPPs were confirmed of their seismic safety because they met the strict technical standards which were based on nearly the same concept with the Examination Guide, the NPP owner utilities made timely and careful reevaluations of the plant pursuant to the Examination Guide. The evaluation was made as part of their

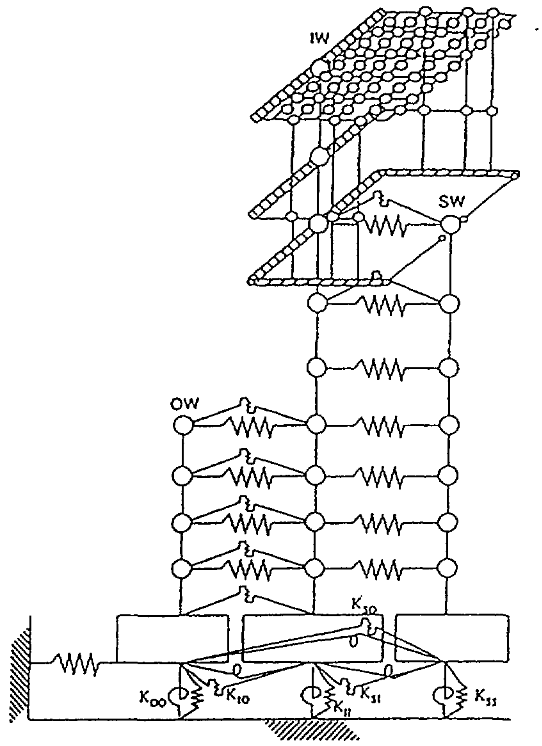


Fig.14
An Example of NPP Reactor Building Model (BWR) which can Calculate Horizontal and Vertical Earthquake Responses Simultaneously.

voluntary efforts to commit to security measures. Methodologies employed in their reevaluation studies were the same as those of current practice, expressed in the Technical Guideline for Seismic Design of NPP, JEAG-4601, issued by JEA(Japan Electric Association) [19]. Generally, every facility of Japanese NPPs has enough margin in its seismic design. These margins helped to produce favorable reevaluation results of older plants. The utilities submitted to MITI plant-by-plant calculation reports on their reevaluation results of the older plants. Assisted by a committee of NSC, MITI examined the reports and the examination was almost completed by late 1994 concluding that there would be no problem for further remodification [4],[20].

Although it is not a case of nuclear facilities, we introduce a seismic reevaluation of high pressure gas facilities including liquefied gas facilities as for a typical seismic reevaluation example in Japan. Those facilities have been under the control of MITI Notice #515, which was issued in 1981. Then we had been working for the remodification of the details of the Notice, and the new version was issued in March 1997. Those critical facilities are actually under the control of local governments. Some of them have been asking the reevaluation of the seismic design of related facilities in the case as follows ;

i) If the seismological survey shows the possibility of higher level of ground motions than the design basis ground motion specified in the MITI Notice #515,

ii) If the facility had been designed before 1981, and its design level was lower than the required design basis ground motion currently.

To back up for such requirements, Kanagawa-Prefecture have been developing techniques for reevaluation and reinforcement since 1984, and recently they issued the summary of their activity as a bounded paper volume consisted of six reports with examples. The documents related to those activities have been used as the text of the lecture by H.SHIBTA, one of the authors, in Seismic Design Course, International Program for Safety Management at NPPs which is held by Association for Japan Electric Power Information every year. This short course is planned mainly for specialists in Eastern Europe, Russia and China. Those consist of simplified elasto-plastic design and new methods for reinforcing existing facilities.

In addition to those, the evaluation of anchor-bolts is important. Some testings were done and summarizing them as design techniques in JEAG 4601-1991, "the seismic design guideline of nuclear power plants", but unfortunately, this part of JEAG-4601 has not translated into English. And in the 1997 new version Notice, we introduce L_2 ground motion in addition to L_1, the previous design basis ground motion. For L_2 ground motion, their allowable limit is similar to those for safety-related components of NPPs under the condition of level IV (or D). Again, its design procedure is a simplified elasto-plastic design. A factor, which is a function of ductility factor, has been employed, and this technique had been developed for the Building Code in Japan for 1981, and widely used for the design check of ordinary buildings.

4. INVESTIGATION TO EVALUATE AGING EFFECT ON NPP SEISMIC SAFETY

The share of power supply by NPPs in Japan amounts to 42,375MWe by August, 1997, over 30% of the total Japanese power supply. Japanese NPPs have been being kept high operation rate over 70% per year per each plant and the frequency of unexpected outage has been kept very small since 1983, one of the highest level in the world. However, the oldest three NPPs of light water reactors gradually advancing their ages, over 26years. Standing on this background, a proposal which pointing out the necessity of the investigations on countermeasure to aged NPPs was made as an urgent issue in the 1994 interim report of nuclear section of the synthetic energy investigation committee in Japan [23].

The investigations were carried out based on this proposal [24]. This investigations consist of ① analyses of NPP operating experiences (analyses of trouble experiences), ②confirmation of a meanings of the investigations on aged NPPs of their safety, ③study on technical issues for aged equipment and structures of NPPs. Figure 15 shows a relationship between a number of troubles per year and NPP operating experiences. The figure shows a trend that a number of troubles per a plant gradually decrease with the increment of operation experiences and no particular trouble increment due to aging are observed. Taking into account this trend, the sub-committee suggests that aging of NPPs don't affect their safety immediately but it is important for considerably aged NPPs to investigate the reliability of their installations. Figure 16 shows a technical flow diagram of the nuclear safety evaluation procedure for an aged NPP.

As for the investigation on the item ③, the sub-committee classified NPP equipment and structures according to their nature into the easy group for inspection, repair and replacement and the difficult group. Then for the difficult group, detailed investigations were carried out. Figure 17 shows selected equipment and structures for a PWR and a BWR by this investigation. A 60years of the plant life was supposed in this study although it did not necessarily mean the intention of plant life extension to the period. Thus three (1 PWR and 2 BWRs) NPPs were investigated of their safety-related equipment and structures with regard to their degradation due to aging in material property and a high cycle fatigue due to vibration live load and/or thermal stress applied during a long-term plant operation.

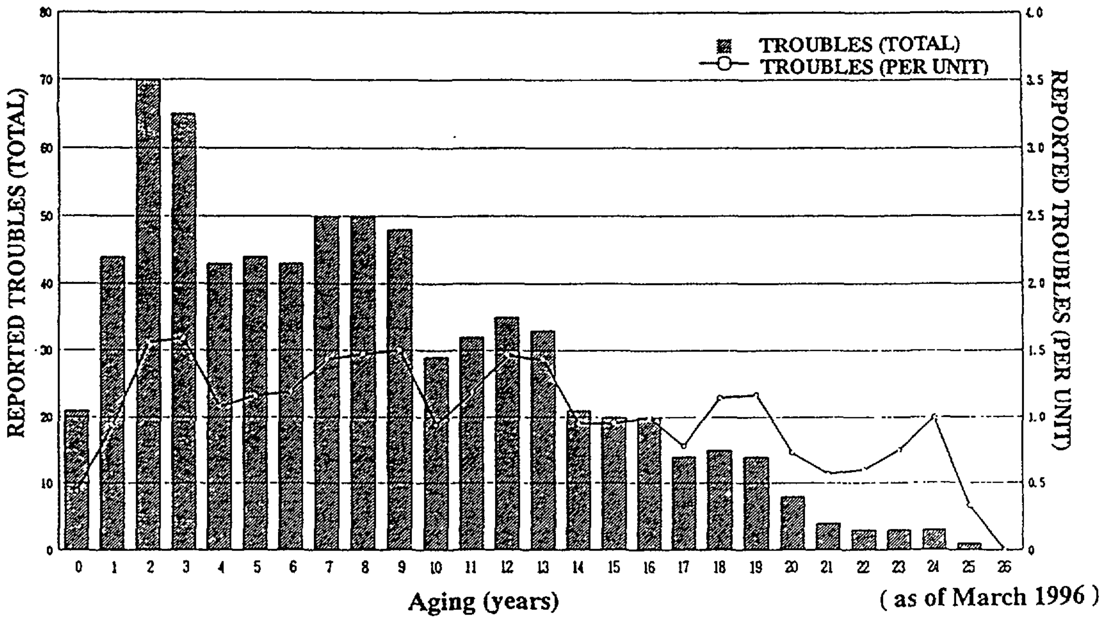


Fig.15
Transition of Annual Trouble Occurrences of Nuclear Power Plant in Japan

As the results of the studies, equipment and structures of the NPPs were confirmed to have enough safety margins against almost all aging phenomena. Moreover, action items for inspections and examinations needed to aged NPPs are extracted for future application.

Seismic safety evaluations were one of the most important items in this investigation. As for the items to be investigated for aged NPPs, followings were listed up to be a cause to deteriorate the mechanical material strength ; high cycle fatigue, neutron irradiation embrittlement, corrosion, stress corrosion crack and thermal embrittlement for metallic materials and the degradation in strength for reinforced concrete materials. It was concluded through the investigation that any severe seismic issues did not occur even for a considerably aged NPP unless it was confirmed that no explicit defect was observed during ordinary inspections and/or examination and a well organized preventive maintenance was carrying out.

5 · CONCLUDING REMARKS

Japanese practice in seismic design of an NPP has been established. The methodology and procedure was reviewed after Hyogoken Nanbu Prefecture Earthquake (Kobe Earthquake) by the Japan Nuclear Safety Commission in the midst of glowing public concern over the seismic safety of NPPs after the earthquake and confirmed that NPPs in Japan have enough structural strength and functional raggedness against given design earthquakes. And yet, a wide range of studies have been continued to date by both public agencies and nuclear industries in hope to further upgrade NPP seismic design and confirmation of seismic safety of vintage NPPs. The studies described in this paper are just a few of such efforts. These studies are indispensable for making two originally incompatible subjects compatible, i.e., to reduce plant construction cost and increase seismic safety.

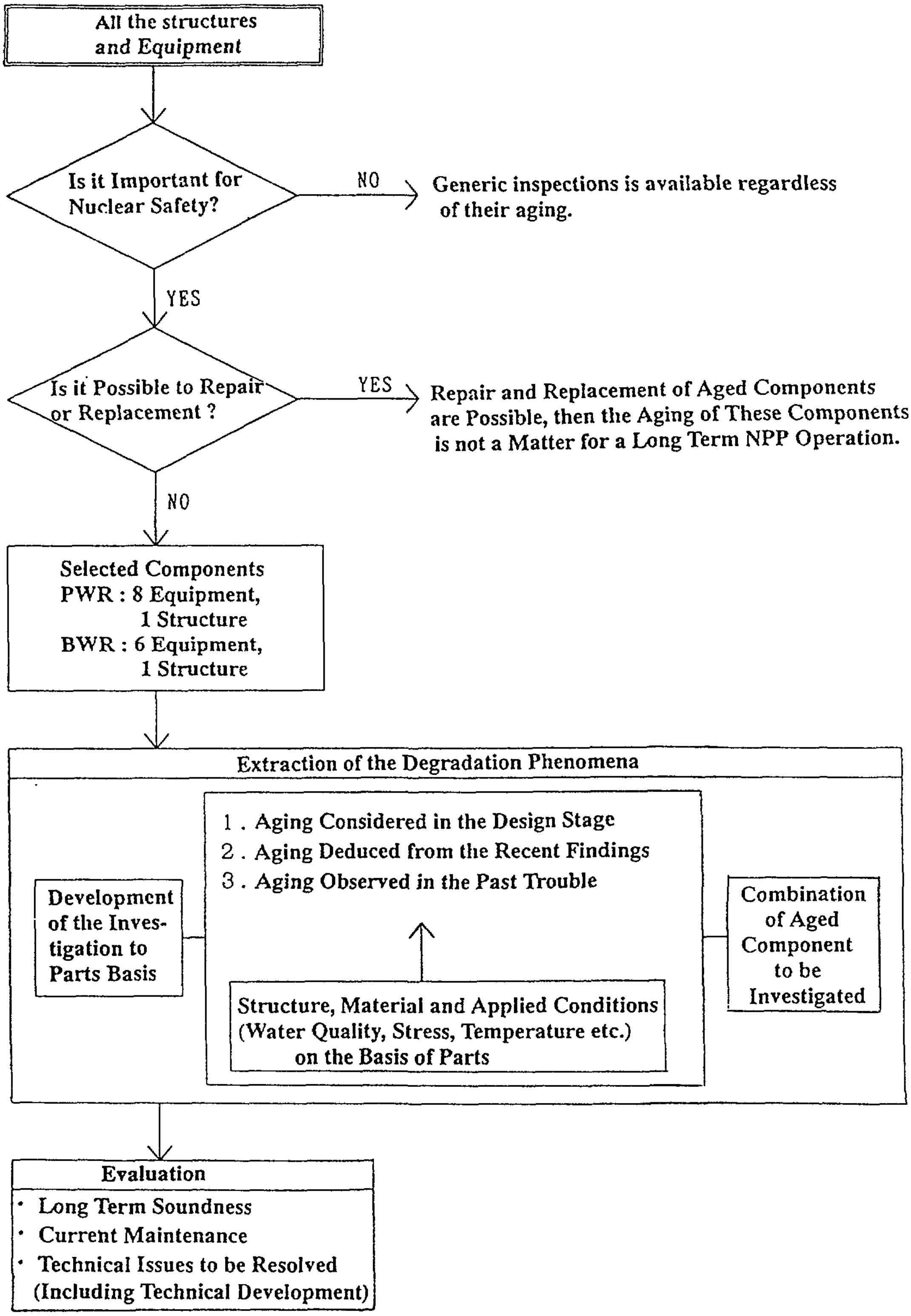
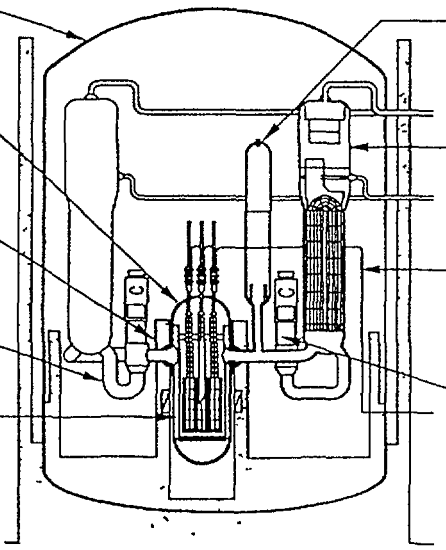
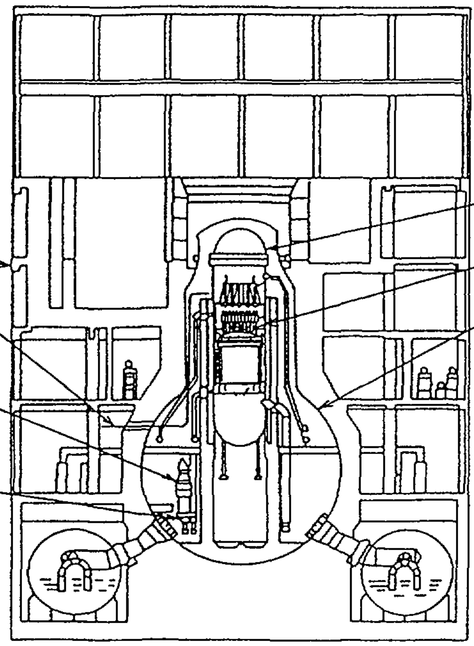


Fig.16
A Technical Procedure to Evaluate Degradation of Aged NPP Components



An Example of PWR



An Example of BWR

Fig.17 Selected Equipments and Structure for the Investigations

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SEISMIC ANALYSIS OF THE SAFETY RELATED PIPING AND PCLS OF THE WWER-440 NPP

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Abstract

This paper presents the results of seismic analysis of Safety Related Piping Systems of the typical WWER-440 NPP. The methodology of this analysis is based on WANO Terms of Reference and ASME BPVC. The different possibilities for seismic upgrading of Primary Coolant Loop System (PCLS) were considered. The first one is increasing of hydraulic snubber units and the second way is installation of limited number of High Viscous Dampers (HVD).

INTRODUCTION

One of the most important safety related systems of WWER-type NPPs are the piping providing Reactor Safe Shutdown function. Mainly these piping are located in Steam Generator (SG) and Main Cooling Pump (MCP) Boxes. On many of WWER plants these systems were designed according to former Soviet Union Standards and Rules, particularly by rather conservative PNAE Code. Nevertheless in some cases questions of seismic protection of the WWER units was out of the plant general design and criteria. That is why in the stream of the world community efforts to upgrade the nuclear safety of NPPs the great emphasis has been made for seismic reanalysis of WWER plants according to modern international practice.

This paper focuses on solving of seismic resistance problem for one of the old project of WWER-440-230 NPP. Initially in start-up period there were no any aseismic devices on PCLS and other safety related piping to withstand an earthquake and other extreme dynamic loads. The years after a number of hydraulic snubbers were installed on many of WWER units in spite of western practice to eliminate or reduce snubbers. This paper presents an accurate seismic analysis of safety related piping systems including PCLS according to modern international Standards on the base of accumulated engineering experience on other WWER NPPs.

METHODOLOGICAL BACKGRAUND FOR SEISMIC ANALYSIS

The main requirements for seismic analysis of equipment and piping of the WWER NPP are condensed in WANO developed "Terms of Reference and Technical Specification for Seismic Upgrading Design of KNPP Units 1 and 2" [1]. This document prescribes using of the Seismic Margin Assessment and ASME BPV Code, Section III approaches as methodological background [2] for seismic analysis of safety related piping located in Steam Generator and Main Cooling Pump Box. The Terms of Reference contains the following general recommendations for load combinations and allowable stress limits in seismic analysis, Table 1. The first column of this table shows the safety classes according to SRP 3.2.2 [3]. In the second column of the table are shown the load combinations (without brackets) strictly according to Terms of Reference and in brackets are pointed the load combinations in interpretation of SRP 3.9.3. The third column presents the formulas that were selected from ASME BPVC for implementation of Terms of Reference recommendations. The description of allowable stresses are shown in the fourth column of Table 1 and in the Table 2.

Table 1

Class	Load Combination	Formulas (NB-3650, NC-3650)	Allowable Stresses
1	DL+LL (P+DL+LL)	$B_1 \cdot \frac{P \cdot D_o}{2 \cdot t} + B_2 \cdot \frac{D_o}{2 \cdot I} \cdot M_i$, (9)	1.5 S_m
	DL+LL+EQi (P+DL+LL+EQi)	$B_1 \cdot \frac{P \cdot D_o}{2 \cdot t} + B_2 \cdot \frac{D_o}{2 \cdot I} \cdot M_i$, (9)	3.0 S_m
	T+EQm (P+DL+LL+EQm)	$C_1 \cdot \frac{P_o \cdot D_o}{2 \cdot t} + C_2 \cdot \frac{D_o}{2 \cdot I} \cdot M_i$, (10)	3.0 S_m
2	DL+LL (P+DL+LL)	$B_1 \cdot \frac{P \cdot D_o}{2 \cdot t_n} + B_2 \cdot \frac{M_A}{Z}$, (8)	1.5 S_h
	DL+LL+EQi (P+DL+LL+EQi)	$B_1 \cdot \frac{P_{max} \cdot D_o}{2 \cdot t_n} + B_2 \cdot \frac{M_A + M_B}{Z}$, (9)	3.0 S_h
	T+EQm	$\frac{i \cdot M_C}{Z}$, (10)	S_A

Table 2

Class	Description	Allowable Stresses
1	S_m	$\min \{S_T/3; 1.1S_T^T/3; S_Y/1.5; S_Y^T/1.5\}$
2	S_c, S_h	$\min \{S_T/4; 1.1S_T^T/4; S_Y/1.5; S_Y^T/1.5\}$
	S_A	$1.25*S_c+0.25*S_h$

Two types of seismic excitation were stipulated for analysis: Review Level Earthquake (RLE) and Local Earthquake (LE) defined in terms of Response Spectra. ZPGA level for RLE was assumed as 0.16g. For the systems which were supported on different elevation levels of structure the envelope spectra has been developed according to Appendix N of ASME Code [4]. For evaluation of seismic capacity of considered systems two analytical approaches have been used: Response Spectrum Modal Analysis Method (RSMAM) and Time History Analysis (THA). In case of TH analysis the TH acceleration was generated from target Response Spectra following to demands of Appendix N of ASME Code (N-1210). The damping ratio for all piping systems was accepted as 0.05 [1].

The load combinations and allowable stresses for seismic capacity evaluation of piping and equipment supports were also defined on the basis of [1, 2] recommendations, Table 3.

Table 3

Element of Support	Load Combination	Failure Mode	Allowable Stresses
Steel Structure	DL+LL+T DL+LL+T+EQi+EQm	Plastic Collapse	Sall 1.6 Sall; 0.7 Su [1)]
Fixed Joints	DL+LL+T DL+LL+T+EQi+EQm	Plastic Collapse	0.5 Su 0.7 Su [1)]
Welded Joints	DL+LL+T DL+LL+T+EQi+EQm	Brittle	0.3 Su 0.42 Su [1)]
Springs of Hangers	DL+LL+T DL+LL+T+EQi+EQm	Limited compression	Pmax

[1)] The level of allowable stresses is defined according to Appendix F of ASME BPVC [2].

One of the most important features of the present methodology is possibility of using of inelastic demand-capacity ratio (ductility factor) that essentially decreases the conservatism of traditional Code (ASME as well as PNAE) approaches. The following recommended values of these inelastic coefficients were implemented to current analysis [5]:

- Distribution System Supports - Fu = 1.25
- Welded Joints of Piping Supports - Fu = 1.0
- Piping - Fu = 1.5

For the following elements of distribution systems the Fu coefficients were used conventionally in terms of device's operability under seismic excitation according to supplier's catalogues [6,7]:

- Springs of Hanger support - Fu = 1.0
- Hydraulic Snubbers - Fu = 1.0
- GERB Dampers (Nseism) - Fu = 1.0 (Nseism = Nnom x 1,7)
- CVS HV Dampers - Fu = 1.0

The strength analysis (seismic capacity of structure) was defined using the above pointed coefficients by the following formulas [16]:

- Stresses (reactions) from the inertial part of seismic load (EQi):

$$S_{Eid} = \frac{S_{Ei}}{F_U} \qquad (1)$$

- Stresses (reactions) from the seismic anchor movement (EQm):

$$S_{Emd} = F_U \cdot S_{Em} \qquad (2)$$

One of the serious obstacles for providing correct analysis of running plants is gathering of necessary authentic information and input data. The only way to solve this problem is realization of walkdown procedure for each system to be analyzed for defining the real terms of equipment, piping, system and their supports installation and operating. It is quite usual that in many cases the typical WWER-type NPPs shortcoming like insufficient lateral restraining is recognized.

The present seismic analysis covers the following systems and their elements: small and large bore piping, piping supports and piping nozzles (Reactor Pressure Vessel, Steam Generator, Pressurizer).

ANALYSIS RESULTS

The full finite-element analytical model of WWER-440 piping systems located in Steam Generator and Main Cooling Pump Box is shown on figure 1.

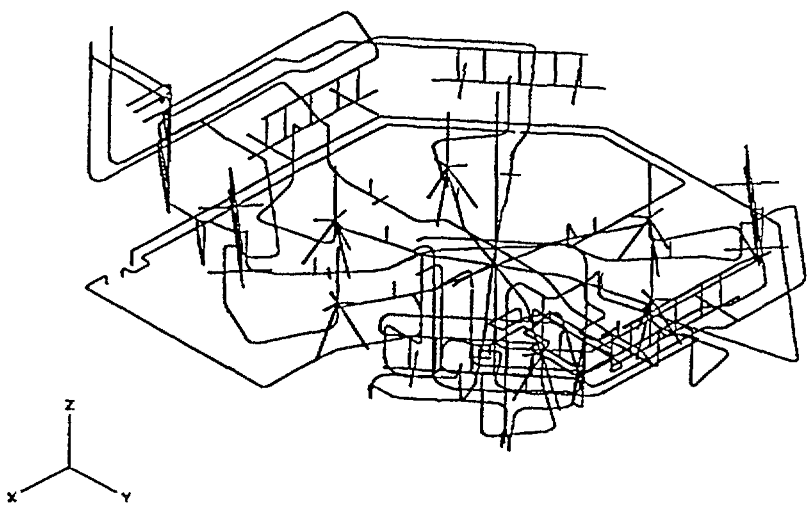

Figure 1 Complex Analytical Model of SG and MCP Box piping

This sketch includes detailed models practically of all large bore hot piping (with diameter more than 100 mm) and simplified models of Reactor Pressure Vessel, SG, MCP and connected equipment for all of six loops of PCLS.

The further consideration for more clear description of the main obtained results will be based on analysis of the first Primary Coolant Loop, Figure 2.

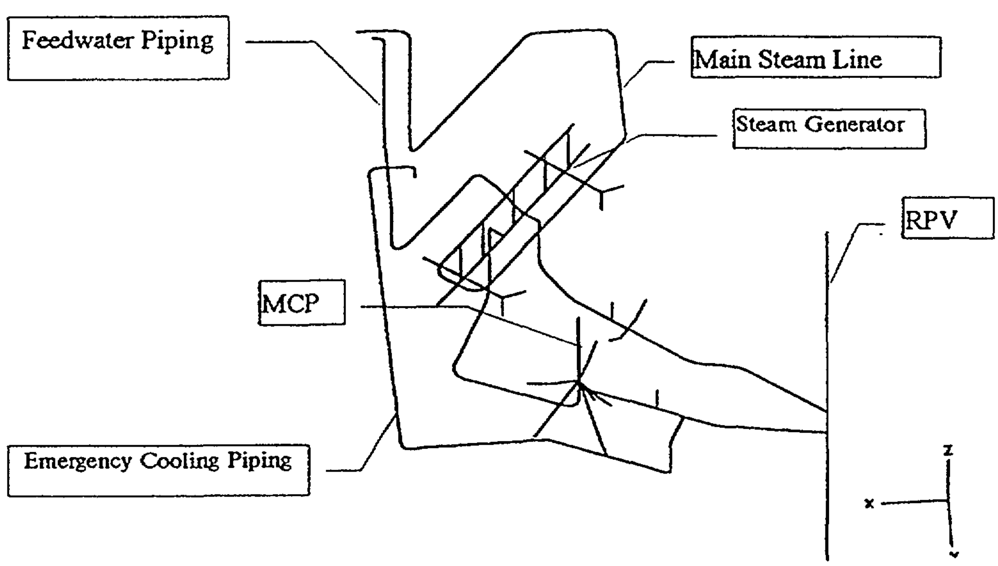

Figure 2 Analytical Model of the first PCLS

PCLS without seismic restraining (initial design)

The first natural frequency of PCLS is shown on figure 3.

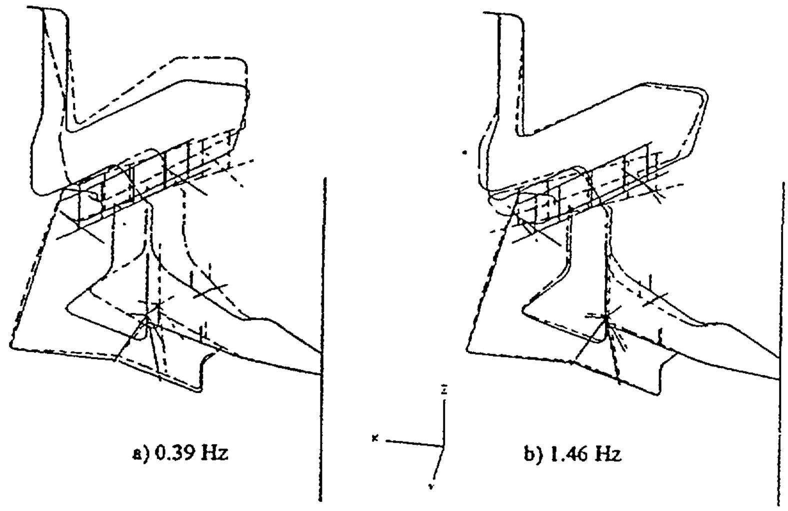

Figure 3 The first natural modes of PCLS without seismic restraining

The low level of PCLS natural frequencies leads to intensive seismic response of structure. The displacement of SG achieves more than 500 mm. Additionally the analysis of PCLS without seismic restraining shows that for many of piping elements (runs, bends and tee elements) the safety requirements are not satisfied even in case of using non-conservative ductility approach. That means that seismic upgrading of PCLS have to be performed obligatory to meet the demands earthquake protection and Terms of Reference. Thus the installation of hydraulic snubbers that was performed in eighties on a number of Ukrainian and East European WWER NPP Units is quite feasible and was in the stream of that and previous time experience.

PCLS with snubber restraining

Figure 4 demonstrates the principal location and types of hydraulic snubbers that usually are installed on PCLS.

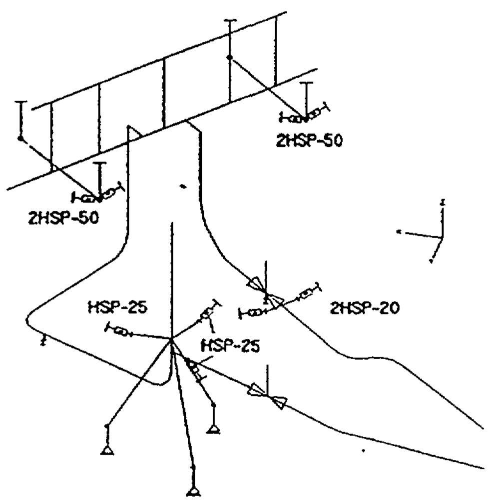

Figure 4 Snubber Location for Loop No 1 of PCLS

The accurate comprehensive non-linear TH analysis of this system has been performed to obtain the realistic dynamic response of the PCLS and snubber reactions. Dynamic characteristics of the snubbers based on their direct testing including specific velocity locking limits of the snubber's piston recommended by manufacturer [6] were involved in this analysis, Figure 5.

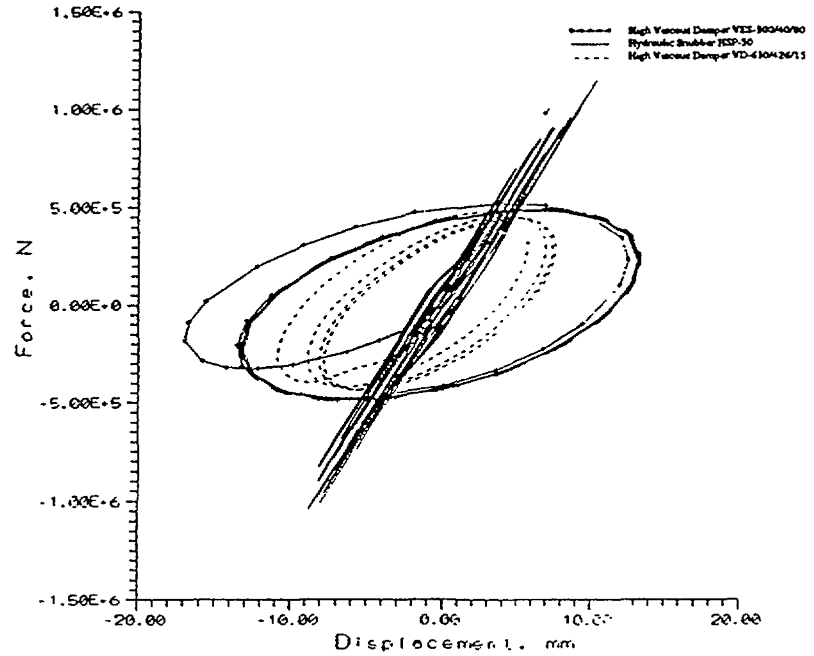

Figure 5 Dynamic characteristics of the ST Hydraulic Snubbers and High Viscous Dampers under sinusoidal 1 Hz excitation

This kind of analysis shows that there are not problems in seismic safety of PCLS and connected piping as itself. However the dynamic reaction of snubbers for some of devices exceeds the recommended capacity (limit load) of snubber for several times, Figure 6.

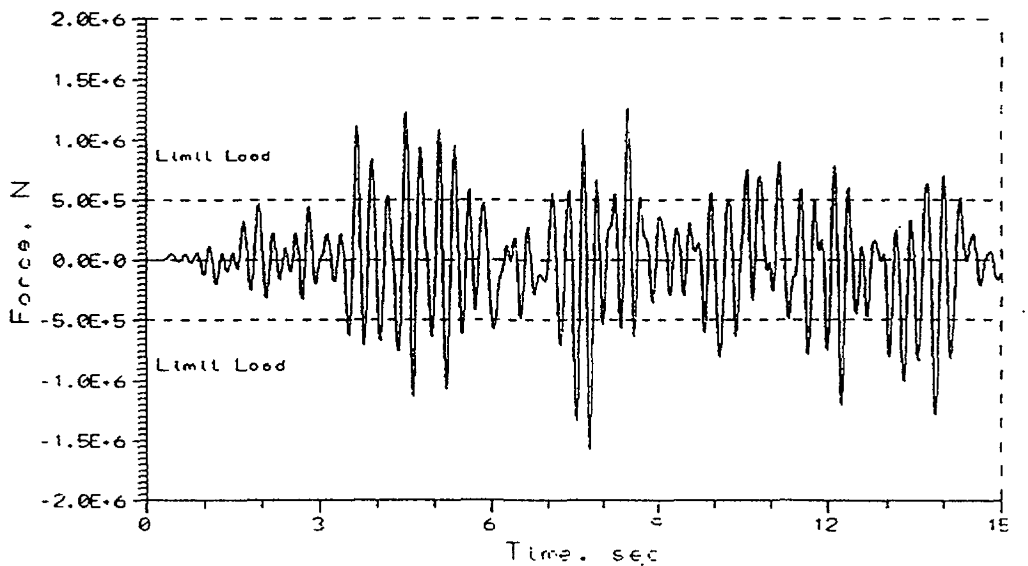

Figure 6 The TH seismic reaction force in overloaded snubber

That is why for meeting the requirements of seismic criteria the additional number of hydraulic snubbers have to be installed. The analysis shows that only double increasing of snubbers with the same load capacity under SG will solve practically the problem of PCLS seismic resistance. The reaction force of snubbers in this case do not exceed more than on 12% their nominal catalogue load capacity that seems to be acceptable. The total number of snubbers for one PC loop in this case increases from 9 to 13.

PCLS with High Viscous Dampers restraining (possible seismic upgrading)

In recent years the more reliable HVD technology has been widely implemented in seismic upgrading of WWER, PWR, BWR and other types of NPPs [7]. The dynamic characteristics, analytical model and significant advantages of these devices were investigated in literature in details [8-15], Figure 5. For purposes of this analysis the 4-parameters Maxwell Model of Viscous Damper that correctly reflects frequency-depended dynamic properties of HVD has been used [11].

Two variants of proposed location for case of HVD installation for PCLS is shown on Figure 7.

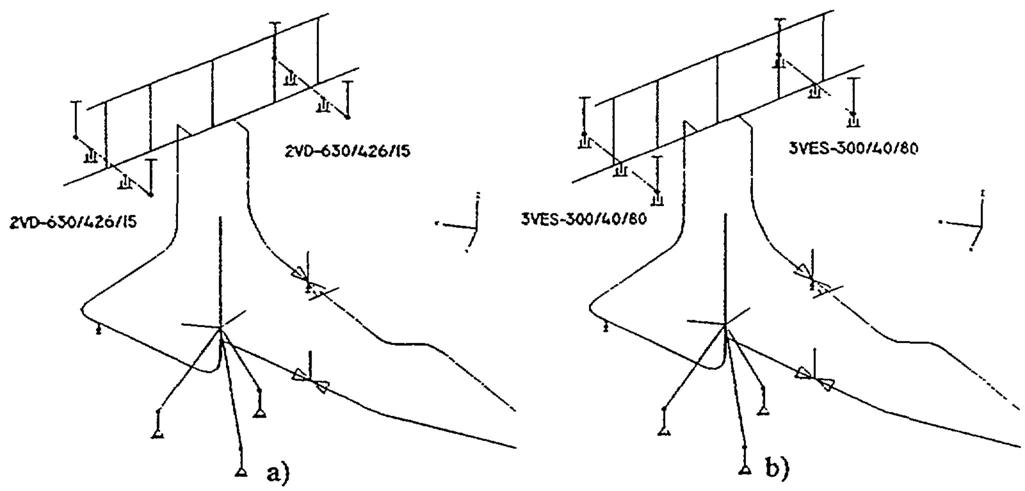

Figure 7 HVD location for Loop No 1 of PCLS

Time History Analysis of PCLS with HVD shows that four units of VD-630/426-15 is enough to provide sufficient seismic resistance of the Loop. In case of VES-300/40/80 installation this number increases up to 6 devices. In both cases stresses in piping, nozzles and supports are meet seismic criteria and requirements.

CONCLUSIONS

1. The accurate seismic analysis of WWER-440 NPP Safety Related Piping Systems and Nozzle Zones including PCLS has been performed to find out the way of possible seismic upgrading.
2. It was shown that for withstanding to earthquake with ZPGA more than 0.1g the application of special seismic devices to the WWER-440 Primary Loop is strictly recommended.
3. The analyses show that PCLS meets the seismic criteria and requirements in case of 13 snubbers versus 6 or 4 High Viscous Dampers depending on type of these devices. The additional benefit of HVD technology is high reliability of devices and low maintenance cost.

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PROPOSAL OF AN INTERNATIONAL DATABASE FOR SUPPORTING EATRHQUAKE EXPERIENCE QUALIFICATION PROCEDURES

(Session IV)

SEISMIC VERIFICATION OF MECHANICAL AND ELECTRICAL COMPONENTS INSTALLED ON WWER-TYPE NUCLEAR POWER PLANTS USING EARTHQUAKE EXPERIENCE DATA

XA0100524

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Abstract

The purpose of this presentation is to describe the modified GIP titled as GIP-VVER which can be used to verify seismic adequacy of the safe shutdown mechanical and electrical equipment and also distribution systems of operating or constructed VVER-type NPPs, namely VVER-440/213 type NPPs.

1. Introduction

Earthquake experience data was recognized in the U.S. as an efficient basis for a simplified and indirect seismic verification procedure of mechanical and electrical NPP's equipment components by the Seismic Qualification Utility Group (SQUG) about 15 years ago. SQUG collected data available from past earthquakes and seismic tests and reviewed them in detail. This review was then used to develop and establish the formal procedure titled as the Generic Implementation Procedure (GIP) [1,2] which is now widely used for verifying of safe shutdown equipment on western NPPs.

The procedure GIP-VVER has been prepared using public available information contained in GIP [3,4] and also experience taken from various seismic inspections and evaluations of VVER-type NPPs performed in the last five years.

The scope of equipment covered by the procedure GIP-VVER includes, similarly as the original GIP, the following classes of mechanical and electrical equipment:

- motor control centers,
- low and medium switchgears,
- transformers,
- horizontal and vertical pumps,
- fluid, motor, and solenoid-operated valves,
- ventilators,
- air handlers and chillers,
- air compressors,
- motor and engine generators,
- distribution panels,
- batteries on racks and battery chargers and invertors,
- instruments on racks,
- temperature sensors,
- I&C panels and cabinets.

European and particularly VVER-type relays, switches, transmitters and electric penetrations are different from those included into the original American SQUG databases. Therefore, these classes of equipment have been excluded from GIP-VVER.

In addition this procedure also includes guidelines for simplified seismic evaluation of the following classes of equipment:

- vertical and horizontal tanks,
- vertical and horizontal heat exchangers,
- cable and conduit raceway systems,
- small bore and cold large bore pipes,
- HVAC ducts,
- anchorage of equipment.

A summary of GIP-VVER equipment classes is given in Table 1.

2. General Description of GIP-VVER

As shown in Figure 1, this procedure is primarily a screening procedure. However, if safe shutdown equipment is classified as an outlier, more detail methods to verify its seismic adequacy may be used. Generally, four major steps of this procedure are as follows:

- selection of Seismic Review Team (SRT),
- identification of safe shutdown equipment,
- screening verification and walkdowns,
- outlier identification and resolution.

An engineering judgment is the major tool used by SRT during the screening verification and walkdowns to evaluate seismic adequacy of the equipment. The SRT should include system engineers, plant operation personnel, experienced and professionally trained seismic capacity engineers, and also personnel to identify and evaluate essential relays (if necessary).

The basic criteria to verify seismic adequacy of an equipment item during the screening walkdown are (see also Figure 1):

- seismic capacity greater than seismic demand (by comparison of the corresponding $ISRS_{SSE}$ or GRS_{SSE} to the Bounding Spectrum (Figure 2, Table 2),
- similarity to the equipment in the seismic experience data bases (checking of caveats, based on walkdowns and information available from documentation),
- adequate anchorage of equipment (calculations or engineering judgment, based on walkdowns and information available from documentation),
- potential seismic interactions evaluated (based on walkdowns).

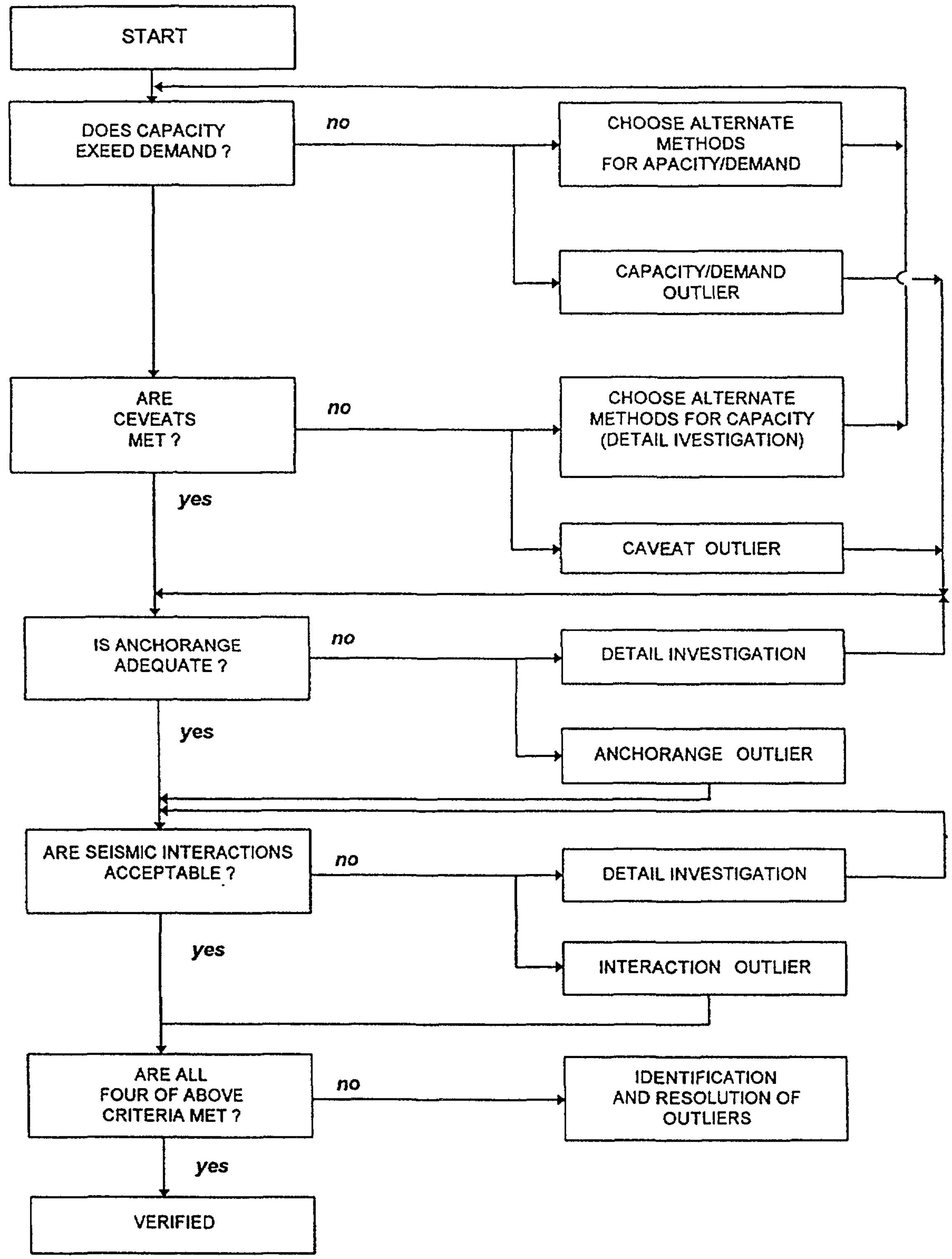
Table 1 Summary of GIP-VVER Equipment Classes

Equipment Class	Data Available for Seismic Verification
Original 20 Classes	
1. Motor Control Centers	SSRAP BS (0.33 g), GIP-VVER
2. Low Voltage Switchgears	SSRAP BS (0.33 g), GIP-VVER
3. Medium Voltage Switchgears	SSRAP BS (0.33 g), GIP-VVER
4. Transformers	SSRAP BS (0.33 g), GIP-VVER
5. Horizontal Pumps	SSRAP BS (0.50 g), GIP-VVER
6. Vertical Pumps	SSRAP BS (0.33 g), GIP-VVER
7. Fluid-Operated Valves	SSRAP BS (0.50 g), GIP-VVER
8. Motor-Operated Valves	SSRAP BS (0.50 g), GIP-VVER
9. Fans	SSRAP BS (0.33 g), GIP-VVER
10. Air Handlers	SSRAP BS (0.33 g), GIP-VVER
11. Chillers	SSRAP BS (0.50 g), GIP-VVER
12. Air Compressors	SSRAP BS (0.50 g), GIP-VVER
13. Motor Generators	SSRAP BS (0.50 g), GIP-VVER
14. Distribution Panels	SSRAP BS (0.50 g), GIP-VVER
15. Batteries on Racks	SSRAP BS (0.33 g), GIP-VVER
16. Battery Chargers and Invertors	SSRAP BS (0.33 g), GIP-VVER
17. Engine Generators	SSRAP BS (0.50 g), GIP-VVER
18. Instrument Racks	SSRAP BS (0.33 g), GIP-VVER
19. Sensor Racks (Temperature Sensors)	SSRAP BS (0.33 g), GIP-VVER
20. I&C Panels and Cabinets	SSRAP BS (0.33 g), GIP-VVER
B. Additional Classes	
21. Relays, Switches, Transmitters, Solenoids, Sensors	not applicable for VVER-type equipment
22. Electrical Penetration Assemblies	not applicable for VVER-type equipment
C. Special Approaches	
22. Cable Supporting Structures	see [4,7]
23. Tanks and Heat Exchangers	see [4,22,23]
24. Filters	only supports, anchorage and interactions
25. Pipes and HVAC Ducts	see [24,25]

Note: 1) SSRAP = Senior Seismic Review Panel [3].

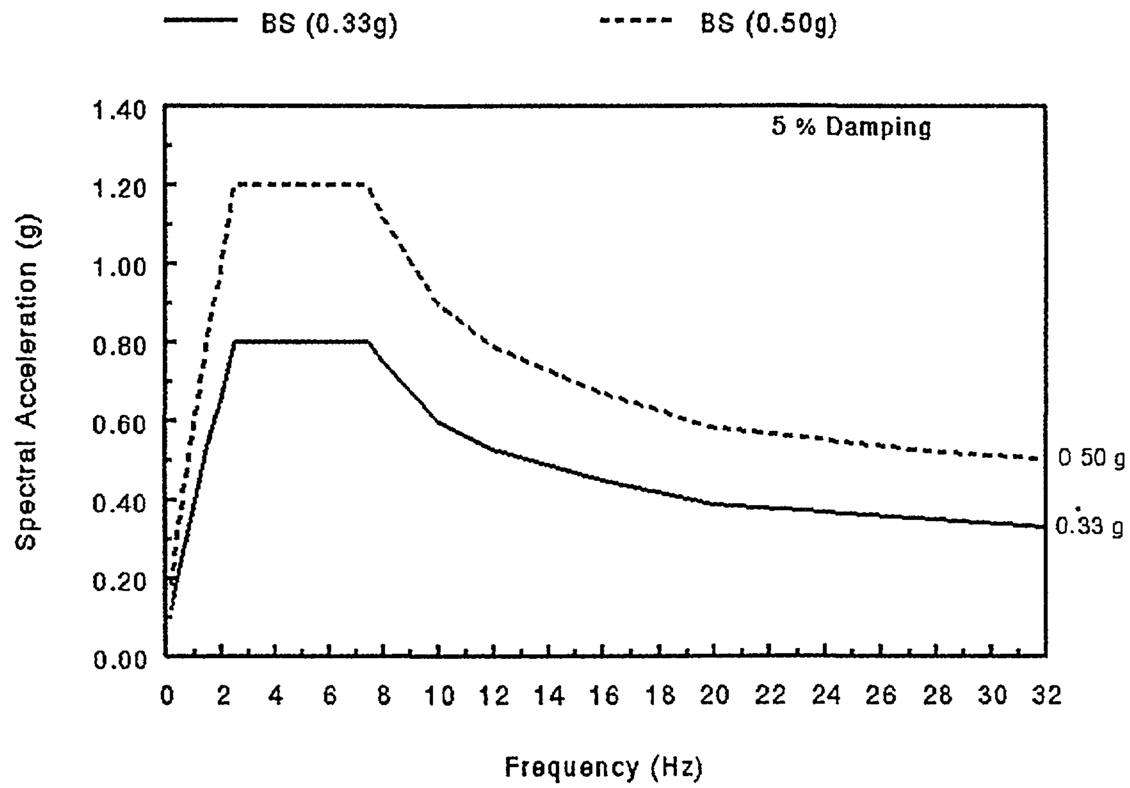
2) The document [28] gives examples of the most important seismic interactions which may occur on facilities as NPPs.

3) The document [29] is prepared for verification of anchorage of typical VVER-type NPP equipment components.



S&A-E Stevenson and Associates - Europe S&A-CZ Stevenson and Associates - Czech Republic	SCREENING VERIFICATION AND WALKDOWN PROCEDURE (GIP-VVER) Figure 1

SEISMIC CAPACITY BOUNDING SPECTRA
(GIP-VVER)



S&A-E Stevenson and Associates - Europe S&A-CZ Stevenson and Associates - Czech Republic	SEISMIC CAPACITY BOUNDING SPECTRA Figure 2

Table 2 Criteria of Comparison Seismic Capacity to Seismic Demand [1)]

A. Comparison with SL2 (SSE) Ground Response Spectra (GRS)
This can be used is mounted below about 12 m above the effective grade and when the natural frequency of equipment is greater than 8 Hz [2)] BS $\geq$ $GRS_{SL2,SSE}$ (5% damping) [3)]
B. Comparison with SL2 (SSE) In-Structure Response Spectra (ISRS)
1.5 x BS $\geq$ realistic (median, mean, best estimated) $ISRS_{SL2,SSE}$ (5% damping) [3)]

Notes:
1) Apply at least one of these two rules, which applicable.
2) Do not apply the 8Hz limit for equipment mounted on piping systems (valves, valve operators etc.).
3) These criteria shall be met for all three orthogonal spatial directions.

The Expert System called GIP-VVER [5] has been developed by Stevenson and Associates on the basis of corporation experience [6,7,8] and it can be efficiently used for practical applications directly on the plant. Two basic documents are available as results of seismic screening verification and walkdowns:

- Screening Verification Data Sheet (SVDS) in which an each equipment component or distribution line to be evaluated is identified simply by a single live item (used by the most experienced experts when all important factors relating to seismic adequacy are evidently obvious)
- Seismic Evaluation Work Sheet (SEWS) as shown in Appendix A for more detail seismic evaluation of individual equipment components or distribution lines.

There is also another sheet titled as Outlier Seismic Verification Sheet (OSVS) in which outlier issues and proposed methods of outlier resolution are described. The form of this sheet is more or less free.

3. Similarity of VVER-440/213 Type Equipment to Equipment Included in the SQUG Databases, Additional Background

Similarity of VVER-440/213 type equipment to equipment included in the original SQUG databases which is the keystone of practical application of this procedure can be estimated on the basis of already performed seismic walkdowns on the VVER-440/213 type NPPs (Perks-Hungary, Jaslovské Bohunice-Slovakia, Mochovce-Slovakia, Dukovany-Czech) as follows:

- pumps, valves	up to 100 %,
- motor control centers, switchgears	about 50 %,
- HVAC equipment	about 90 %,
- transformers	about 80 %,
- generators	up to 100 %,
- distribution panels, cabinets	about 80 %,
- batteries	about 80 %,
- relays, switchers, transmitters	low,
- cable supporting structures	about 80 %,
- tanks, heat exchangers, HVAC ducts, pipes	up to 100 %,

- anchorage details are similar with several specific exclusions.

Additional Background:

- systematic review of experience data from application of GIP to seismic evaluation and reevaluation of different NPPs [9 to 17],
- original Soviet seismic procedures and engineering documentation (f.e. OTT-82, OTT-87 [18]),
- seismic walkdowns and evaluations performed on the VVER-type NPPs in relation to the IAEA guide [19],

- IAEA sponsored Benchmark Study for the seismic analysis and testing of VVER-type NPPs [20],
- experience database of Romanian facilities subjected to the last three Vrancea earthquakes [21].

4. Practical Aspects of GIP-VVER Applications and Conclusion

Based on experience from several seismic walkdows performed during the last five years on the VVER-type NPPs, it may be concluded that the main problems related to seismic adequacy of their mechanical equipment components which may occur in some cases are:

- missing or non-proper anchorage of components, missing anchor bolts, non-proper tightening of anchor bolts,
- large seismic nozzle loads due to long unsupported attached pipes,
- large valve operator cantilever length,
- motor operated valves with remoted drivers (cardan-type connection must be evaluated),
- missing or non-properly performed pipe and duct supports,
- additional pipe restraints (f.e. application of viscous dampers for large hot pipe systems),
- replacement of brittle elements (f.e. glass level indicators etc.),
- inadequate base isolation,
- potential seismic interactions.

For electrical and I&C equipment components the main problems related to their seismic adequacy are:

- missing or non-proper anchorage of components, missing bolts, nuts and screws, non-proper tightening of anchor bolts,
- seismic functionality of relays, switches and similar items must be verified by seismic tests, performed as usually separately from the supporting cabinets or panel,
- determination of in-cabinet seismic response spectra necessary for separate verification of internal items,
- fixation of internal drawers, relays, switches, sensors and similar items to the cabinet or panel structure is often weak,
- original accumulator batteries must be replaced,
- potential seismic interaction.

<u>The GIP screening criteria should be used with caution</u>. Some equipment in VVER-type NPPs is not adequately represented in the SQUG experience database to confidently apply the GIP screening criteria without some modifications. While the most equipment components are seismically rugged, there are some unique items that have been observed during seismic walkdowns for which the screening criteria clearly are not applicable without additional engineering justification. The modified GIP-VVER procedure contains several such modifications of screening criteria.

It is anticipated that the GIP-VVER procedure will become a more or less standard procedure for verification of seismic adequacy of equipment installed on existing VVER-type NPPs.

Abbreviations

BS	Bounding Spectrum
GIP	Generic Implementation Procedure
HVAC	Heating, Ventilating and Air Conditioning
ISRS	In-Structure Response Spectrum
NPP	Nuclear Power Plant
OSVS	Outlier Seismic Verification Sheet
SEWS	Seismic Evaluation Work Sheet
SQUG	Seismic Qualification Utility Group
SSE	Safe Shutdown Earthquake
SSRAP	Senior Seismic Review and Advisory Panel
SVDS	Screening Verification Data Sheet

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APPENDIX

SAMPLES OF SVDS & SEWS AS USED IN GIP-VVER

Seismic Verification Data Sheet
(SVDS)

Equipment Class	Safety Class	Seismic Class	ID No.	Room	Elevation	Capacity vs Demand	Caveats	Anchorage	Interactions	Result

Note: Y .... Yes or Satisfactory
N .... No or Unsatisfactory
U .... Unknown
N/A .... Non Applicable

Seismic Evaluation Work Sheet
(SEWS)

Component Class : 1. Motor Control Center

Plant Name:		Unit:		Safety Class:	

PART A: DESCRIPTION

I.D.Number:		Building:	
Model No.:		Room:	
Elevation:			

PART B: CAPACITY VS DEMAND

1.	Capacity based on:	
2.	Demand based on:	

Does capacity exceed demand? ____

PART C: CAVEATS - BOUNDING SPECTRUM

1.	Earthquake experience equipment class.	
2.	Rating of 600 V or less.	
3.	Adjacent cabinets bolted together.	
4.	Attached weight of 45 kg or less.	
5.	Externally attached items rigidly connected.	
6.	General configuration similar to national standards.	
7.	Cutouts not large.	
8.	Doors/buckets secured.	
9.	Natural frequency relative to 8 Hz limit considered.	
10.	Adequate anchorage.	
11.	Potential chatter of essential relays evaluated.	
12.	No other concerns.	

Is the intent of all caveats met for Bounding Spectrum? ____

PART D: ANCHORAGE

1.	The sizes and locations of anchors have been determined.	
2.	Appropriate equipment characteristics have been determined (mass, CG, natural freq., damping, center of rotation)	
3.	The type of anchorage is covered by GIP-VVER.	
4.	The adequacy of the anchorage installation have been evaluated - weld quality and length - missing nuts and washers - expansion anchor tightness	
5.	Factors affecting anchorage capacity or margin of safety have been considered: - embedment length	

	- anchor spacing - free-edge distance - concrete strength/condition - concrete cracking	
6.	For bolted anchorages, any inadmissible gaps under the base.	
7.	Factors affecting essential relays have been considered. - gaps under the base - capacity reduction for expansion anchors	
8.	The base has adequate stiffness and the effect of prying action on anchors has been considered.	
9.	The strength of the equipment base and the load path to the CG is adequate.	
10.	The adequacy of embedded steel, grout pads or large concrete pads have been evaluated.	

Are anchorage requirement met? ______

PART E: INTERACTION EFFECTS

1.	Soft targets are free from impact by nearby equipment or structures.	
2.	If the equipment contains sensitive relays, it is free from all impact by nearby equipment or structures.	
3.	Attached lines have adequate flexibility.	
4.	Overhead equipment or distribution systems are not likely to collapse.	
5.	No other adverse concerns were found.	

Is equipment free of interaction effects? ______

<u>IS EQUIPMENT SEISMICALLY ADEQUATE?</u> ______

<u>Note:</u> Y .... Yes or Satisfactory
N .... No or Unsatisfactory
U .... Unknown
N/A .... Non Applicable

Certification: **All necessary evaluations of the equipment were made by the persons trained in accordance with GIP-VVER methodology and all information corresponds to the reality.**

Responsible for Part A: Date:
Responsible for Parts B-E: Date:

Seismic Evaluation Work Sheet (SEWS)

Component Class : 5. Pump Horizontal

Plant Name:		Unit:		Safety Class:	

PART A: DESCRIPTION

I.D.Number:		Building:	
Model No.:		Room:	
Elevation:			

PART B: CAPACITY VS DEMAND

1.	Capacity based on:	
2.	Demand based on:	

Does capacity exceed demand? ____

PART C: CAVEATS - BOUNDING SPECTRUM

1.	Earthquake experience equipment class.	
2.	Driver and driven component on rigid skid.	
3.	Thrust bearings in both axial directions.	
4.	Check of long unsupported piping.	
5.	Base vibration isolation system checked.	
6.	Sufficient slack and flexibility of attached lines.	
7.	Adequate anchorage.	
8.	Potential chatter of essential relays evaluated.	
9.	No other concerns.	

Is the intent of all caveats met for Bounding Spectrum? ____

PART D: ANCHORAGE

1.	The sizes and locations of anchors have been determined.	
2.	Appropriate equipment characteristics have been determined (mass, CG, natural freq., damping, center of rotation)	
3.	The type of anchorage is covered by GIP-VVER.	
4.	The adequacy of the anchorage installation have been evaluated - weld quality and length - missing nuts and washers - expansion anchor tightness	
5.	Factors affecting anchorage capacity or margin of safety have been considered: - embedment length - anchor spacing - free-edge distance - concrete strength/condition - concrete cracking	

6.	For bolted anchorages, any inadmissible gaps under the base.	
7.	Factors affecting essential relays have been considered. - gaps under the base - capacity reduction for expansion anchors	
8.	The base has adequate stiffness and the effect of prying action on anchors has been considered.	
9.	The strength of the equipment base and the load path to the CG is adequate.	
10.	The adequacy of embedded steel, grout pads or large concrete pads have been evaluated.	

Are anchorage requirement met? ______

PART E: INTERACTION EFFECTS

1.	Soft targets are free from impact by nearby equipment or structures.	
2.	If the equipment contains sensitive relays, it is free from all impact by nearby equipment or structures.	
3.	Attached lines have adequate flexibility.	
4.	Overhead equipment or distribution systems are not likely to collapse.	
5.	No other adverse concerns were found.	

Is equipment free of interaction effects? ______

IS EQUIPMENT SEISMICALLY ADEQUATE? ______

Note: Y .... Yes or Satisfactory
N .... No or Unsatisfactory
U .... Unknown
N/A .... Non Applicable

Certification: **All necessary evaluations of the equipment were made by the persons trained in accordance with GIP-VVER methodology and all information corresponds to the reality.**

Responsible for Part A: Date:
Responsible for Parts B-E: Date:

Component Class : 8. Valve Motor-Operated and Solenoid-Operated

Plant Name:		Unit:		Safety Class:	

PART A: DESCRIPTION

I.D.Number:		Building:	
Model No.:		Room:	
Elevation:			

PART B: CAPACITY VS DEMAND

1.	Capacity based on:	
2.	Demand based on:	

Does capacity exceed demand? ____

PART C: CAVEATS - BOUNDING SPECTRUM

1.	Earthquake experience equipment class.	
2.	Valve body not of cast iron	
3.	Valve yoke not of cast iron.	
4.	Mounted on 25 mm diameter pipe line or greater.	
5.	Valve operator cantilever length for motor-operated valves.	
6.	Actuator and yoke not independently braced.	
7.	Sufficient slack and flexibility of attached lines.	
8.	No other concerns.	

Is the intent of all caveats met for Bounding Spectrum? ____

PART D: ANCHORAGE

1.	The sizes and locations of anchors have been determined.	
2.	Appropriate equipment characteristics have been determined (mass, CG, natural freq., damping, center of rotation)	
3.	The type of anchorage is covered by GIP-VVER.	
4.	The adequacy of the anchorage installation have been evaluated - weld quality and length - missing nuts and washers - expansion anchor tightness	
5.	Factors affecting anchorage capacity or margin of safety have been considered. - embedment length - anchor spacing - free-edge distance - concrete strength/condition - concrete cracking	
6	For bolted anchorages, any inadmissible gaps under the base	

7.	Factors affecting essential relays have been considered. - gaps under the base - capacity reduction for expansion anchors	
8.	The base has adequate stiffness and the effect of prying action on anchors has been considered.	
9.	The strength of the equipment base and the load path to the CG is adequate.	
10.	The adequacy of embedded steel, grout pads or large concrete pads have been evaluated.	

Are anchorage requirement met? ____

PART E: INTERACTION EFFECTS

1.	Soft targets are free from impact by nearby equipment or structures.	
2.	If the equipment contains sensitive relays, it is free from all impact by nearby equipment or structures.	
3.	Attached lines have adequate flexibility.	
4.	Overhead equipment or distribution systems are not likely to collapse.	
5.	No other adverse concerns were found.	

Is equipment free of interaction effects? ____

IS EQUIPMENT SEISMICALLY ADEQUATE? ____

Note: Y .... Yes or Satisfactory
N .... No or Unsatisfactory
U .... Unknown
N/A .... Non Applicable

Certification: **All necessary evaluations of the equipment were made by the persons trained in accordance with GIP-VVER methodology and all information corresponds to the reality.**

Responsible for Part A: Date:
Responsible for Parts B-E: Date:

Component Class: 25. PIPING

Plant Name:		Unit:		Safety Class:	

PART A: DESCRIPTION

I.D.Number:		Building:	
Elevation:		Room:	

System Description:	

Piping System Topology:	

PART B: ACCEPTANCE CRITERIA

Material:		Joint Type:	Welded
Operating Temperature:		Operating Pressure:	
Pipe Contents:	Water	Insulation:	
Crossectional Char.:			

Are Acceptance Criteria Met? ____

PART C: CAPACITY VS DEMAND

1.	Capacity Based on:	
2.	Demand Based on:	

Does capacity exceed demand? ____

PART C: PIPING CAVEATS

1.	Piping Meets Maximum Vertical Supports Span (incl. Cantilever Segments)	
2.	Piping Meets Maximum Lateral Supports Span	
3.	Long Straight Piping Segments Axially Restrained	
4.	Seismic Anchor Movement Evaluated	
5.	Construction Adequacy	
6.	Ductile Pipe Supports	
7.	Flexible Joints Adequately Restrained	
8.	No Corrosion or Erosion	
9.	No Hard Spots	
10.	In-Line Valves Acceptable	
11.	No Other Concerns	

Is the intent of all caveats met ? ____

PART E: SUPPORT CAVEATS

1.	Ductile Anchors	
2.	No Cracks in Concrete	
3.	No Gaps Under Base Plate	
4.	Support Connection Seismically Adequate	
5.	Unidirectional Supports are Acceptable	
6.	No Other Concerns	

Is the intent of all caveats met ? ____

PART F: EQUIPMENT CONSIDERATIONS

1.	Adequate Equipment Anchorage	
2.	Adequate Nozzle Loads Capacity	
3.	Adequate Piping Flexibility	
4.	No Other Concerns	

Are Nozzles Seismically Adequate? ____

PART G: INTERACTION EFFECTS

1.	Soft Targets in Piping System Free From Impacts	
2.	Attached Lines Have Adequate Flexibility	
3.	Overhead Components or Distribution Systems Not Likely to Collapse	
4.	No Other Adverse Concerns	

Is Piping Free of Interaction Effects? ____

IS PIPING SEISMICALLY ADEQUATE? ____

Note: Y .... Yes or Satisfactory

N .... No or Unsatisfactory

U .... Unknown

N/A .... Non Applicable

All necessary evaluations of the equipment were made by the persons trained in accordance with GIP-VVER methodology and all information corresponds to the reality.

Responsible for Part A: Date:
Responsible for Parts B-G: Date:

XA0100525

PROPOSAL ON DATA COLLECTION FOR AN INTERNATIONAL EARTHQUAKE EXPERIENCE DATA

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Abstract

Earthquake expérience data was recognized as an efficient basis for verification of seismic adequacy of equipment installed on NPPs. This paper is meant to initiate a database setup in order to use the seismic experience to establish the generic seismic resistance of NPP's equipment applicable namely to the Middle and East European countries. Such earthquake experience database should be then compared to the already existing and well-known SQUG-GIP database.

To setup such an operational earthquake database will require an important amount of efforts. It must be understood that this goal may be achieved only based on a long term permanent activities an coordinated cooperation of various institutions.

1. General Considerations

There are three types of experience data which can be used:

- data collected from real earthquakes,
- data collected from already performed seismic tests,
- data collected from already performed seismic analyses.

1.1. Data Collected from Real Earthquakes

The use of experience from strong motion seismic events has growing application. It has been only within the pass ten or fifteen years that data from strong motion earthquakes have generally and systematically been collected in detail and quality necessary to provide information required for direct application to individual equipment items.

The Post-Earthquake Investigation Team (PEIT) should be setup to conduct reconnaissance and detail research investigations by a special walkdown of power and industrial facilities affected by an earthquake. The objective is to gather field experience data on equipment and supporting structures similar to those in NPPs and to study on this basis the general seismic behavior of these affected power and industrial facilities. The PEIT investigations are only the way to evaluate seismic performance of equipment in its actual installed and operational conditions. They also provide insights into seismic behavior of building structures and their effects on equipment supported by these structures.

The main attributes of this approach are:

- real earthquake motion involved,
- field mounting and anchorage conditions are typical of actual installation,

- for NPPs equipment that is also found in non-nuclear facilities the information base is large,
- equipment is subjected to realistic operational conditions, natural aging effects and actual interfaces to connecting equipment or systems.

The following unresolved issues are the most important in relation to this activity:

- PEIT status and personnel (two or three international teams each of about five or more experts, organized and working under the direct IAEA guidance and administration?)
- financial support (NPPs, European Community?),
- working method (similar to that provided in the U.S. by EPRI or modified?),
- availability of the area and facilities affected by an earthquake (using IAEA reputation?),
- availability of earthquake and post-earthquake reports or other relevant documentation (using IAEA reputation?).

The Electric Power Research Institute (EPRI) in the U.S. established an expert team of about 30 investigators of several organizations from which the PEIT is formed. When an earthquake occurs, EPRI sends the PEIT to the earthquake area immediately for a period of about one weak to identify and investigate local power and industrial facilities. If a sufficient number of facilities is found in the affected area and ground motions are high (more than 0.2 g), the PEIT visits that area one or more times for detail investigations.

Earthquake data mainly may be found from the post-earthquake reports owned by utilities. In absence of an organized program regarding to post earthquake activities it should be very difficult and time consuming to collect such information at required quality.

The data which should be collected and evaluated in relation to an investigated equipment item are as follows:

- ID number of the investigated equipment item,
- generic class [1],
- description of equipment type,
- description of the current seismic event (magnitude, epicentrum etc.)
- equipment location (place, distance from the epicentrum, type of the building structure, elevation, rank of building damage due to current seismic event),
- manufacturer the investigated equipment item,
- equipment size & weight,
- environment parameters (if available),
- post-earthquake report descriptors (number, revision, title, authors, date etc.),
- investigation date,
- description of equipment anchorage,
- description earthquake motion affected the investigated equipment item (PGA, spectra, accelerograms, whatever available from the post-earthquake report or estimated, etc.),
- description of damage (if any),
- description of functional failure (if any),
- description o observed seismic interactions,
- any other comments.

Note: 1) Proposed generic classes of equipment:

- motor control centers,
- low voltage svitchgears,
- medium voltage switchgears,
- transformers,
- horizontal pumps,
- vertical pumps,
- fluid operated valves,
- motor operated valves,
- ventilators,
- air handlers,
- chillers,
- air compressors,
- motor generators and associated equipment,
- distribution panels,
- batteries on racks,
- battery chargers and invertors,
- engine generators and associated equipment,
- instruments on racks,
- sensors on racks,
- I&C cabinets and panels,
- relays, switches, transmitters, sensors,
- instrument readouts (displays, indicators, recorders etc.)
- electrictrical penetrations,
- cable supporting structures,
- tanks,
- heat exchangers,
- filters,
- pipes (above ground),
- pipes (buried),
- HVAC ducts,
- other.

1.2. Data Collected from Already Performed Seismic Tests

The Seismic Test Investigation Team (STIT) should be setup to provide a collection of available seismic test data of NPP's equipment components. The objective is to gather seismic test data and to study on this basis the general seismic behavior of components during and after their seismic tests.

In comparison to the approach described in the previous section, the test data offer a different set of attributes:

- seismic tests involve relatively high levels of simulated input motions that are controlled, measured and documented,
- seismic test methods incorporate a number of conservative aspects,
- In-Structure Resonse Spectra (ISRS) are used as test input criteria and they must be properly enveloped by the broad-band Test Response Spectra (TRS),
- generally the Zero Period Acceleration (ZPA) of TRS is several times greater than that of ISRS,
- documented functional tests are normally included,
- failure mode information and fragility test data are available (from seismic test to failure),
- aging effects can be reproduced during some seismic test.

It is a common practice to use seismic testing for equipment whose functionality during and after an earthquake has to be assured. The most often form of seismic tests is testing on shaking tables. The equipment component to be tested is usually mounted on a programmable shaking table which provides its required base motions. When reduced scale testing is performed, similarity requirements of seismic testing must be considered.

The following unresolved issues are the most important in relation to this activity:

- STIT status and personnel (one research team of about three experts, organized with one of the well-known testing laboratories as ISMES, EUROTEST-S&A, IEEES etc. working with the general IAEA commission?)
- financial support (NPPs, European Community?),
- working method (similar to that provided in the U.S. by EPRI or modified?),
- availability and exchange of seismic test reports or other relevant documentation (using IAEA reputation?).

The data which should be extracted from the seismic test reports (by STIT or by originators of test reports) and evaluated (by STIT) in relation to an investigated equipment item are as follows:

- ID number of the tested equipment item,
- generic class,
- description of equipment type,
- manufacturer of the tested equipment item,

- manufacturer standards used,
- equipment size & weight,
- environment parameters (if available),
- seismic test organization,
- organization for which the seismic test was performed,
- test report descriptors (number, revision, title, authors, date etc.),
- test date,
- description of equipment anchorage and test mounting,
- type of the performed seismic test (according to [5]),
- description of the applied seismic input (direction, type etc.),
- functions monitored,
- acceptance criteria,
- resonant search,
- description of damage (if any),
- verification of equipment functionality (if any),
- any other comments.

It is believed that there is a lot of already performed and interesting seismic tests which may be very useful for the proposed earthquake experience database. A number of such useful results from Romania (EUROTEST-S&A) has been already extracted from available documentation and investigated by Stevenson and Coman [4].

1.3. Data Collected from Already Performed Seismic Analyses

The Seismic Analysis Investigation Team (SAIT) should be setup to provide a collection of available seismic analysis data of NPP's equipment components. The objective is to gather seismic analysis data and to study on this basis the general seismic behavior of components during and after their seismic tests.

The following unresolved issues are the most important in relation to this activity:

- SAIT status and personnel (one research team of about three experts, organized with one of the well-known analysis offices as S&A, SIEMENS, WESTINGHOUSE, STUSSI etc. working with the general IAEA commission?),
- financial support (NPPs, European Community?),
- working method (similar to that provided in the U.S. or modified?),
- availability and exchange of seismic analysis reports or other relevant documentation (using IAEA reputation?).

The data which should be extracted from the seismic analysis reports (by SAIT or by originators of analysis reports) and evaluated (by SAIT) in relation to an investigated equipment item are as follows:

- ID number of the analyzed equipment item,
- generic class,
- description of equipment type,
- manufacturer of the tested equipment item,

- manufacturer standards used,
- equipment size & weight,
- environment parameters (if available),
- seismic analysis organization,
- organization for which the seismic analysis was performed,
- analysis report descriptors (number, revision, title, authors, date etc.),
- description of equipment anchorage,
- type of the performed seismic analysis,
- description of the used seismic input ,
- acceptance criteria,
- results of the analysis,
- any other comments.

Seismic qualification by analysis is generally applied to such items as heavy passive mechanical components, distribution systems, and civil structures. As stated in [4], due to well known uncertainty in analysis, this approach should be considered as only supporting information in earthquake experience data process, namely when the equipment is component is qualified by combined analysis and testing. It is believed that there is a lot of already performed and interesting seismic analyses which may be very useful for the proposed earthquake experience database.

From the other side, there is a good chance to receive some information also about seismic resistance of NPP's civil engineering structures which are usually analyzed, not tested.

1.4. Data Evaluation and Research Coordination

The data evaluation process generally should consist of the following main steps:

- obtain collected or extracted data,
- review data for suitability and completeness,
- enter data in the database and store on the disk,
- evaluate seismic capacity spectra related to an each equipment class, compare them to the SQUG bounding spectrum and available GERS spectra [2],
- determine caveats (inclusion and exclusion rules) related an each equipment class and compare them to the corresponding SQUG caveats [3],
- perform guidelines for evaluation of seismic interactions and anchorage of equipment typically occurred on European NPPs.

<u>Research coordination of all three activities described above seems to be extremely important and should be, perhaps, performed by IAEA.</u>

2. Conclusion

This paper presents the first proposal on data collection for an international earthquake experience data. Also data evaluation process is generally described. The paper is meant to initiate discussion and then a database setup in order to use the seismic experience to establish the generic seismic resistance of NPP's equipment applicable namely to the European countries. The future of this process will strongly depend on creation of the corresponding coordinated international program. Unresolved issues which are the most important in relation to these activities are also outlined in this paper.

Once the earthquake experience database will become operational, the first and most important benefit will be the reduction of efforts related to seismic evaluation and reevaluation of NPP's mechanical and electrical equipment, and namely that which is installed on VVER-type NPPs.

Abbreviations

GERS	Generic Equipment Ruggedness Spectra
GIP	Generic Implementation Procedure
EPRI	Electric Power Research Institute
IAEA	International Atomic Energy Agency
IEEES	Institute of Earthquake Engineering and Engineering Seismology
ISRS	In-Structure Response Spectrum
NPP	Nuclear Power Plant
PGA	Peak Ground Acceleration
PEIT	Post-Earthquake Investigation Team
SAIT	Seismic Analysis Investigation Team
SQUG	Seismic Qualification Utility Group
STIT	Seismic Testing Investigation Team
S&A	Stevenson and Associates
TRS	Test Response Spectrum
ZPA	Zero Period Acceleration

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CONSIDERATIONS ON EMERGENCY PREPAREDNESS DURING/AFTER EARTHQUAKE OCCURENCE

(Session V)

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PROCEDURES AND ACTIONS PROPOSED IN BELGIUM TO IMPROVE THE PREPAREDNESS OF THE NUCLEAR POWER PLANTS IN CASE OF EARTHQUAKES

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Abstract

In Belgium, an evaluation has been made on the actions and decisions to be taken after the occurence of an earthquake which is felt in a nuclear power plant.
Initially, the procedures recommended to stop the plant if the earthquake was above a certain level (OBE or S2 earthquake) or if some damage due to the earthquake were observed.
No more details were given on the level of damage and its influence on the safety of the plant, as well as on the damaging potential of the earthquake. No indications were given to the operator on the type and the most likely location of the damage that he could observe after an earthquake.
No priorities were given to the operator on the specific immediate actions to be taken in case of earthquake, in addition to the measures dictated by the safety rules.
Moreover, if the decision was taken to stop the plant, no instructions were given about a verification of the readiness of the plant for a shut-down.
No criteria were given to the operator to allow him to restart the plant when it had been stopped, in case of damage not affecting the safety of the plant.
To answer these questions, it was decided to adapt the EPRI recommendations to the Belgian nuclear practice.
This paper describes the procedures that were recommended by the authors to the Belgian utility for the immediate and restart actions after an earthquake, as well as the long term evaluations to be made to give the assurance that the plant is ready again to sustain a SSE or S1 earthquake; the authors have supplemented these procedures by a walk-down and a screening of the files of seismic evaluation of the equipment in order to set up a list of the most earthquake sensitive representative equipment and the associated locations.
To enable the implementation of these procedures, it was necessary to replace the obsolete seismic instrumentation as well as to propose more realistic criteria to decide whether the observed earthquake was more severe than the reference earthquake (OBE or S2 earthquake). One describes how the instrumentation has been already modified in Tihange and the proposed criteria.

1. INTRODUCTION

If a large earthquake hits a nuclear power plant, it is most likely that process alarms and safeguard systems will initiate a shut-down. However, when the earthquake is of a lower amplitude, the decision to shut down the plant will not be taken by some automatic device but by the operator of the plant
To help him take a fast and correct decision, it is necessary to give him some tools and criteria when he is facing an event that is very unusual in countries of low to moderate seismicity like Belgium. Adequate seismic recording instruments and alarms must help him evaluate the level of the earthquake.

Because of the precautions taken to design and build the nuclear power plants and because of the margins used to define the site seismicity, the probability of observing important damage to the plant due to the earthquake is very low.

Even if the plant is shut down when the earthquake exceeds the OBE level or if some damage occurred, it is very likely that the plant will be able to resume operations very quickly after the earthquake.
The loss of the power after shut-down of a nuclear power plant can have important consequences on the life of the population and on the economy of a country. It is thus very important to describe how to evaluate the extent of the damage on the plant due to the earthquake and to define some tests and inspections to be made as well as the criteria to be satisfied in order to allow a fast and safe restart of the plant.

Finally, even if the plant has resumed operation, it is necessary to define how to check and reassess the seismic adequacy of the equipment and components necessary for the safety of the nuclear power plant, in order to confirm that the plant will safely sustain another earthquake.

2. CHARACTERISTICS OF THE NUCLEAR POWER PLANTS AND OF THE SEISMICITY IN BELGIUM

Seven nuclear power plants have been commissioned in Belgium (fig 1.) on two separate sites :
- the site of Doel on the Scheldt river in the North West of Belgium. Four PWR units have been built on this site; the first one started operation in 1973 and was not designed against

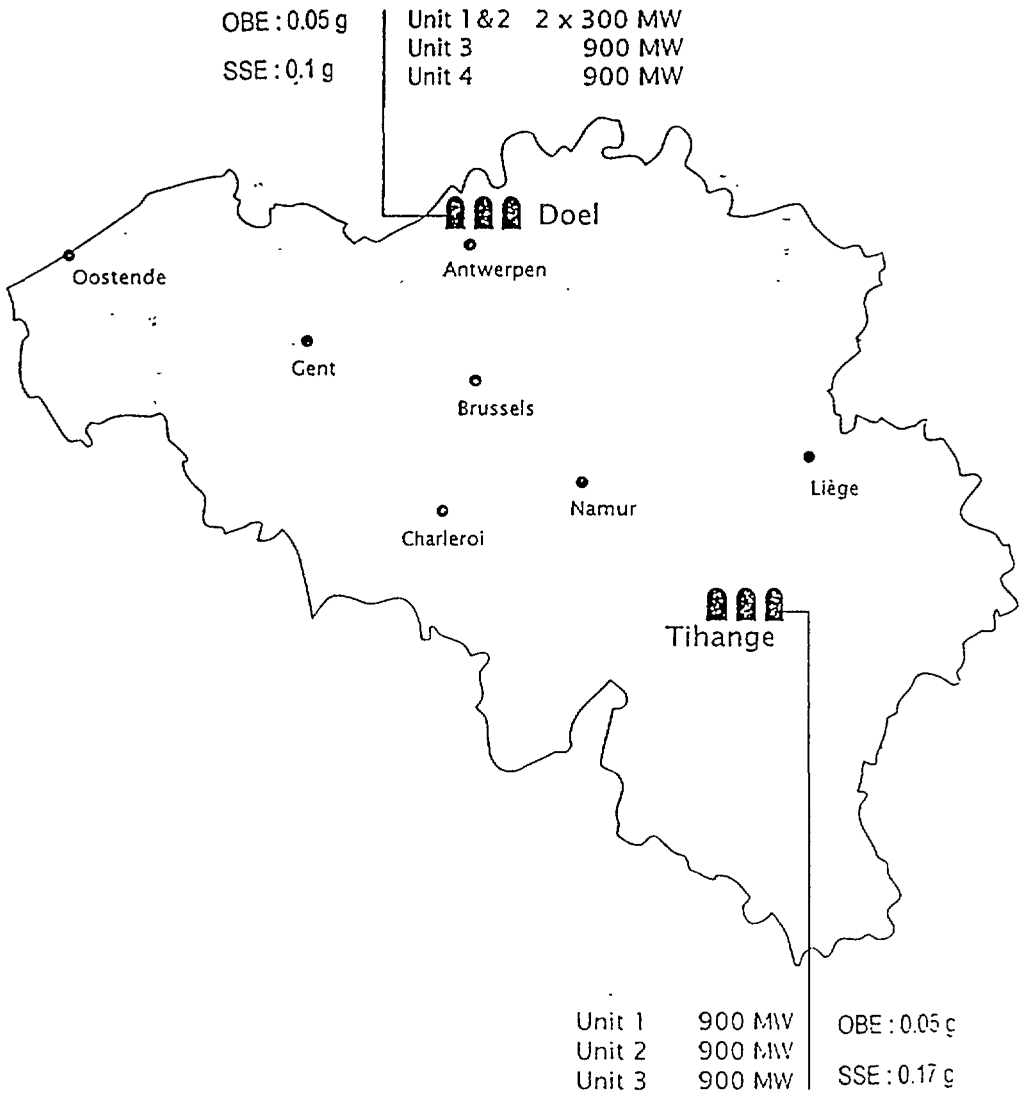

Figure 1 - Nuclear Plants in Belgium

earthquakes. The last ones started operation in 1983 and1985. They were designed against OBE (0,05 g) and SSE (0,10 g).
- the site of Tihange on the Meuse river in the Eastern part of Belgium where three similar PWR nuclear power plants were commissioned from 1975 to 1985. They too were designed against OBE (0,05 g) and SSE (0,10 g).

Belgium is a country of low to moderate seismicity. The SSE was first evaluated for both sites to a maximum horizontal acceleration of 0.10 g with an associated OBE of 0.05 g. Later on, at the Tihange site, the maximum SSE horizontal reference acceleration was raised to 0.17 g with an OBE kept to a maximum of 0.05g.

3. SITUATION BEFORE THE IMPLEMENTATION OF THE NEW PROCEDURES

3.1 *Original seismic instrumentation*

One unit per site has been instrumented according to the requirements of R.G. 1.12 [1] and ANSI 18.5 [2] (see fig. 2):

- At the base of the reactor building : one 3D (three directional) triggering unit, one 3D peak accelerometer and one 3D seismic spectrum recorder, which are connected to an alarm unit located in the control room.
- Two triaxial accelerometers, one at the base of the containment, the second at the top of the cylinder of the containment; a third set of accelerometers has been placed in the free field for the site of DOEL only.
- Peak acceleration and peak response spectrum recorders on the internal structures of the reactor building and on some primary equipment or piping support of the reactor building.

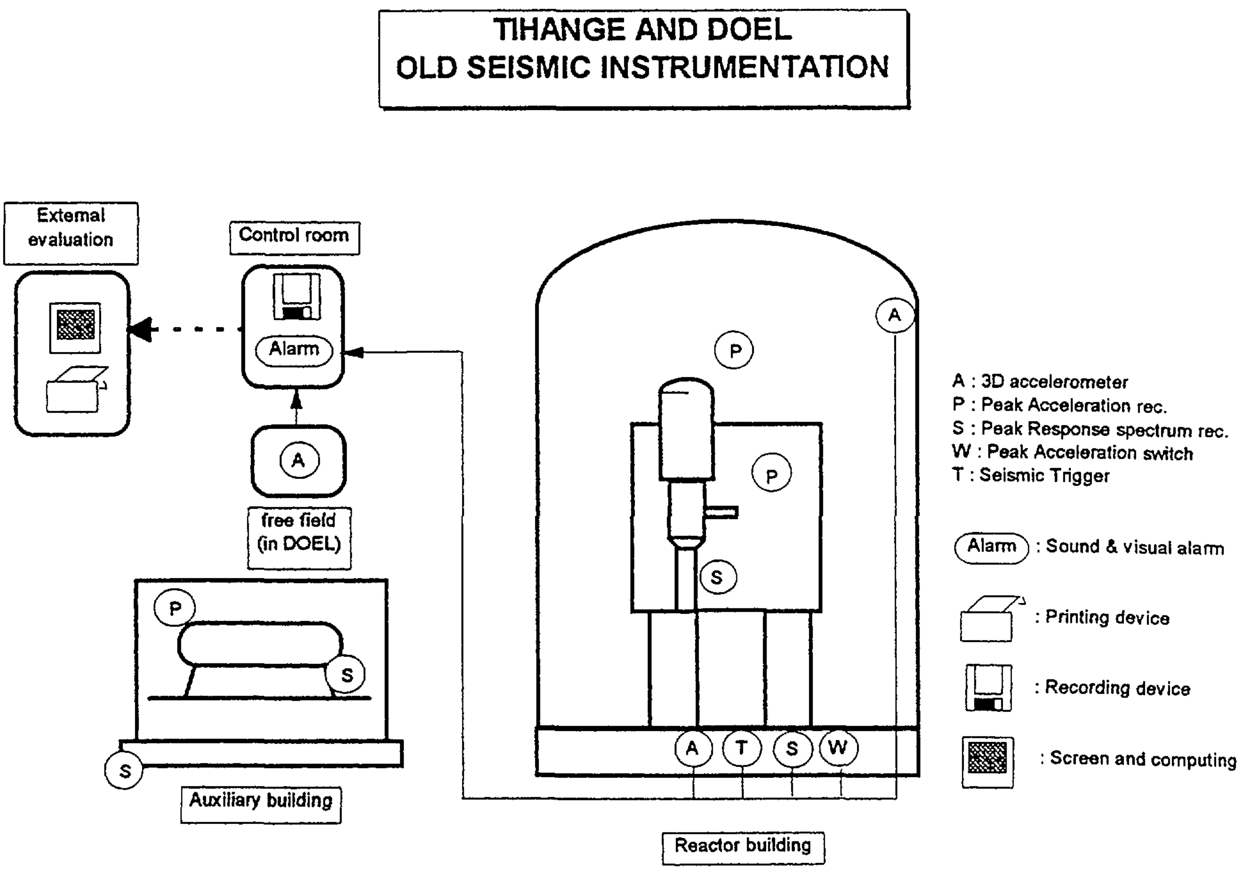

Figure 2 - Tihange and Doel - Old Seismic Instrumentation

- Peak acceleration and peak response spectrum recorders on some primary equipment or piping support in one of the auxiliary buildings.

The analogue recording of the accelerometers starts when the trigger at the base of the reactor building is activated in any of the three directions (acceleration of the earthquake > 0.01 g). An alarm is activated in the control room when any of the three devices at the base of the reactor building is activated :

- the trigger acceleration exceeds 0.02 g;
- the peak acceleration exceeds the OBE acceleration;
- the peaks recorded on the response spectrum recorder exceed the OBE response spectrum.

3.2 *Original OBE exceedance criteria*

The OBE is considered to have been exceeded when two of the three possible alarms in the control room have been activated by the earthquake.
In case of exceedance, the operator initiates the shut-down procedures of the plant.

3.3 *Original procedures to be applied after an earthquake*

These procedures were written according to SRP 3.7.4 [3]:

The operator organises a walk down to identify any significant damage due to the earthquake that has occurred in the plant.

The records of the accelerometers are corrected and digitised. Their response spectra are calculated and compared to the design response spectra.
The records on the peak acceleration and the peak response spectrum recorders are collected and interpreted.
The measured answers are compared to the results of the seismic design.

If the results of the analyses using the real earthquake as input are lower than or equal to the results of the analyses with the OBE and if no damage or anomalies have been discovered during the walk down, the plant can be restarted, provided acceptance by the Safety Authorities.
If the results of the OBE analyses are exceeded or if damage or anomalies due to the earthquake are observed, more in depth analyses and inspections are performed to evaluate the restart possibilities and to fix the conditions for this eventual restart.

Before any restart after an earthquake, the operator proceeds to a functional check of the seismic instrumentation.

3.4 *Experience from the Roermond earthquake*

An earthquake with a maximum epicentral intensity of MSK VII and a Richter Magnitude of 5.7 occurred near Roermond in April 1992. This earthquake was felt within a radius of 400 km.
No triggering occurred in Tihange (epicentral distance 85 km) or Doel (epicentral distance 100 km). After inspection of the safety indicators in the control room, walk downs were performed in all units of both sites. Some equipment and supports of class 1 structures like the containment, the steam generators or primary pumps supports or basic structures like the turbine supports or the cooling tower were also inspected. No damage nor anomalies due to the earthquake were detected.

The recording system was not triggered. In Doel, the passive recorders outside of the containment were inspected. The peak accelerometers and peak response spectra recorders with scratch plates showed very high accelerations (from 0.2 g at 10 Hz to more than 1.0 g in some cases at frequencies above 20 Hz). These values were judged by specialists to be more likely the traces of shocks near or on the recording device than the results of an earthquake.

This real test showed that the operator of the plant reacted correctly by ordering an inspection of the plant in spite of no alarm having been activated. However, this inspection did take place the next day after the event and addressed mainly structures and supports rather than components and functional aspects. The operator observed no anomalies, in spite of the fact that he had no detailed instructions on what to observe and where to observe it.

This demonstrated also that the instrumentation system was not adequate or not correctly used. The passive instruments were not reliable and the interpretation took too much time : it was only one week after the event, that a specialist was asked to interpret the scratches on the plates. If records of the accelerograms had been taken, three days would have been necessary to interpret and correct the records.

Belgium is a small country and this medium earthquake was felt on both sites. This means that a bigger earthquake would have triggered the alarms on both sites, requiring probably to stop all the nuclear plants in the country.
Even without significant damage or if the earthquake had a low energy content, the plants would be shut-down. Because there were no detailed instructions on how to evaluate in a fast and efficient way the possible damage and the capability of resuming operations and because of the long delay to interpret the records, the consequences of the loss of power would have been dramatic.

3.5 *Benefits of the SQUG inspections*

All three units of the Tihange plant had to show seismic adequacy of both the structures and equipment for earthquakes more severe than those considered during the design stage [17]. To demonstrate seismic adequacy of the required equipment, the Belgian utility became member of SQUG (Seismic Qualification Utilities Group) right in 1985. The SQUG program has been applied to the three units (1989 to 1992, see [11] to [17]) and it allowed to accept most of the equipment with slight or no reinforcements.

Application of the SQUG approach requires the following ([11], [12]) :

- The list of required equipment must be set up : the SSEL (Safe Shut-down Equipment List). This list defines all the equipment that is needed to bring the plant to a "safe state" after a SSE has occurred. Safe state was defined in agreement with the Belgian Safety Authorities as any state between hot and cold shut-down. The list needs include all supporting equipment, be it for power supply (Air, electricity,...) or instrumentation and control. It should be noted that equipment on the SSEL is not necessarily part of the originally Seismic Class I equipment set. The SSEL is set up by the systems engineer, who determines the equipment needed to perform the four so called "vital function" to bring the plant to the "safe state". These "vital functions" are defined in the SQUG methodology.

- The equipment on the SSEL needs be classified in the 22 SQUG classes of equipment (See table 1). These are the classes on which experience exist about the seismic behaviour. References [11] and [12] define the classes and give the restrictions ("Caveat") that apply in order to include a given component as member of a class.

- All the available information on the equipment needs be gathered and organised. This amounts to set up a real data base on the plant equipment. The following examples of documentation type can be mentioned : equipment drawings, anchorage data, catalogues, calculations notes, nozzle loads reports (forces, moments, displacements for all kinds of loadings, ...). As part of this documentation, all seismic floor response spectra had to be made available to the project.

Table 1 - The 22 SQUG Equipment Classes

N°	CLASS	EXAMPLES
[1].	MOTOR CONTROL CENTERS	- Motor control centres - Wall- or rack-mounted motor controllers
[2]	LOW VOLTAGE SWITCHGEAR	- Low voltage draw-out switchgear (480 Volt) - Low voltage disconnect switches (480 Volt) - Unit substations - Automatic transfer switches
[3]	MEDIUM VOLTAGE SWITCHGEAR	- Medium voltage draw-out switchgear (4160 Volt) - Low voltage disconnect switches (4160 Volt) - Unit substations - Automatic transfer switches
[4]	TRANSFORMERS	- Liquid-filled medium/low voltage transformers (typically 4160/480 Volt) - Dry-type medium/low voltage transformers - Low voltage transformers (typically 480/120 Volt)
[5]	HORIZONTAL PUMPS	- Motor-driven horizontal centrifugal pumps - Engine-driven horizontal centrifugal pumps - Turbine-driven horizontal centrifugal pumps - Motor-driven reciprocating pumps
[6]	VERTICAL PUMPS	- Vertical single-stage centrifugal pumps - Vertical multi-stage deep-well pumps
[7]	FLUID-OPERATED VALVES	- Diaphragm-operated pneumatic valves - Piston-operated pneumatic valves - Piston-operated hydraulic valves - Spring-operated pressure relief valves
[8]	MOTOR OPERATED VALVES	- Motor-operated valves - Solenoid-operated valves
[9]	FANS	- Blowers - Axial fans - Centrifugal fans
[10]	AIR-HANDLERS	- Louvers - Cooling coils - Water-cooled air handlers - Refrigerant-cooled air handlers (including enclosed chiller) - Heaters
[11]	CHILLERS	- Water chillers - Refrigerant chillers
[12]	COMPRESSORS	- Reciprocating-piston compressors
[13]	MOTOR-GENERATORS	- Motor-generators
[14]	DISTRIBUTION PANELS	- Distribution panelboards (120-420 Volt, AC & DC) - Distribution switchboards (120-420 Volt, AC & DC)
[15]	BATTERY RACKS	- Batteries - Battery racks
[16]	BATTERY CHARGERS & INVERTERS	- Battery chargers - Rectifiers - Static inverters
[17]	ENGINE-GENERATORS	- Piston engine-generators - Gas turbine-generators
[18]	INSTRUMENT RACKS	- Wall-mounted transmitters - Rack-mounted transmitters - Supporting racks
[19]	TEMPERATURE SENSORS	- Thermocouples - RTD's
[20]	CONTROL & INSTRUMENTATION CABINETS	- Wall-mounted & rack-mounted control panels - Wall-mounted & rack-mounted control CABINETS - Dual switchboard control cabinets - Duplex switchboard & benchboard (walk-in) control boards
[21]	TANKS & HEAT EXCHANGERS	
[22]	CABLE AND CONDUIT RACEWAYS	

- As part of the SQUG program, a thorough walk-down was performed on the entirety of the equipment of the SSEL. This allowed to ascertain the state of the equipment (at the time of the walk-down or at the time of the reinforcements).

At the Doel plant (Four units) a cursory walk-down was performed, the purpose of which was to verify that generic problems encountered in Tihange were not present in Doel. No data base was set up for this site.

4. IMPROVEMENT OF THIS SITUATION : THE WORK OF EPRI

In the late eighties, EPRI started a reflection on the subjects of the responses to be given in case of an earthquake [4] based on the experience gained in the observation of the consequences of real earthquakes on industrial plants and the fact that nuclear power plans are more robust by design.

The damaging character of one earthquake versus another was also considered by EPRI to elaborate more realistic criteria of OBE exceedance [5].

This work has been confirmed by the NRC in several projects of regulatory guides devoted to seismic instrumentation (DG 1033 [6]), pre-earthquake planning and immediate actions after earthquake (DG1034 [7]) and restart of a nuclear power plant after an earthquake (DG 1035 [8]). These documents are endorsed by the project of a new federal document, 10 CFR 50 appendix S, which will be applicable for all US nuclear power plant applying for a licence in the future.

This work can be summarised as follows:
When an earthquake strikes a nuclear power plant, three set of actions have to be taken in a timely order (see fig. 3):

1. Immediately after the earthquake it is necessary to :
   - stabilise the plant;
   - evaluate the immediate effects of the earthquake on the physical state of the plant ;
   - determine whether shut-down is necessary because the plant has been damaged or because the earthquake exceeded the reference earthquake used for design;
   - evaluate whether the plant can be shut down safely, if shut-down has been decided.

2. In order to help the decision to restart the plant after a shut-down due to an earthquake, make planned and organised inspections and tests so that :
   - the plant can be restarted if the damage is less than a minimum level and if no damage occurred to safety related equipment;
   - if not, expanded inspections are made on safety related equipment or non safety related balance of the plant equipment necessary for normal operation. According to the level of the damage observed, the plant is restarted after repairs, or after satisfactory leak tests and repairs.
   - in case of severe damage, all the safety equipment is checked to establish its acceptability and, if satisfactory, the plant can the be restarted after repairs and leak tests of the containment that give satisfactory results.

3. After restart of the plant, or before restart in case of important damage, check the consequences of the earthquake on the safety equipment by comparing the effects of the earthquake calculated from the records to the design values for the SSE. In case of exceedance, the equipment is qualified by functional tests or detailed analysis.

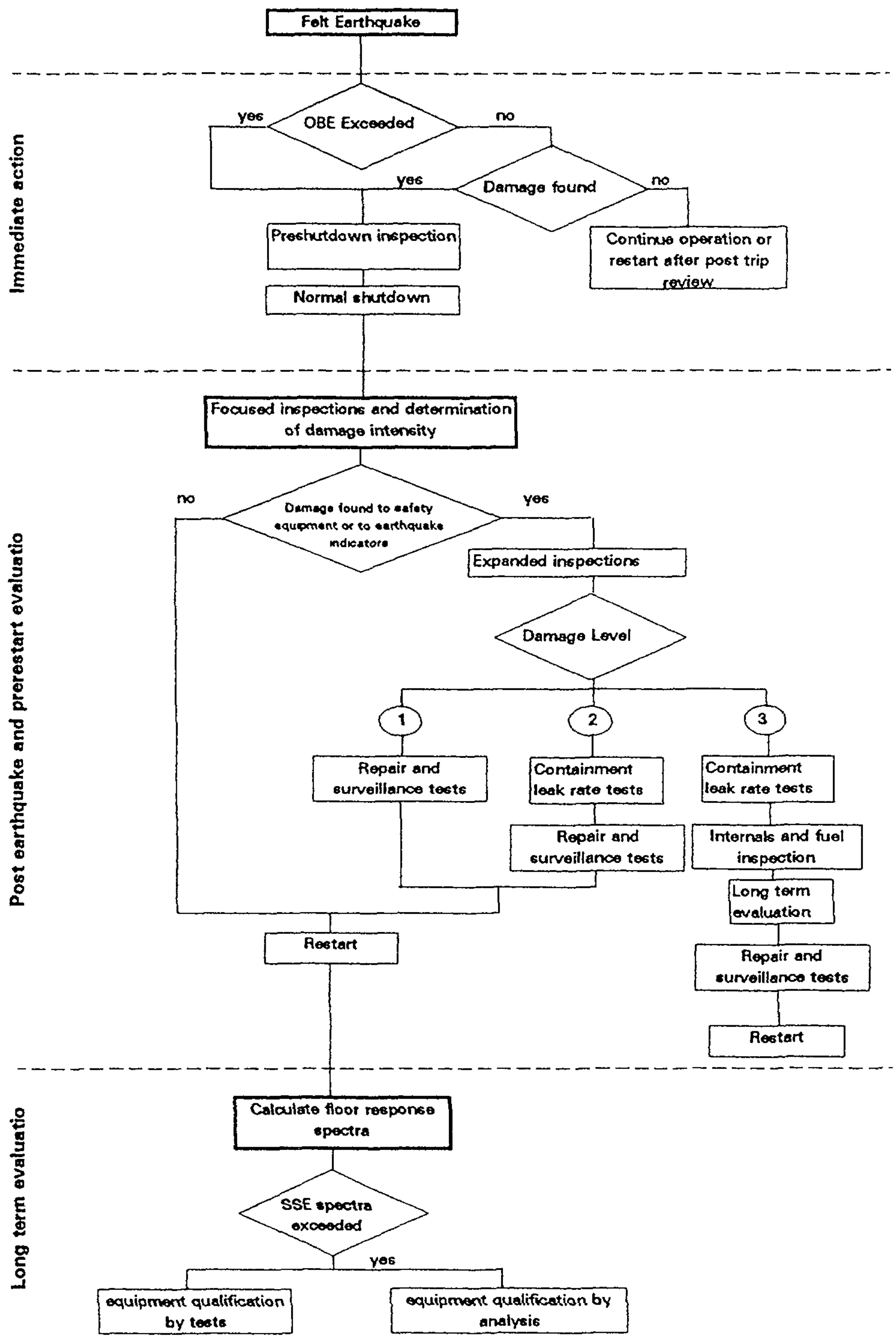

Figure 3 - Summary of Actions

5. DESCRIPTION OF THE PROCEDURES RECOMMENDED IN BELGIUM

The new Belgian post earthquake procedures have been written in 1995 and implemented in 1996 from the work of EPRI adapted to the situation of the Belgian practice and regulations. The three levels of actions are detailed as follows:

5.1 *Immediate post earthquake actions*

These are the necessary actions to determine and control the physical condition of the plant immediately after an earthquake and to evaluate the gravity of this earthquake. The sequence of these actions is given on the logical chart (See fig. 4).

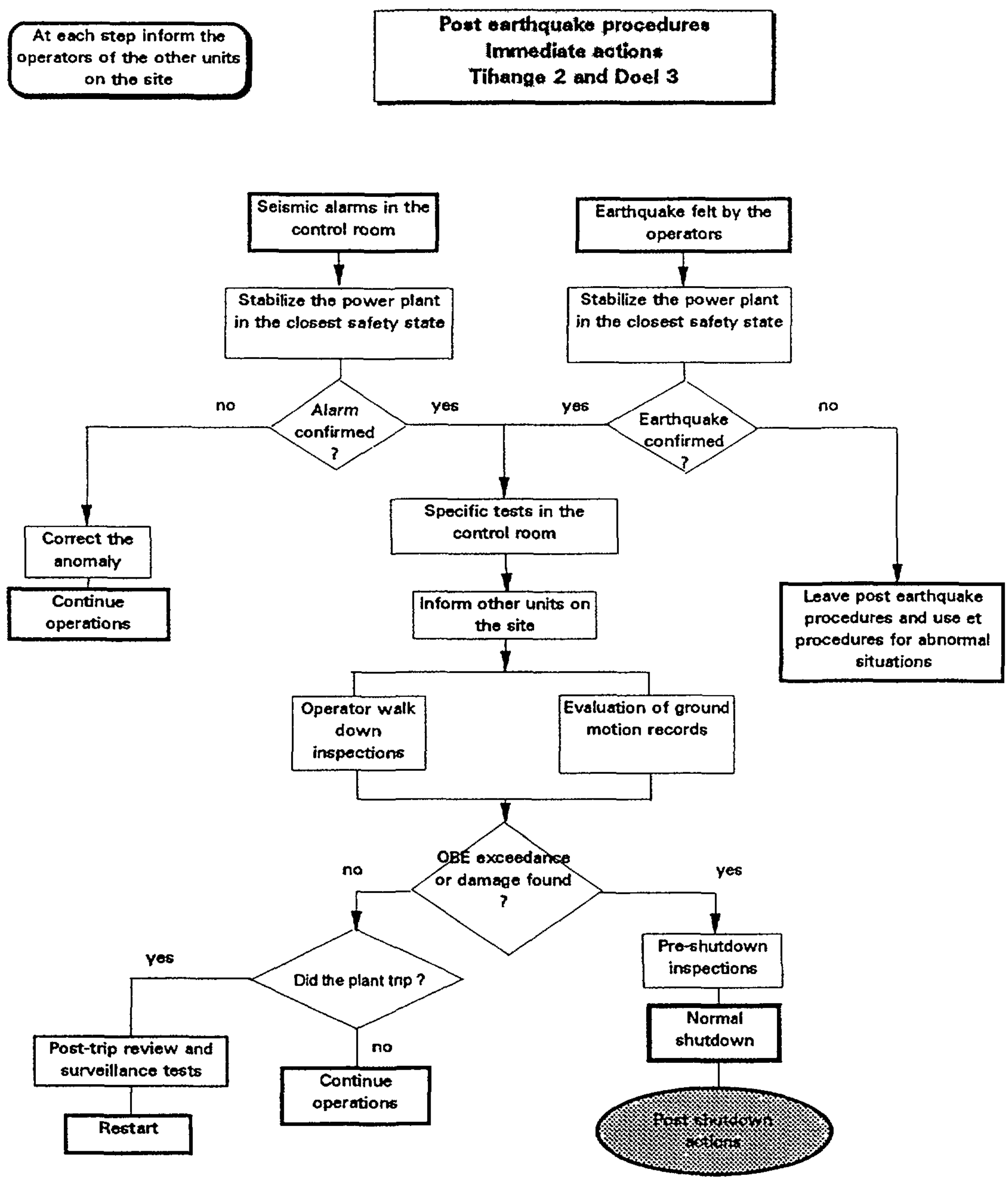

Figure 4 - Post earthquake immediate Actions

This procedure is started if a seismic alarm is triggered, if an earthquake was felt on the site, or if an earthquake of a certain magnitude was signalled at a certain distance from the plant :

1. The operator immediate actions are to bring the plant into the closer safe and stable state during the period necessary to decide of the shut-down of the plant and initiate it if required. These actions are taken in accordance with the approved procedures in response to operational symptoms observed in the control room.

2. The operator or the specialised personnel available on the plant performs a visual inspection of the plant in a similar way to the daily rounds. During this walk-down, he will identify any significant damage that has occurred in the plant.

3. Simultaneously, the ground motion records and the earthquake parameters from the site seismic instrumentation are collected and interpreted. The values are evaluated against the OBE exceedance criteria.

4. If the OBE has been exceeded or if significant damage has been found during the walk-down, the plant should be shut down in an orderly manner to allow further inspections and tests. If the plant has tripped due to the earthquake under conditions of shut-down, it would remain shut down for the inspections and tests.

5. If the OBE has not been exceeded, or if no significant damage were observed, the plant can resume operations or restart if it was stopped, following the verifications and tests of the restarting procedures.

6. After the decision to shut down the plant, the operator or specialised personnel will inspect visually the essential safe shut-down equipment to establish its readiness (particularly the essential safe shut-down equipment which is not normally in use during power operation) so that any repair can be performed or alternate equipment can be readied. The availability of the off site power sources is checked and if it is recognised as uncertain, the availability of the on site emergency power sources should be determined.

7. Operate the shut down when the plant capability to shut down safely has been verified.

All the actions necessary to decide the shut-down of the plant must be performed within the next 4 hours after the earthquake. The checking of the shut-down capabilities must be performed within 4 hours following the decision to shut down. All these actions must be performed by the operator and trained personnel of the plant.

5.2 *Pre-restart actions*

These are the necessary tests and inspections to determine, in details, the physical condition of the plant and its capacity to restart it, if it was shut down due to the immediate post earthquake actions or due to a trip caused by the earthquake.

The planned pre-restart tests and inspections must allow a fast and reliable restart of the plan, distinguishing between the immediate and necessary actions (tests, inspections and repairs) necessary to let the plant work normally, and the long term actions, which can be taken after restart, to check and, if necessary, restore the full integrity and the long term reliability of the plant.

In general, for equipment, the inspections consists of a visual observation of the condition of the equipment anchorage or of the condition of the attached piping and conduits or a check for other evidence of physical or functional damage.
Any damage that has the potential to impair the operability, the functionality or the reliability of structures or components necessary for the safe operation of the nuclear power plant is considered as a significant damage.

The sequence of these actions are given on the logical chart of fig. 5

These actions consist of :

1. **Focused inspections** : these are the detailed and visual inspections of a pre-selected sample of structures and components. This sample represents all types of structures and equipment which are safety related in a nuclear power plant (including the 22 classes of equipment of the SQUG necessary for a safe shut-down, low and high pressure storage tanks, piping, electrical raceways, air handling ducts) as well as typical structures and equipment non safety related. The latter are selected as indicators because the damage that they may suffer in case of earthquake are representative on a seismic severity scale.

2. Determine an intensity of damage by comparison of the results of the tests and inspection with a seismic damage scale that has been developed by EPRI from the experience gained after real earthquakes.
   This scale contains four levels, which can be classified as follows:
   - Level 0 : No damage and no alarms for safety related structures and equipment seismically designed. Some light damage and alarms on vibration sensitive components non seismically designed. This level corresponds to earthquakes slightly below the OBE exceedance criterion.
   - Level 1 : Similar to level 0 for safety related structures and equipment. Light generalised damage and alarms in structures and equipment non seismically designed. Level 1 corresponds to earthquakes that are slightly above the OBE exceedance criterion.
   - Level 2 : First signs of damage, leaks or cracks on safety related structures and equipment which have been designed for earthquake. Widespread significant damage on non seismically designed structures and equipment.
   - Level 3 : Clear and evident signs of damage, cracks and permanent deformations on safety related structures and equipment. Severe damage to non seismically designed structures.

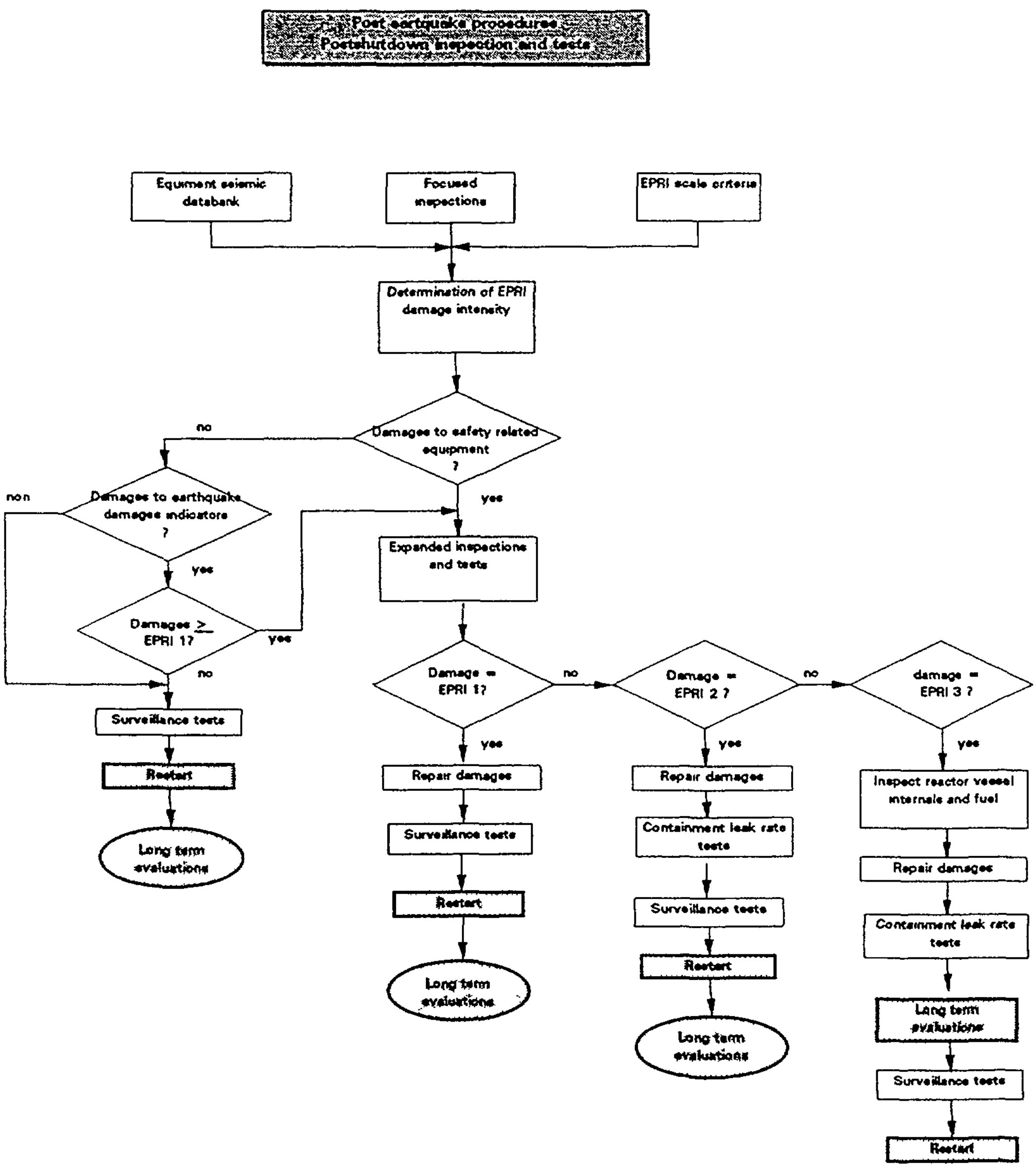

Figure 5 - Post shut-down Inspections and Tests

3. If there is no significant damage to safety related equipment or structures or if the damage to the equipment selected as damage indicators is smaller than level 1 on the EPRI scale, operation can be resumed after the tests and inspections required before start-up of the plant.

4. **Expanded inspections** : if there is significant functional or physical damage to safety related equipment or structures or if the damage to the equipment selected as damage indicators is equal to or greater than level 1 on the EPRI scale, **expanded inspections** are necessary. All safety related equipment and structures and all non safety related balance of plant related equipment and structures required for the normal operation of the plant must be inspected and tested. The damage, if any, is reported and an EPRI level of damage determined.

5. If after this expanded inspection, the evaluated EPRI level is smaller than 2, the plant can be restarted after all reported damage has been repaired or corrected as required. Surveillance tests, as required by the plant technical specifications should also be performed. During surveillance testing, the vibration of rotating equipment should be closely monitored.

6. If the evaluated EPRI level is equal to or higher than 2, a leak rate test of the containment buildings must be performed in addition to the requirements of repairs and corrections as well as the surveillance tests required for the lower level (see [5]). The plant can be restarted after the repairs, the corrective actions and satisfactory results of the surveillance and leak rate tests.

7. If the evaluated EPRI level is equal to or higher than 3, it is necessary to inspect the reactor vessel internals and the fuel elements in addition to the containment leak rate tests and the necessary repairs and corrective actions detailed in point 6. Moreover, the plant will not be restarted before performance of the long term evaluations of the safety related equipment and structures.

In all circumstances, when a nuclear power plant has been shut down because of OBE exceedance or because seismic damage has been found, long term evaluations must be performed. The plant can be restarted before the results of this long term evaluation, except if the EPRI damage level is equal or higher than 3.

The inspections and tests are performed by the plant operation personnel assisted by a group of utility and contractor specialists experienced in civil/structural, mechanical and electricity engineering and trained in observation of earthquake induced damage.
The focused inspection must be prepared in order to complete it within 24 hours after shut-down. The expanded inspections, if necessary, should be completed within two weeks after shut-down, except if it is necessary to remove the reactor vessel head.

5.3 *Long term actions*

These actions aim at evaluating the possible hidden damage to structures or equipment and which could impair their long term reliability. When completed they assure that the plant can safely operate for a long period and that it is able to sustain another earthquake (See fig. 6).

These actions are only required if shut-down was due to OBE exceedance or if earthquake induced damage was sustained by the plant. If the plant had been shut down and if damage was found to be below level 2 on EPRI scale, it is not required that they are completed to restart the plant.

1. If the "design parameters" (Floor response spectra, seismic forces and moments,...) are below their allowable values (Design or upgraded values), no further action is required.

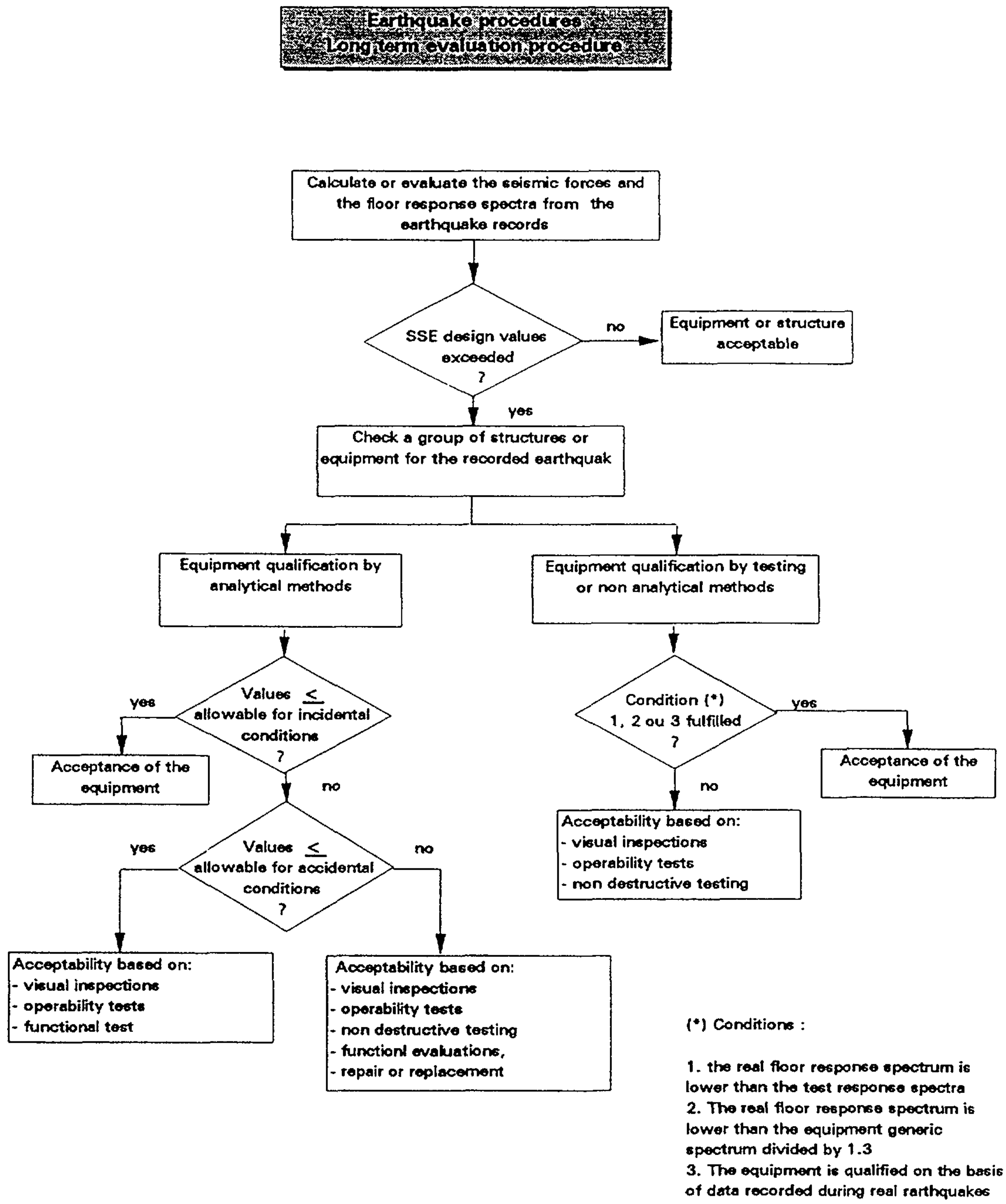

Figure 6 - Long Term Evaluation Procedure

2. If it is not the case, a representative sample of the equipment needs be assessed.

   a. If assessment is done by analysis, a first step is to compare calculated values to allowables corresponding to emergency conditions (ASME Level C).

      If this condition is not met, faulted conditions criteria (ASME Level D) can be used, provided some precautions are taken (Inspections, tests, ...). Eventually if faulted criteria are not satisfied, expanded, in depth inspections or tests may lead to accept it, otherwise reinforcement, modification or replacement must be considered.

   b. Assessment may be based on tests. Real floor (in-cabinet?) response spectra are compared to the test response spectra. In case "Generic Response Spectra" (GERS) are used, a margin of 30% is required.

c. Assessment may be based on real earthquakes experience. The method is well documented in specialised reports [12]. Its description is beyond the scope of this paper.

6 NECESSARY PRE-EARTHQUAKE ACTIONS

To make these procedures work effectively, it is necessary to improve the tools used for seismic detection as well as the criteria used to determine the seismic exceedance.
In a country of low to moderate seismicity, where people cannot imagine the consequences of earthquakes, it is recommended to familiarise the personnel of the plant with what they can expect in case of earthquake and to give them a reminder of what and where the damage is most likely to occur.

6.1 *Realistic criteria of OBE exceedance*

It is known that earthquakes having a spectrum with a higher frequency content and a lower duration are less destructive than the earthquakes used for the design, even if their spectral accelerations may exceed the design accelerations [4]. Similarly, in United States, peak accelerations higher than the OBE accelerations have been observed on some non commissioned plants with no significant damage to the plant or the equipment [5].
The old criterion based on the activation of two out of three alarms in the control room (triggering (1), exceedance of the OBE acceleration (2) and of the response spectrum (3) at the base of the reactor building) is not realistic. In some instances, shut-down might be required with no damage at all to the plant, causing unnecessary loss of power during the period necessary to check the plant.

EPRI has proposed a new criterion [5], completed by the NRC [7], based on the correlation between the damage observed during real earthquakes and the records of the earthquakes. It takes also into account the low frequency content and the duration of the earthquake.

The OBE is considered to have been exceeded if the two following conditions are met simultaneously for one of the three components of the earthquake:

1. one of the three components of the 5% damped response spectrum of the natural earthquake, recorded in the free field :
   - at frequencies between 2 and 10 Hz, exceeds the corresponding OBE design response spectrum or 0.2 g, whichever is greater;
   - at frequencies between 1 and 2 Hz, exceeds the corresponding velocity spectrum or 15 cm/s, whichever is greater.
2. the Cumulative Absolute Velocity (CAV) is greater than 0.16 g.s. The CAV is the absolute area under the accelerogram, calculated for consecutive steps of 1 seconds where the peak acceleration exceeds 0.025g.

It should be noted that the criterion for determining whether the OBE has been exceeded is independent of the plant 's design OBE and SSE ground response spectrum.

To use this criterion for European nuclear power plants, it is necessary to :

- correlate the damaging characteristics of significant European earthquakes with a CAV threshold level similar to the one fixed by EPRI;
- compare the characteristics of the safety equipment of the European plants with the US data base and particularly the design limits for the seismic loading [11], [12];
- compare the design seismic criteria for piping, supports and anchorage of equipment;
- compare the design seismic criteria of the structures and their foundations.

A recent paper published by Cabanas, Benito and Herraiz [9], based on recent recording of Italian earthquakes and on damage observed in the vicinity of the seismic instrumentation, showed that there was a good correlation between the CAV (based on 0.2 g.s) and the macroseismic information.

Since the Belgian nuclear power plants have been designed or reassessed using the US regulations and data base, it is the intent of the Belgian utilities to use the same OBE criteria. The evaluation of the CAV criterion for the site of Tihange is underway, based on the natural earthquakes accelerograms used for the site specific spectrum generation. During the evaluation period, the old criteria are still valid.

6.2 *Improvement of the seismic instrumentation*

As described in a paper presented by the first author at the SMIRT conference [10], the Belgian utilities have renewed the seismic instrumentation.
This was due mainly to the obsolescence of the sensors which are not any more made and to the cost of maintenance.
This instrumentation had to be completed by a computing system to respect the new procedures, because it is necessary to interpret the earthquake records on the site and to reduce the time delay from 48 hours to 4 hours which is not possible with the old analogue system.
It was decided to place a new instrumentation not using anymore the passive recording devices which were not reliable and cost much money in maintenance and training.

The new seismic instrumentation consists of a digital recording station and a computer in the control room linked to three 3D acceleration sensors in the free field, at the base and at the top of the containment of the reactor building (See fig. 7).
The recording system provides a pre-event memory and is able to record continuously for at least 25 min.
The measured acceleration on any of the sensors is continuously compared to the triggering levels. When it is exceeded, the system gives an alarm which initiates the post earthquake procedure, it starts the recording and calculate the spectra and the CAV from the free field records and it gives an alarm in the control room in case of OBE exceedance.

The operator can visualise the accelerograms and the spectra and where the exceedance occurred at each location of the sensors.

The computer examines also the type of the triggering acceleration and signals the operator which events are not of seismic origin; the operator must however always acknowledge an alarm when triggering occurs.

In case of problems on the computing system or on the transmission lines, the free field station can be battery operated and the records saved and processed on a laptop computer.
This instrumentation is considered as a minimum and it can be completed by other recorders linked to 3D acceleration sensors placed on the internal structures within the containment or in an auxiliary building.

6.3 *Data base of components and equipment*

In order to validly assess the damage caused by the earthquake, a "snapshot" of the pre-existing state should be known. The information needs cover representative equipment and structures, safety related as well as non-safety related. The SSEL (Safe Shut-down Equipment List) set up to perform the SQUG walk-down's is a good starting point to pick a sample of representative, safety related equipment. For non-safety related equipment the following may be considered : large tanks, supporting columns of cooling towers, equipment in turbine hall, joints between buildings, ...

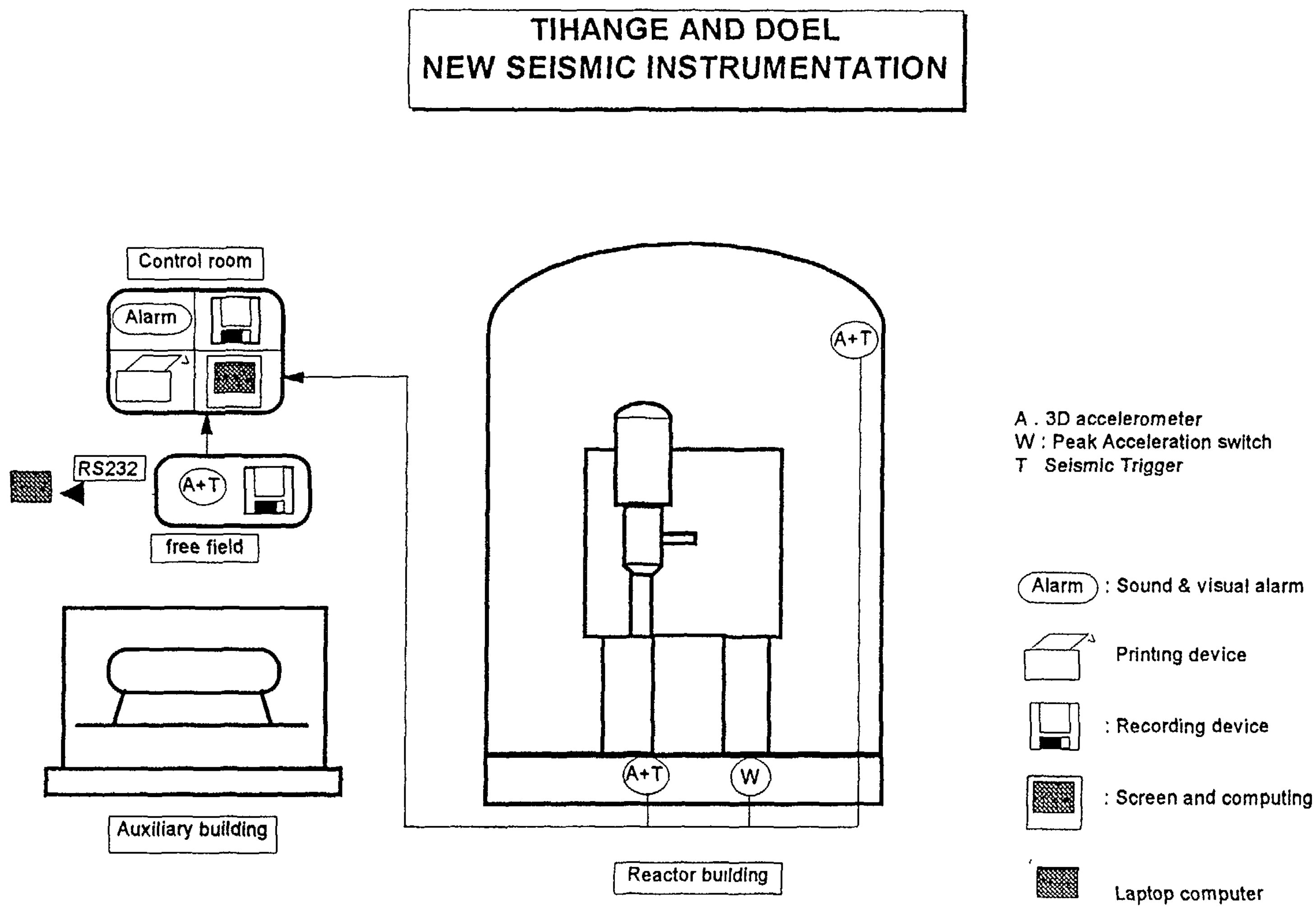

Figure 7 - TIHANGE and DOEL New Seismic Instrumentation

For both types, choice should be guided to maximise damage detection, i. e. equipment with components known to be earthquake sensitive (e. g. isolators on electrical equipment) and located in high amplification area's (e. g. upper floors of tall buildings). An initial inspection, including pictures or any other means of recording should be performed to assess the initial state of the equipment and structures. The documentation gathered on the SQUG equipment can ease this task, as the state of that equipment is known and documentation filing is organised. To take care of any possible "drift" due to maintenance, engineered modifications, ageing, etc. periodic cursory walk-down's should be performed to check that the information is kept up-to-date.

6.4 *Practical implementation and training of the personnel*

Implementation of the approach is done in several steps. A first step ("Conceptual phase") consists in analysis of the existing documentation from various origins (USA, France, Germany,..., synthesise it and prepare a series of recommendations and guidelines for practical implementation. These recommendations and guidelines are subsequently analysed, commented and eventually accepted by the Safety Authorities.

Before the approach can be implemented in a practical case, the guidelines need be translated in actual technical procedures defining the work down to the last detail. These procedures have to define who will accomplish each task and describe this task, such as what equipment or structure to look at, in which building area, distinguishing among the various types of inspections (General, focused, ...). Acceptance criteria should be included as well as subsequent actions in case of meeting / failing the criteria. Detailed knowledge of the plant is therefore needed and these procedures should consequently be written by plant personnel, with the assistance of a consultant familiar with seismic problems. Familiarity with the SQUG approach is obviously an asset.

Focused inspections have to be carried out within 24 hours of the earthquake. These inspections need consequently be executed by the available plant personnel. In order to get meaningful results in such a reduced time span, the inspection need be rigorously organised (Inspection spots well identified) and prepared (Pre-earthquake state well documented). The inspecting personnel need also to be trained in what to look at.

Expanded inspections need be completed within two weeks after the earthquake. As more time is available and more inspection spots are involved, these inspections may be carried out with the assistance of specialised consultants who can then exercise their engineering judgement in evaluating the importance of the damage that might be discovered.

7. STATUS OF PREPAREDNESS

- The generic procedures for al seven Belgian units are written and accepted by the Safety Authorities.
- The new instrumentation is defined, accepted by the Safety Authorities and is (or will soon be) operational : Tihange site since December 1996, Doel site in December 1997.
- Concerning the OBE exceedance criteria, the "old" criteria are still applied. The "new" criteria have been proposed and are under discussion with the Safety Authorities.
- The new Technical Specifications for immediate action and for restart are written and submitted to the Safety Authorities.
- For the "long term actions", no Technical Specifications are needed. The documentation to perform the work is available, as well for civil structures as for equipment.
- For the Tihange site, lists of equipment for "focused" and "expanded" inspections have been prepared. The preparatory walk-down to ascertain the "pre-earthquake" state of the equipment has been carried out.

8. CONCLUSIONS

It is well understood that these new rules are mandatory for new plants only and may be applied on a voluntary basis for existing plants. The new rules seem to be more demanding, but they present the advantage of clarity, especially in the conditions for restarting the plant. As a matter of fact, under the old rules, whereas plant shut-down is not legally mandatory in case of OBE exceedance, restart after a shut-down induced by indirect consequences of earthquake entirely depends on the approval of the Safety Authorities. They might require extended inspections and evaluations, sensibly delaying the plant restart. At the contrary, under the new rules, conditions for shut-down and restart are clearly set beforehand and the plant can be restarted without undue delay. This is an obvious advantage, for after an earthquake, electrical power is most needed.

Preparation of the new rules implementation required an in-depth knowledge of the plant design bases (Systems, Civil structures, Equipment characteristics and capacity, ...). This detailed knowledge is not available within the plant personel of the Belgian NPP's. The fact that the engineering office was involved as well in the design, construction and subsequent engineering activities (such as SQUG programs) greatly helped in achieving the implementation in a rigorous yet efficient manner.

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SEISMIC ALARM SYSTEM FOR IGNALINA NUCLEAR POWER PLANT

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Abstract

A seismic alarm system will be installed at the Ignalina Nuclear Power Plant (INPP) in Lithuania. There are two reactors, both RBMK 1500 MW units. Each reactor is a water cooled, graphite moderated, channel type reactor. INPP has the most advanced version of the RBMK reactor design series. The first and second units of INPP went into service at the end of 1983 and in August 1987 respectively. Their design lifetime is approx. 30 years. The various buildings and plant have been designed for two earthquake levels, that is the design earthquake and the maximum possible earthquake with peak ground accelerations ranging from 1.2% to 10% of the acceleration due to gravity. Certain parts of the buildings and some of the equipment of the first and second units do not comply with Western seismic standards. As seismic strengthening of the existing buildings and equipment is not feasible economically, a reactor protection system based on an earthquake early warning system was recommended. This system essentially consists of six seismic stations encircling INPP at a radial distance of approx. 30 km and a seventh station at INPP. Each station includes three seismic substations each 500 m apart. The ground motion at each station is measured continuously by three accelerometers and one seismometer. Data is transmitted via telemetry to the control centre at INPP. Early warning alarms are generated if a seismic threshold is exceeded. This paper discusses the characteristics of INPP, the seismic alarm system presently under construction and the experience with other early warning and seismic alarm systems.

1. INTRODUCTION

The group of seven Industrialised Nations (G-7) agreed in March 1992 on an action plan to upgrade the safety of Soviet designed reactors. From the fund, called the Nuclear Safety Account, administered through the European Bank for Reconstruction and Development (EBRD) a grant of USD 38 million was allocated in 1994 for projects to upgrade the Ignalina RBMK plant. The EBRD have set up a project management unit (PMU) at the Ignalina plant, comprised of plant staff and Western experts to manage these projects.

In 1996 a joint venture formed by Electrowatt Engineering and GeoSys was awarded the contract to install an earthquake early warning system at the Ignalina Nuclear Power Plant (INPP), situated in Lithuania in the Baltic area. The purpose of this warning system is to provide information and alarms to allow the safe shut-down of the two reactors in the event of seismic waves from moderate to strong earthquakes approaching the plant.

The power plant was designed for two types of earthquakes. These are the design earthquake and the maximum possible earthquake having peak ground accelerations ranging from 1.2 % to 10 % of the acceleration due to gravity, i.e. 0.012 to 0.1g. The reactor building was designed for accelerations of 0.026 to 0.051g which may result from rather moderate earthquakes. Some of the buildings and equipment of the first and second units of the INPP do not fully comply with Western seismic standards. Such buildings and equipment should be strengthened. However as seismic strengthening was not considered to be economically feasible, other options have been studied. In order to protect the reactor from earthquake damage, it was decided to install an early warning system and to shut down the reactor should a sufficiently strong earthquake occur in the vicinity of INPP. Accordingly, six seismic stations are to be installed in a ring centred on the plant at a distance of approximately 30 km. The stations are uniformly distributed as shown in Fig. 1. Each consists of three independent substations which are approx. 500 m apart. The ground motion is recorded continuously and transmitted to the control centre via telemetry as discussed in the subsequent sections.

2. CONCEPT OF SEISMIC ALARM NETWORKS

Research in earthquake prediction has shown that we are still some way from the accurate prediction of the time, location and magnitude of strong earthquakes. However, present technology in seismic instrumentation and telecommunications permits the implementation of systems for early warning of earthquakes. Such systems are capable of providing a warning of from several seconds to tens of seconds before the arrival of the strong ground tremors caused by a large earthquake.

An earthquake early warning system has the potential for the optimum benefit as it can provide the critical alarms and information needed (i) to minimise loss of property and lives, (ii) to direct rescue operations, and (iii) to prepare for recovery from earthquake damage (Lee et al., 1996).

The basic features of a seismic alarm network are shown in Fig. 2 (Heaton, 1985). Ground motions recorded by an array of seismometers are telemetered to a central processing site. The main parameters of an earthquake, i.e. the location, time of origin, magnitude, amplitude of ground tremors and reliability estimates are computed. Based on the location and the geological conditions the nature of the ground motions expected at the site is determined. On the basis of this information the appropriate action is taken.

The problem of false alarms is minimised by continuous updates regarding the size of the ground motions at differing stations in the seismometer array or by redundancy from several measurements at the same geographic location.

However, if the user is far from the epicentre, then considerable time is available before shaking begins. This time may be used to receive further information from external organisations

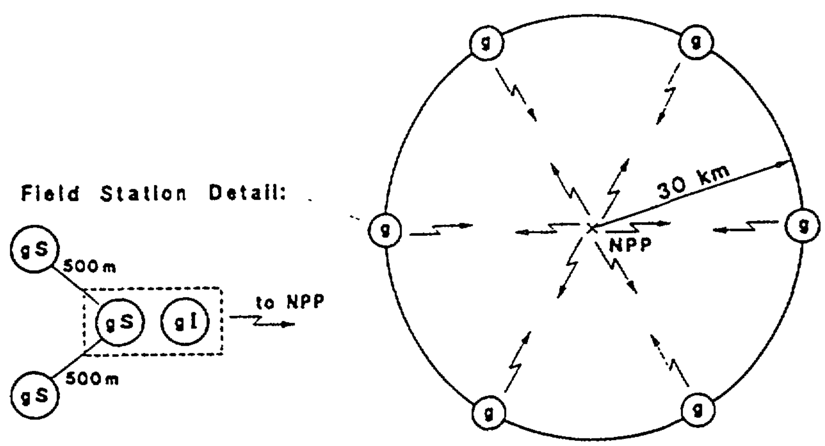


Fig. 1 Layout of seismic monitoring system of Ignalina nuclear power plant (NPP)
(g: station communication by telemetry; gl: substation with borehole seismometer;
gS: substation accelerometer with alarm switch and communication by cable)

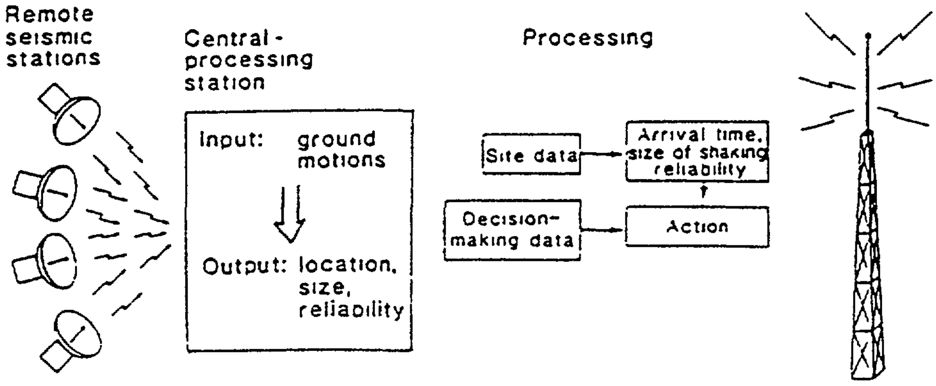


Fig. 2 Conceptual design of an earthquake early warning system

about the size of the earthquake. In this way, users at large epicentral distances take action only for the large earthquakes that present a real hazard, and each user adjusts the decision-making process to the needs of the site.

After the occurrence of an earthquake, the seismometer array provides information regarding the strength of shaking in different geographic locations. This information can be used to estimate regions of substantial damage, so that emergency services can be allocated promptly and properly. Because the seismometers in the array would have a large dynamic range, the seismic network may routinely record ground motions from numerous small earthquakes and teleseismic events. Such data are important for basic research in the fields of ground-motion prediction, earthquake prediction, and earth structure investigation. Also, the routine use of a seismic network for studies of numerous small events would help to ensure that the system operates properly when relatively rare large events occur.

Although relatively large peak accelerations occur at small distances from the numerous smaller earthquakes, they rarely cause great damage because the duration of intense shaking is short. Response spectral velocities of 1 second are usually considered to give a better estimate of damage potential than peak acceleration (Heaton, 1985).

For earthquakes with epicentres within a radius of 30 km of INPP the alarm time is reduced. A seismic station has been installed at INPP that can generate a seismic alarm by the onset of P- or S-waves. At Ignalina, this aspect is of less significance due to the geology and historically low seismic activity at the site. With regard to other nuclear power plants, the seismic properties of the site should be carefully investigated. An extended seismic array could provide seismic protection for seismically active sites.

3. EXPERIENCE WITH EARLY WARNING AND SEISMIC ALARM SYSTEMS

At the moment there are two early warning systems in operation for civilian purposes, i.e.

(i) Urgent earthquake detection and alarm systems (UrEDAS) in Japan.
This real-time earthquake disaster prevention system is used for railways. The special feature is the rapid alarm using information from P-wave data. Systems for different railways have been in operation since 1983 (Nakamura, 1996). UrEDAS detects initial P-wave motions, estimates epicentre azimuth and magnitude, calculates epicentral distance and local depth. This system is not only useful for railways but also for nuclear power plants, etc. Seismic data is transmitted to the interested parties 4 minutes after an earthquake.

(ii) Seismic alert system (SAS) for Mexico City.
Most of the large earthquakes which are likely to cause damage in Mexico City have their source in the subduction zone of the Pacific coast at a distance of about 320 km. The warning time varies between 58 and 74 seconds.

The Seismic Alert System for Mexico City consists of four elements: the Seismic Detection System, a Dual Telecommunications System, a Central Control System and a Radio Warning System for public and corporate users. The seismic detector system consists of 12 digital strong motion field stations located along a 300 km stretch of the Guerrero coast, arranged 25 kilometres apart. Each field station includes a microcomputer that continually processes local seismic activity which occurs within a 100 km radial coverage area around each station.

The Dual Telecommunications System consists of a VHF central radio relay station, located near Acapulco, and three UHF radio relay stations located between the Guerrero coast and Mexico City. Two seconds are required for information sent by one of the field stations on the Guerrero coast to reach Mexico City, this data is sent digitally coded.

The Central Control System continually receives information on the operational status of the field stations and communication relay stations, as well as the actual detection of an earthquake in progress. Information received from the stations is processed automatically to determine magnitude and is used in the decision to issue a public alert.

The Radio Warning System for users disseminates the seismic early audio warnings via commercial radio stations and audio alerting mechanisms to residents of Mexico City, public schools, government agencies with emergency response functions, key utilities, public transport agencies and some industries. Public and some private buildings, factories and offices are equipped with specially designed radio receivers to obtain the SAS alert. In each place there is a person in charge of the SAS receivers whose duties are to check the status of the receivers and co-ordinate all the activities of a disaster prevention including evacuation exercises and drills. There are a total of 98 radio receivers in operation of which 28 are installed in schools. During rush hours approximately 4.4 million people are covered by the system. The system has been operating since 1991. The system cost USD 1.2 million to develop and install and has running costs of USD 0.2 million per year for operation and maintenance (Espinosa-Aranda et al., 1996).

Other early warning systems have been reported by Shin et al. (1996), however, these are still in an experimental phase.

4. CHARACTERISTICS OF IGNALINA NUCLEAR POWER PLANT

4.1 General

The Ignalina NPP contains two RBMK-1500 reactors. This reactor type is the most advanced and powerful version of the RBMK reactor design series. The first unit went into service at the end of 1983 and the second unit in August 1987. Their design life is about 30 years. A total of 17 such reactors have been built in the former Soviet Union. In August 1991 INPP came under the authority of the Lithuanian Republic.

INPP belongs to the category of channel type boiling water reactors. The entire building of the two units covers an area of 600 m by 51 m and the reactor building is 61 m high. A cross-section through the reactor building of one unit is shown in Fig. 3. (Almenas et al., 1994).

4.2 Site Conditions

The INPP is located in an area with neotectonic motion of approx. 3.5 mm per year. The surface elevation varies from 150 to 180 m above sea level. The surface layer with a depth of 60 to 200 m consists of quaternary sediments which are very non-homogeneous. They were formed during the retreat of the last glaciers. Later on, alluvial, marsh and lake sediments were formed.

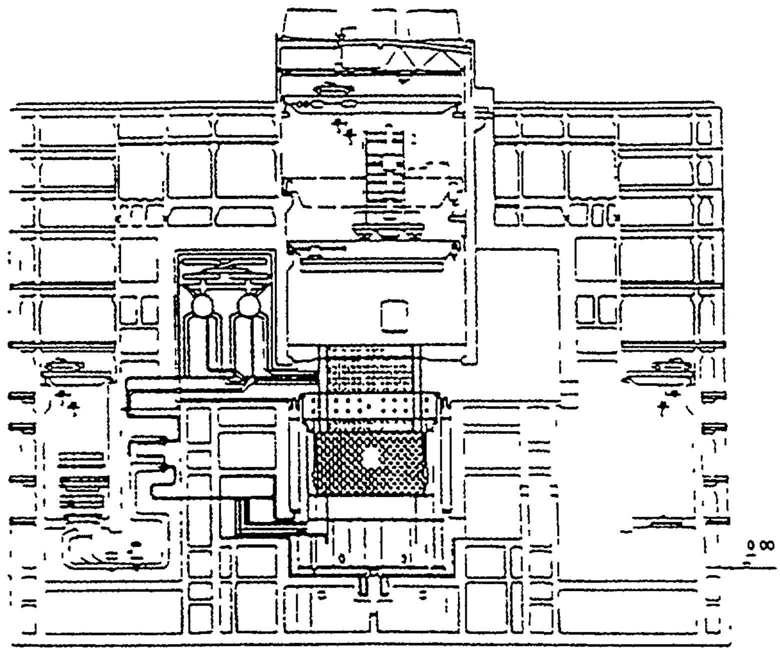


Fig. 3 Cross-section through reactor building of one unit of Ignalina RBMK-1500 nuclear power plant

The Baltic region is usually regarded as a region of low seismicity. In comparison to Latvia, Estonia and Belarus, Lithuania has the lowest seismic activity. However, the available data indicates, that there is a possibility of strong earthquakes occurring. The maximum possible earthquake in the surroundings of INPP is estimated to have a magnitude of 4.5 and a focal depth of 5 to 8 km.

4.3 Earthquake Resistant Design

For Soviet designed nuclear power plants two levels of earthquakes were taken into account, i.e. the design earthquake and the maximum possible earthquake. The first is a maximum earthquake which may happen during the service life of the plant . The second is the maximum possible earthquake in the area . For INPP the design and maximum possible earthquakes have peak ground accelerations, respectively, of 0.012 to 0.05g and 0.025 to 0.1g. This was considered appropriate for the seismic activity of this region.

Depending on their function during and after an earthquake, all buildings and equipment were subdivided into different seismic categories. For each category different seismic design criteria were applicable. The earthquake analyses were performed using a response spectrum method.

This approach to earthquake resistant design has not been adopted in Western standards. A review of the structural integrity of the plant was carried out in 1995. Measures aimed at strengthening the building structures and equipment were considered and judged to be uneconomical. Consequently it was decided to install an earthquake warning system as a first step to increase plant safety in the event of an earthquake.

5. CHARACTERISTICS OF THE IGNALINA SEISMIC ALARM SYSTEM

5.1 Description of the System

Usually, a time period of 2 seconds is required for the insertion of the control rods of the nuclear reactor. After that, the nuclear thermal capacity is strongly reduced and the reactor core is prevented from meltdown in the case of a severe accident. A core meltdown would entail the risk of radioactivity release to the environment.

The existing earthquake early warning systems, see Section 3 of the present report, require by far more than 2 seconds to indicate a seismic event. Therefore, a system had to be designed specifically suitable to nuclear power plants. In the INPP Seismic Alarm System (SAS), six seismic stations using accelerometers are installed at a distance of 30 km from the power plant. The signals are transmitted to INPP by radio waves, which requires virtually no time. Assuming a seismic shear wave velocity of 3.5 km/s, the pre-warning time would be 8.5 s. In practical terms, this is reduced to 4 s by the required transfer and processing times. It is concluded that the Ignalina SAS is able to effectuate the insertion of the control rods before the arrival of the damaging seismic waves, i.e. the shear waves, at the NPP.

At each seismic station three accelerometers are located at substations 500 m apart. The SAS accelerometers input to seismic switches which are factory preset to an initial acceleration threshold of 0.025 g. When this threshold is exceeded the seismic switch produces an alarm signal. These signals are digitally encoded and sent via a separate transmission channel to the control centre.

Here a 2-out-of-3 voting logic is used to determine if a seismic event has occurred and to generate a seismic alarm in the main reactor control rooms.

The alarm system is complemented by a seismic monitoring system (SMS), which provides seismic data recording and processing. One seismometer is located at each seismic stations. In addition, the SMS includes sensors inside of the reactor building and on two key items of equipment namely on the cooling water pump and on the steam separator drum of each unit. The data is processed by two redundant central computers located at each unit.

The program of works foresees the installation of the seismic stations, the telemetry system and the seismic evaluation system at the end of 1997.

After implementation, the Ignalina Nuclear Power Plant will be one of the first nuclear power plants in the world to have an earthquake early warning system using both accelerometers with seismic switches and seismometers.

5.2 Technical Outline of the Seismic Alarm System

The SAS is outlined in the Block Diagram, Fig. 4. The SAS system is seismically qualified. It is based on three separate measurements, transmission and reception channels and a 2-out-of-3 voting logic. This gives a high degree of reliability, operability and protection against false alarms. Each seismic station has three substations designed to Western seismic standards. An external power supply is required for each seismic station. Triaxial accelerometers are used as sensors in each substation. To reduce the effect of signal noise the analogue signals from each sensor are digitised at its substation. The seismic switches and radio frequency data transmission telemetry equipment of the three substations are located in the cabin of the seismic station. The signals of the seismic switches are transmitted to the power plant by radio communication in the Ultra High Frequency (UHF) band. UHF communication requires line of sight conditions, which poses comparatively little difficulty in the flat area of Lithuania.

The trigger threshold of each seismic switch is software adjustable. The initial setting of 0.025g will be assessed after a trial period and optimised. The records of the Seismic Monitoring System will be used for this assessment.

Receiving antennas are mounted on the reactor building roof. The telemetry equipment is located nearby. For each seismic station, this equipment and the associated cabling is separated into

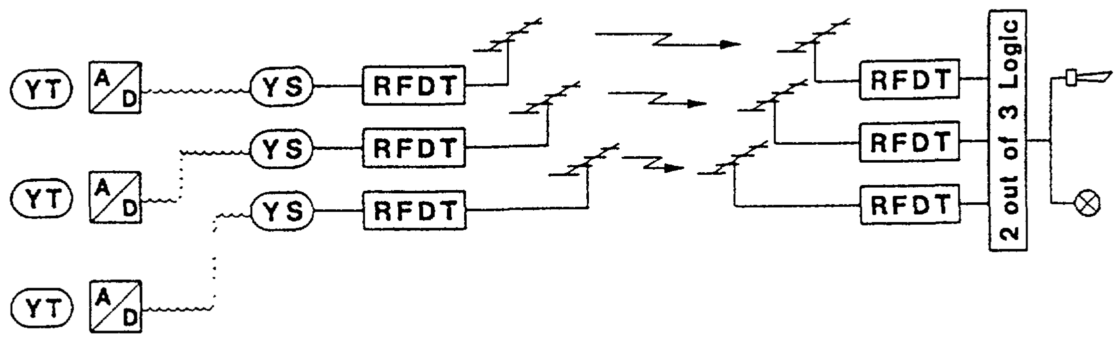


Fig. 4 Seismic Alarm System SAS, Block Diagram, one of six stations

three measurement channels up to the 2-out-of-3 voting logic located adjacent to the reactor control room. This logic initiates the alarm signals to the main control room for each reactor.

The seismic alarm system for INPP has been designed to provide an economical and adequately comprehensive solution to concerns regarding the seismic integrity, with respect to Western standards, of some of the INPP buildings and equipment. The use of accelerometers and seismic switches by the seismic alarm system maximises the available warning time.

5.3 Technical Outline of the Seismic Monitoring System

The SMS is outlined in the Block Diagram, Fig. 5. The system includes six seismometers, one at the cabin of each seismic station, plus one seismometer and two accelerometers at INPP. Each seismometer is located in a bore hole. The location of the seismic stations have been chosen so as to be remote from environmental noise.

Four triaxial accelerometers are located in the reactor building three on the base of the building and one at the 20 m level. Biaxial accelerometers are located on the cooling water pump and on the steam separator drum in each unit.

The values measured by the field SMS seismometers are combined into data packets, which are transmitted to the power plant by radio communication. Separate radio frequencies are used to permit continuous transmission of data. At the power plant data is digitised and input to the central processor using RS-485 links. Sampling rates up to one or two hundred samples per second will be possible. Data is stored and processed in two central computers.

The provision of the seismic monitoring covering off-site and on-site locations permits the determination of the seismic transfer function from the off-site locations to the NPP and a quantitative measurement of the building and equipment response to seismic activity. This can be used in two ways. Firstly, to assess the stress caused by seismic activity in order to confirm that the integrity of the plant has not been compromised. Secondly, in conjunction with a reactor seismic model to identify potentially susceptible plant. Once identified the necessary structural improvements can be determined.

5.4 Central Data Recording and Processing

On the central computers, a PC based QNX multitasking, multi-user operating system is installed. The management of data acquisition is performed by the SEISLOG application software and the analysis is accomplished using the SEISAN earthquake analysis software. SEISLOG and SEISAN were developed by the Institute of Solid Earth Physics, University of Bergen, Norway. The SEISLOG data acquisition system is used as the major data collection system in the national seismic networks of Norway, United Kingdom, Ireland and in several countries of Central America. In addition, SEISLOG is used at about 40 stations in eight other countries in Europe, Africa and Asia.

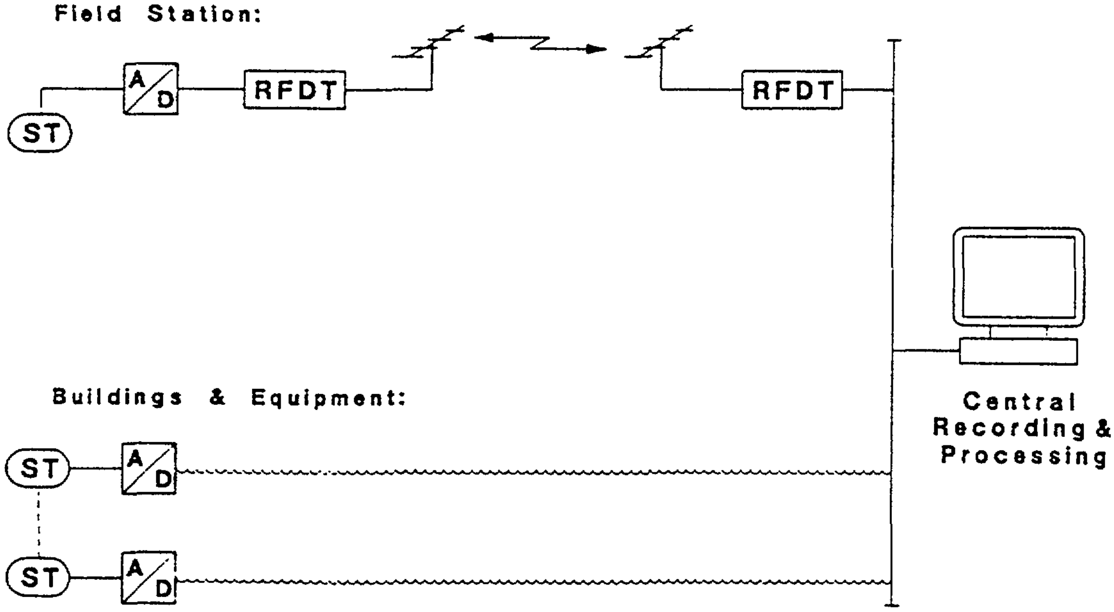
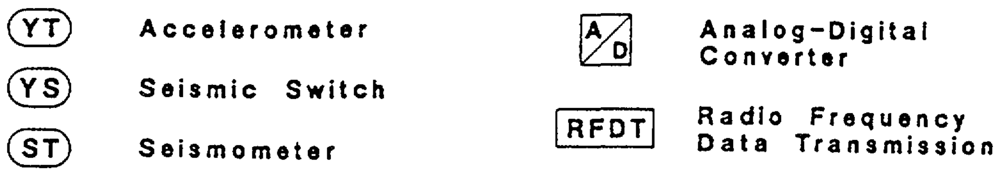


Fig. 5 Seismic Monitoring System SMS, Block Diagram, one of six field stations, typical instrumentation of buildings and equipment



Legend to the Block Diagrams

The digitised data packets, sent from the Seismic Monitoring System, are displayed on the central computer terminal by SEISLOG. When each page of data is accumulated, it is printed out in the analogue mode. This approach of printout is recommended because it provides the least cost and most reliable method of obtaining hard copies of the signals. Alternative plotting methods are selectable. SEISLOG has flexible user defined trigger criteria which can be tailored to both local and distant earthquakes. The system can be set up with up to five different trigger criteria sets in order to independently trigger on local and distant earthquakes.

The data sets are transferred from SEISLOG to the processing and analysis software SEISAN by floppy disk, tape, removable disk, modem or Ethernet. SEISAN has been in operation since 1988. It is used as the main processing tool in the SEISLOG installations mentioned above. SEISAN has the advantage of being a complete system with a data base and integrated processing tools. In addition to working smoothly with data from SEISLOG, SEISAN can also process data from many well known data acquisition systems and data banks. In particular, it has been well integrated with International Seismological Centre data formats. Large amounts of data can be processed either manually or automatically.

SEISAN can calculate all normally used magnitudes. It locates earthquakes with the latest global model (IASP91) or with user selectable models. Earthquake location can be done with several thousand stations and arrival times. More than 100 types of phases can be used. The data base can be searched for more than twenty different criteria. The results are displayed in terms of the hypocentral distribution in time and space and a statistical analysis can also be carried out.

A seismic model of the reactor will be developed and the predicted response spectra compared with monitored data. By developing the seismic model of the reactor an assessment will be possible of the stress caused by seismic activity to different parts of the reactor building and equipment.

5.5 Future Developments

The following future developments in the field of earthquake early warning systems are foreseen:
1) discrimination techniques to distinguish seismic activity from environmental noise;
2) investigation into satellite transmission systems for data communication over long distances or hilly areas;
3) development of stand alone seismic stations which are independent of an external power supply;
4) closer and faster data transfer links with international seismological monitoring organisations;
5) improved data telemetry equipment which can transmit more data in a given band-width.

6. CONCLUSIONS

A seismic "fence" having a radius of 30 km will be installed around the Ignalina Nuclear Power Plant to provide an alarm before potentially damaging earthquake tremors reach the reactors. The alarm threshold is preset at 0.025 g, which will be adjusted according to the experience gained.

Seismic safety upgrading of a nuclear power plant by means of a seismic fence is an economical solution for existing plants with inadequate or unknown seismic resistance of vital components in the case of strong earthquakes. It is also a recommended solution for existing power plants where earthquakes have occurred which exceed the level anticipated at the time of design and construction of the power plant.

This system cannot only reduce the consequences of a reactor accident caused by an earthquake but help confirm plant integrity following an earthquake.

Because of the many benefits and the low cost, earthquake early warning systems have excellent prospects in connection with increasing safety demands for nuclear power plants.

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