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NUCLEAR ENERGY AGENCY
COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

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Workshop on the Seismic Re-evaluation of all Nuclear Facilities

Workshop Proceedings

Ispra, Italy
26-27 March, 2001

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ORGANISATION FOR ECONOMIC CO-OPERATION AND DEVELOPMENT

Pursuant to Article 1 of the Convention signed in Paris on 14th December 1960, and which came into force on 30th September 1961, the Organisation for Economic Co-operation and Development (OECD) shall promote policies designed:

- to achieve the highest sustainable economic growth and employment and a rising standard of living in Member countries, while maintaining financial stability, and thus to contribute to the development of the world economy;
- to contribute to sound economic expansion in Member as well as non-member countries in the process of economic development; and
- to contribute to the expansion of world trade on a multilateral, non-discriminatory basis in accordance with international obligations.

The original Member countries of the OECD are Austria, Belgium, Canada, Denmark, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, the Netherlands, Norway, Portugal, Spain, Sweden, Switzerland, Turkey, the United Kingdom and the United States. The following countries became Members subsequently through accession at the dates indicated hereafter: Japan (28th April 1964), Finland (28th January 1969), Australia (7th June 1971), New Zealand (29th May 1973), Mexico (18th May 1994), the Czech Republic (21st December 1995), Hungary (7th May 1996), Poland (22nd November 1996), Korea (12th December 1996) and the Slovak Republic (14th December 2000). The Commission of the European Communities takes part in the work of the OECD (Article 13 of the OECD Convention).

NUCLEAR ENERGY AGENCY

The OECD Nuclear Energy Agency (NEA) was established on 1st February 1958 under the name of the OEEC European Nuclear Energy Agency. It received its present designation on 20th April 1972, when Japan became its first non-European full Member. NEA membership today consists of 27 OECD Member countries: Australia, Austria, Belgium, Canada, Czech Republic, Denmark, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Japan, Luxembourg, Mexico, the Netherlands, Norway, Portugal, Republic of Korea, Spain, Sweden, Switzerland, Turkey, the United Kingdom and the United States. The Commission of the European Communities also takes part in the work of the Agency.

The mission of the NEA is:

- to assist its Member countries in maintaining and further developing, through international co-operation, the scientific, technological and legal bases required for a safe, environmentally friendly and economical use of nuclear energy for peaceful purposes, as well as
- to provide authoritative assessments and to forge common understandings on key issues, as input to government decisions on nuclear energy policy and to broader OECD policy analyses in areas such as energy and sustainable development.

Specific areas of competence of the NEA include safety and regulation of nuclear activities, radioactive waste management, radiological protection, nuclear science, economic and technical analyses of the nuclear fuel cycle, nuclear law and liability, and public information. The NEA Data Bank provides nuclear data and computer program services for participating countries.

In these and related tasks, the NEA works in close collaboration with the International Atomic Energy Agency in Vienna, with which it has a Co-operation Agreement, as well as with other international organisations in the nuclear field.

COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

The NEA Committee on the Safety of Nuclear Installations (CSNI) is an international committee made up of scientists and engineers. It was set up in 1973 to develop and co-ordinate the activities of the Nuclear Energy Agency concerning the technical aspects of the design, construction and operation of nuclear installations insofar as they affect the safety of such installations. The Committee's purpose is to foster international co-operation in nuclear safety amongst the OECD Member countries.

CSNI constitutes a forum for the exchange of technical information and for collaboration between organisations which can contribute, from their respective backgrounds in research, development, engineering or regulation, to these activities and to the definition of its programme of work. It also reviews the state of knowledge on selected topics of nuclear safety technology and safety assessment, including operating experience. It initiates and conducts programmes identified by these reviews and assessments in order to overcome discrepancies, develop improvements and reach international consensus in different projects and International Standard Problems, and assists in the feedback of the results to participating organisations. Full use is also made of traditional methods of co-operation, such as information exchanges, establishment of working groups and organisation of conferences and specialist meeting.

The greater part of CSNI's current programme of work is concerned with safety technology of water reactors. The principal areas covered are operating experience and the human factor, reactor coolant system behaviour, various aspects of reactor component integrity, the phenomenology of radioactive releases in reactor accidents and their confinement, containment performance, risk assessment and severe accidents. The Committee also studies the safety of the fuel cycle, conducts periodic surveys of reactor safety research programmes and operates an international mechanism for exchanging reports on nuclear power plant incidents.

In implementing its programme, CSNI establishes co-operative mechanisms with NEA's Committee on Nuclear Regulatory Activities (CNRA), responsible for the activities of the Agency concerning the regulation, licensing and inspection of nuclear installations with regard to safety. It also co-operates with NEA's Committee on Radiation Protection and Public Health and NEA's Radioactive Waste Management Committee on matters of common interest.

FOREWORD

The Committee on the Safety of Nuclear Installations (CSNI) of the OECD-NEA co-ordinates the NEA activities concerning the technical aspects of design, construction and operation of nuclear installations insofar as they affect the safety of such installations.
The Integrity and Ageing Working Group (IAGE WG) of the CSNI deals with the integrity of structures and components, and has three sub-groups, dealing with the integrity of metal components and structures, ageing of concrete structures, and the seismic behaviour of structures. This workshop was proposed by the sub-group dealing with the seismic behaviour of structures.

Seismic re-evaluation is identified as the process of carrying out a re-assessment of the safety of existing nuclear facilities for a specified seismic hazard. This may be necessary when no seismic hazard was considered in the original design of the plant, the relevant codes and regulations have been revised, the seismic hazard for the site has been re-assessed or there is a need to assess the capacity of the plant for severe accident conditions and behaviour beyond the design basis. Re-evaluation may also be necessary to resolve an issue, or to assess the impact of new findings or knowledge.

In 1997, CSNI recognised the increasing importance of seismic re-evaluation for nuclear facilities throughout the world. It prepared a status report on seismic Re-evaluation NEA/CSNI/R(98)5 which summarized the current situation for Member countries of the OECD. The report suggested a number of areas of the seismic reevaluation process, which could be considered in the future. In May 2000, the seismic sub-group reviewed these suggestions and determined that it was timely to address progress on this topic through this workshop. The workshop focused on methods and acceptance criteria and, on countermeasures and strengthening of plant.

The workshop had 2 technical sessions listed below devoted to presentations, and a 3rd session devoted to a discussion of the material presented and to the formulation of workshop conclusions to update conclusions of the 1998 report.

Session 1

- Methods and acceptance criteria
- Benefits and disadvantages of the various methods of re-evaluation (Seismic PSA, Margins, deterministic, databases, tests …) in particular circumstances
- Role and scope of the peer review process
- Definition of the scope of the plant to be selected for the re-evaluation process
- Differences between re-evaluation and design criteria

Session 2

- Countermeasures/strengthening
- Civil engineering structures
- Post earthquake procedures and measures
- Strategies and priorities
- Recent innovation or research outputs

In the area of the seismic behaviour of structures, the CSNI is currently preparing among others a workshop on relations between seismological data and seismic engineering analysis to evaluate uncertainties and margins through a better description of real ground motion spectrum as opposed to a ground response design. Short reports on "lessons learned from high magnitudes earthquakes with respect to nuclear codes and standards" are under preparation and will cover several recent earthquakes.

Seismic reports issued by the group since 1996 are:

- NEA/CSNI/R(1996)10 Seismic shear wall ISP: NUPEC's seismic ultimate dynamic response test: comparison report, 1996. also referenced as: OCDE/GD(96)188

- NEA/CSNI/R(1996)11 Report of the task group on the seismic behaviour of structures: status report, 1997. also referenced as: OCDE/GD(96)189

- NEA/CSNI/R(1998)5 Status report on seismic re-evaluation, 1998.

- NEA/CSNI/R(1999)28 Proceedings of the OECD/NEA Workshop on Seismic Risk, CSNI PWG3 and PWG5, Tokyo, Japan 10-12 August 1999.

- NEA/CSNI/R(2000)2/VOL1 Proceedings of the OECD/NEA Workshop on the "Engineering Characterisation of Seismic Input, BNL, USA 15-17 November 1999 -

- NEA/CSNI/R(2000)2/VOL2 Proceedings of the OECD/NEA Workshop on the "Engineering Characterisation of Seismic Input, BNL, USA 15-17 November 1999

The complete list of CSNI reports, and the text of reports from 1993 onwards, is available on http://www.nea.fr/html/nsd/docs/

Acknowledgement

Gratitude is expressed to the European Commission Joint Research Centre, Ispra (VA), Italy for hosting the workshop as well as to the Organization for Economic Co-operation and Development (OECD) / Nuclear Energy Agency (NEA) / Committee on the Safety of Nuclear Installations (CSNI) / Integrity and Aging Working Group (IAGE) (Integrity of Components and Structures) for sponsoring our work.
Thanks are also expressed to chairmen of the sessions for their effort and co-operation.

The organizing Committee members were:

Mr John Donald, HSE (UK)
Mr Jean-Dominique Renard, Tractebel (B)
Dr Vito Renda, JRC/ISPRA (I)
Prof. Pierre Labbé, IAEA
Dr. Tamas Katona, PAKS (HU)
Dr Andrew Murphy, USNRC (USA)
Mr Eric Mathet, OECD/NEA

OECD/NEA WORKSHOP ON THE SEISMIC RE-EVALUATION OF ALL NUCLEAR FACILITIES

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Ispra, Italy

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SESSION 2:
COUNTERMEASURES/STRENGTHENING
Chairman: J. D. Renard

Seismic Assessment of Existing Facilities Using Non Linear Modeling
Some General Consideration and Examples of Validation on Experimental Results.

D.Combescure, P.Sollogoub [(1)]

(1) Seismic Mechanic Study Laboratory, DEN/DM2S/SEMT, CEA Saclay, 91191 Gif/Yvette Cedex, France

ABSTRACT

This paper aims at presenting the methodology and the non-linear modelling used for the seismic evaluation of existing nuclear facilities. The main problems to be solved during the seismic assessment of existing facilities regarding the non linear modeling such as the definition of damage index according safety requirements are discussed in this paper.

INTRODUCTION

In the oldest industrial countries specially with moderate seismicity such as France, the main part of the building stock has been designed before the application of the modern seismic code. Because of the absence of any seismic consideration at the period of the construction or the modification of the action levels, many structures do not satisfy the actual requirements. Due to safety reasons, the case of nuclear facilities and plants may be critical and requires detailed seismic evaluations. The application of simplified procedures used for design -elastic computation and reduction by a q-factor- tends to give unrealistic results and, overall, masks the critical points for the facility safety.

This paper aims at presenting some general consideration about the non linear modelling of existing buildings and nuclear facilities and highlighting some difficulties to be faced by the engineers during the analysis of the results of the non linear computations.

Firstly, the main non linear models used under dynamic loading are briefly reminded. The second part show an application of these models to the study of the 1/3rd scaled Camus wall structures tested on shaking table in Saclay. These dynamic tests have been conducted using 2 types of signal representative of far-field and near-field moderate magnitude earthquakes.

In the last part, the full scaled 4-storey frames representative of the buildings of the 1940-1970 period tested with the pseudodynamic testing method in Ispra is analysed.

These 2 last parts shows the importance of the experimental results for the validation of the non linear models and their acceptance for the seismic assessment of existing nuclear facilities.

DESCRIPTION OF THE NON LINEAR MODELLING

Examples of Non Linear Constitutive Laws

The non-linear numerical models are usually divided in two classes :

- the global models reproducing the behaviour of complete structural elements at a reduced cost can be used to study the behaviour of complete structures under complex static and dynamic loading. At this level of modelling, beams and columns are usually replaced by beam elements with concentrated hinges, shear walls by global shear beam model and infill panels by two diagonal trusses supporting only compression forces.
- the local models allow to study structural components knowing only the characteristics of the basic materials but are much more time consuming to use - not only in term of cpu time but also in term of preparation of datas and processing of results -.

These two levels can be used in a complementary way : the local modelling allows used to identify the parameters of the global models and highlight the limitations of these models (for example, what are the limits of the assumption of the 2 diagonal for the infill panels). Such approach has been applied with Castem2000 finite element code [1] for the analysis of infilled frame [2] and shear wall structures [3].

A Timoshenko Beam Element With Non Linear Shear Behaviour

The seismic analysis of complete building structures with dynamic or simplified push-over analysis requires simplified non linear finite elements. The behavior of reinforced concrete members such as columns, beams but also structural walls can be very well reproduced using non linear beam elements with fiber type assumptions at the section level.

This modeling is based on a geometrical description (Fig 1) of the beam section in fibers (or layers in 2D). The axial and shear strains in each fiber are deduced directly from the average axial $\bullet_x$ and shear strains $\bullet_y$, $\bullet_z$, the curvatures (in flexion $\bullet_y$, $\bullet_z$ and torsion $\bullet_x$) of the beam element and the section geometry.

$$(\varepsilon_x)_i = \varepsilon_x - y_i \cdot \phi_z + z_i \cdot \phi_y$$

$$(\gamma_y)_i = \gamma_y - z_i \cdot \phi_x \text{ and } (\gamma_z)_i = \gamma_z + y_i \cdot \phi_x$$

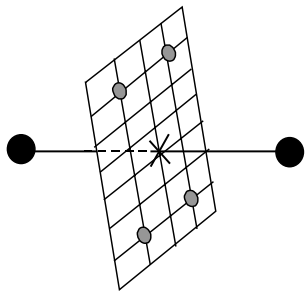
The normal force N_x, bending moments M_y, M_z, shear forces T_y, T_z and twisting moment M_x are calculated by integrating the axial and shear stresses in the section.

$$N_x = \int_S \sigma_x dS \,,\; M_y = \int_S z \cdot \sigma_x dS \text{ and } M_z = -\int_S y \cdot \sigma_x dS$$

$$T_y = \int_S \tau_y dS \,,\; T_z = \int_S \tau_z dS \text{ and } M_x = \int_S (y \cdot \tau_z - z \cdot \tau_y) dS$$

Each fiber supports a uniaxial law $\bullet\bullet\bullet)$ representative of concrete or steel behaviour. Fig 2 and Fig 3 show the laws used in the present study respectively for concrete (with softening in compression and tension) and for steel (with hardening, Bauschinger effect and buckling).

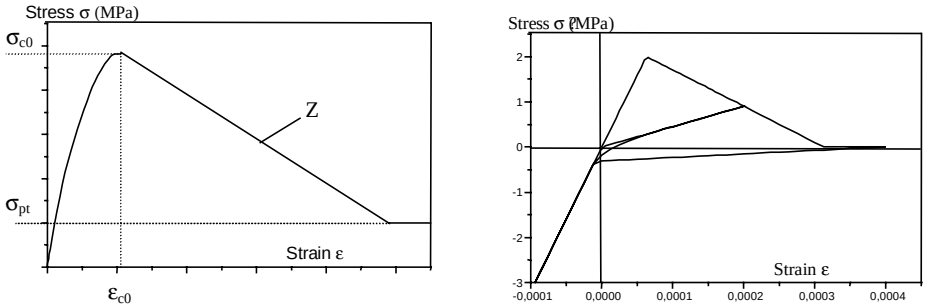
A simple Timoshenko beam element has been adopted in order to allow shear distortion and so the use of non linear constitutive laws not only for bending but also for shear and torsion. In order to avoid shear locking, this 3D beam element has a unique Gauss point and the axial strain, curvature and shear strain remain constant on the element.
Details of this beam element and the uniaxial constitutive laws can be found in [4] and [5].



Beam level: $(u, \theta) \Rightarrow (\varepsilon_0, \phi, \gamma)$ (M, N, T)

Fibre level: $(\varepsilon, \gamma) \Rightarrow (\sigma_{xx}, \tau_{xy}, \tau_{xz})$

Fig. 1. Non linear Fiber Beam Model



a/ Behaviour in compression **b/ Behaviour in traction**

Fig. 2. Uniaxial Constitutive Law for Concrete

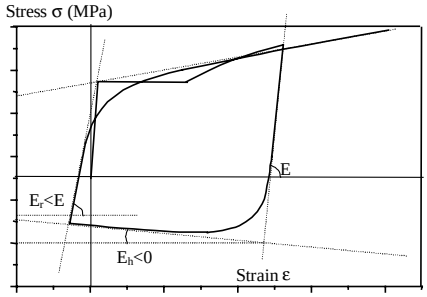


Fig. 3. Menegotto-Pinto uniaxial law with Bauschinger effect and buckling for steel

Some remarks on the Influence of the Construction Details on Modelling

The accuracy of the modeling and the prediction of failure depends strongly on the capacity to take into account of the construction details specially for the reinforced concrete frame structures.
For example, the concrete law shown Fig 2 although it is uniaxial can be directly influenced by the confinement of the stirrups by modifying the ultimate compressive strength and the softening slope Z. A decrease of softening due to higher confinement ratio improves the curvature ductility capacity. The

confinement can also be taken into account by modifying the local failure criteria (let say the concrete ultimate strain).
Another major difficulty in the modeling of frame structures up to flexural failure is the localization phenomena due to softening or limited hardening after yielding of the steel bars. This phenomena makes the local results (curvature and strain demands) strongly dependent on the mesh size and requires to fix the length of the elements –the plastic hinges- where damage may concentrate. This is equivalent to consider plastic rotations or chord rotations as failure criteria.
Priestley [6] gives some formulae to determine the length of the plastic hinge H_{hinge}.
$H_{hinge} = 0.08\ H_{column} + 6\ d_{bar}$
This length depends not only on the column height H_{column} but also on the steel bars diameter d_{bar} since spread of steel yielding in the footing has been evidenced by several experimental results.

A constitutive law for anchorages and lap splices
A specific constitutive law for has been introduced in the fibre model in order to check the possible failure of lap splices and anchorages. The approach already implemented for bridge piers by Monti [7] and Xiao [8] has been adopted.
This uniaxial law • • •) is based on the partition of the total strain • between the strain in the steel bar $•_s$ and the slippage between steel and concrete s (Fig 4-a). This partition can be written incrementally:

$$• • • = • •_s + •s/L_{anc}.$$

with

$$• •_s = •. • • • \text{ and } •s = L_{anc}.(1-• •. • •$$

L_{anc}: Length of anchorage or splices
• : Partition factor between the 2 types of deformations.
The axial stress in the steel bar $•_s$ and the bond stress • are given by 2 appropriate constitutive laws respectively for steel rebar $•_s(•_s)$ and for bond slip • •(s). A law similar to the Eligehausen law has been adopted for bond slip (Fig 4-c, [9]).
The partition factor • can be calculated iteratively with the static equilibrium between the force in the steel bar F_{steel} and the bond stress F_{bond} which is supposed constant on the complete length of the anchorage or lap splices (Fig 4-b). An iterative modified Newton-Raphton algorithm is used to verify the equilibrium.
$• F_{steel} + • F_{bond} = 0 = f(•)$

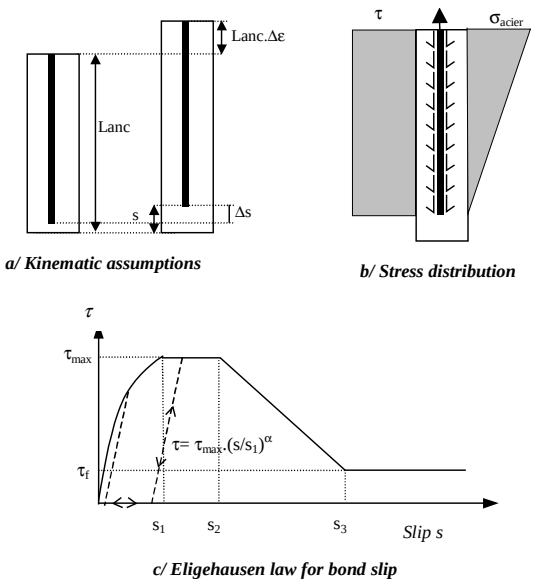


Fig 4. Phenomenological uniaxial law for anchorage and lap splices

Application to the columns with insufficient lap splices

The fiber beam element has been applied to the modeling of a series of flexural columns tested by Aboutaha at Austin University [10]. For the column FC 15, the lap splices failed before developing the flexural strength of the section. A unique beam element and the special law for anchorage and lap splices have been considered for the plastic hinge at the base of the column.

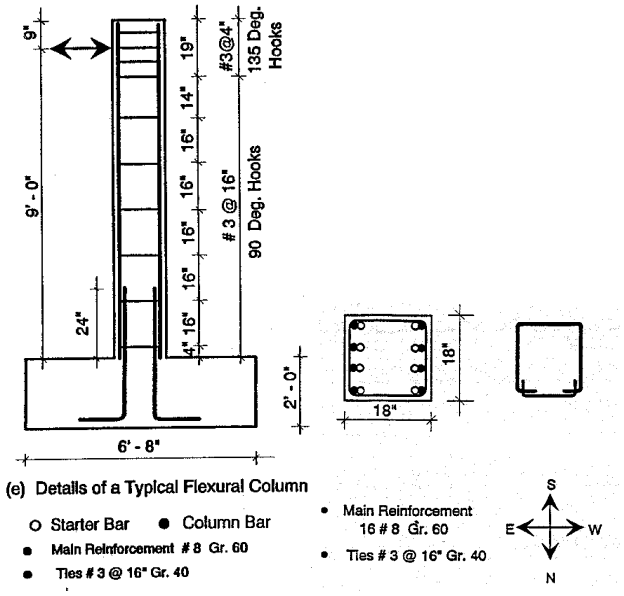
The Priestley formulae gives a length of plastic hinge of 37.2 cm for a column height equal to 274.3 cm and diameter of the steel bars of 25.4mm (#8).

The physical length of $20d_{bar}$ has been considered for the lap splices. The bond characteristics recommended by Eligehausen for unconfined concrete (bond strength $\bullet_u$=5 MPa) has been chosen for the bond slip model.

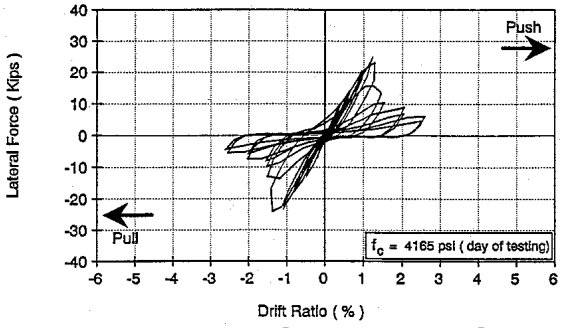
The upper part of the column has been discretized by 7 Timoshenko beam elements with non linear constitutive law for concrete and steel.

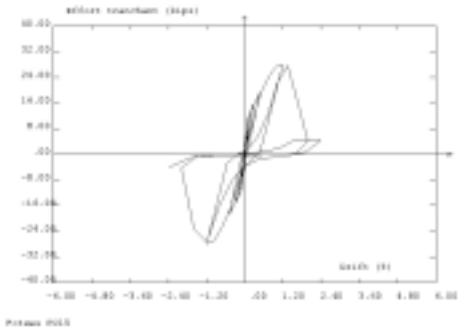
The failure mechanism and the global strength observed during the tests have been well captured by the numerical model (Fig 5). Important softening can be observed also in the computation after having reached the maximum strength which is equal to 124 kN (27.9 kips) in the calculation versus 111 kN (25 kips) measured experimentally. These values can be compared to the strength of the FC 17 flexural column which is equal to the FC15 column but strengthened with a steel jacket: 147 kN (33 kips) in the calculation and 142 kN (32 kips) experimentally.

Despite this good agreement between numerical and experimental results, the model adopted in this work for the anchorage and the lap splices can present some difficulties if columns with higher length of splices ($30d_{bar}$ or $40d_{bar}$ which are current values in some part of Europe such as in France) but insufficient numbers of stirrups are considered since the bond strength does not depend on the local ductility demand in the steel bars. The lack of experimental results on reinforced concrete columns with this type of detailing used in Europe must also be highlighted.



a/ Column geometry and reinforcement





b/ Experimental curve **c/ Numerical results**

Fig. 5. Analysis of a flexural column with failure of the lap splices

PUSH-OVER ANALYSIS OF A 1/3RD SCALED 5 STOREY STRUCTURE

The present chapter shows an application of the simplified push over method to the analysis of a 5 storey wall structure with a total mass of 36 tons tested on the Azalee shaking table in Saclay.

DESCRIPTION OF THE CAMUS 3 WALL STRUCTURE

This structure is part of a series of three 1/3rd Camus wall structures made of 2 reinforced concrete bearing walls and 5 storeys (Fig 4 and [11]). Each specimen has a total mass of 36 tons. Two types of signal have been applied to these structures: an artificial signal named as Nice has been used for the majority of the seismic tests while intermediate seismic tests have been performed with recorded signals (San Francisco and Melendy Ranch signals) representative of near-field moderate magnitude earthquakes.

For the Camus 3 specimen designed according EC8 provisions, the level of the seismic input has been determined with a push over analysis. The non linear fiber model has been used for the modeling of the structural walls. A static computation has been performed with a monotonic horizontal loading. Because of the difficulties to impose horizontal forces with constitutive laws with softening, loading is imposed to one point at 2/3rd of the building height and is controlled in displacement. This assumption is not so bad since the mass of the system is regularly on the building height and the design obliges the formation of a unique plastic hinge at the base of the building. For real structures, this loading can be very uncorrect.

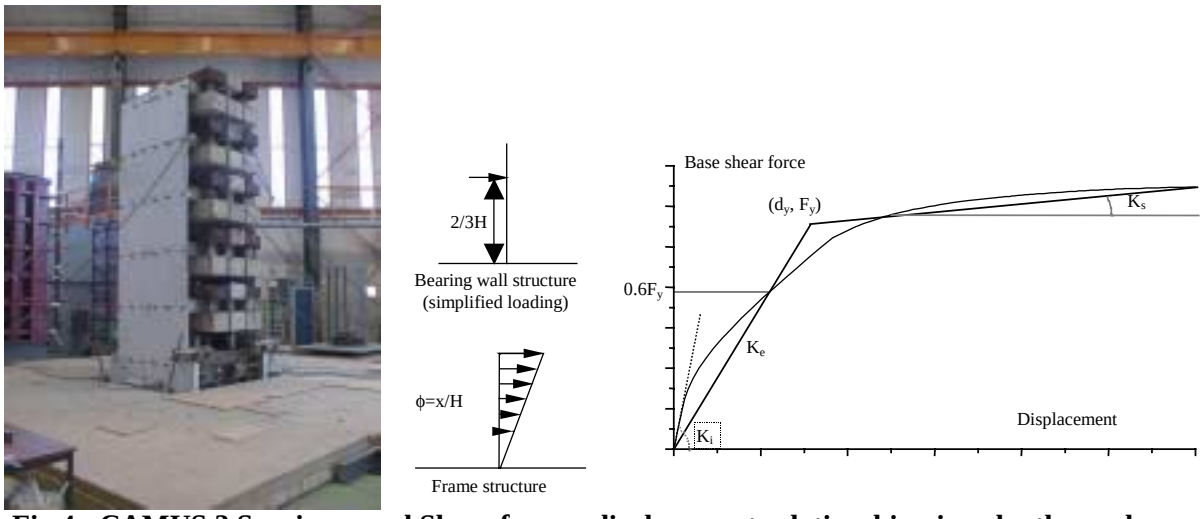


Fig 4: CAMUS 3 Specimen and Shear force – displacement relationship given by the push over analysis

Determination of the spectral acceleration vs spectral displacement curve and the performance point

The base shear-top displacement curve (Fig 4) is converted to spectral acceleration-spectral displacement curve (Fig 6) using the following relationships:

$$d_{top}=PF\cdot S_d \quad \text{with} \quad PF=\frac{\left(\sum m_i\cdot\phi_i\right)\phi_i}{\left(\sum m_i\cdot\phi_i^{\,2}\right)}$$

$$T_{base}=\alpha\cdot M\cdot S_a \quad \text{with} \quad \alpha=\frac{\left(\sum m_i\cdot\phi_i\right)^2}{\left(\sum m_i\right)\left(\sum m_i\cdot\phi_i^{\,2}\right)}$$

M : Total mass

M_i : Mass at floor i

• $_i$: Fundamental deformed shape ($\phi_i=\left(\frac{x}{H}\right)^{1.5}$ for a wall structure and $\phi_i=\frac{x}{H}$ for a frame structure)

The spectral acceleration versus spectral displacement curve is very useful because of the possible direct comparison with the design and earthquake response spectra. Fig 5 shows the design response spectra of the PS 92 French design code and a comparison of the elastic response spectra of the 2 different signals used for the seismic tests on Camus 1 structure. The San Francisco signal is representative of a near field moderate magnitude earthquake and although the peak ground acceleration is higher, the displacement demand for the low value of frequency is much reduced than the artificial Nice signal. Fig 6b shows also a comparison between the artificial Nice signal and the Melendy Ranch signal used for the Camus 3 seismic tests.

The determination of the performance point requires the calculation of an equivalent damping since the method is similar to a linear method based on a secant stiffness. The formulae given by Priestley has been used:

$$\zeta=\zeta_e+\alpha\cdot\left(1-\frac{1}{\mu^{\beta}}\right) \quad \text{with} \bullet\bullet \text{ between 20 et 30\% and } \bullet \text{ between 0.5 et 1}$$

The value of elastic damping has been chosen equal to the measured damping value determined with a random noise which is lower than the suggested value by [6] for reinforced concrete building: the measured value corresponding to the first eigenmode is equal to 2% versus 5% for the suggested value.

$$\zeta = 2 + 20 \cdot \left(1 - \frac{1}{\mu^{0.5}} \right) \%$$

Main results

The determination of the performance point is not direct if one wants to determine the displacement corresponding to a certain earthquake level: one must iterate to obtain the coherence between the displacement and ductility demand and the damping value. In order to avoid iterations, it was preferred to determine the level of acceleration corresponding to a certain displacement in the present study.

Table 1 shows the main results: the level of earthquake necessary to obtain each target displacement and ductility demand is given for the 2 types of earthquake. This table shows that, for the low level of input, the Melendy Ranch earthquake is more demanding than the Nice earthquake which is in accordance with the elastic response spectra of the 2 signals. When the structure becomes strongly non linear, it is the opposite: the Nice signal is more demanding.

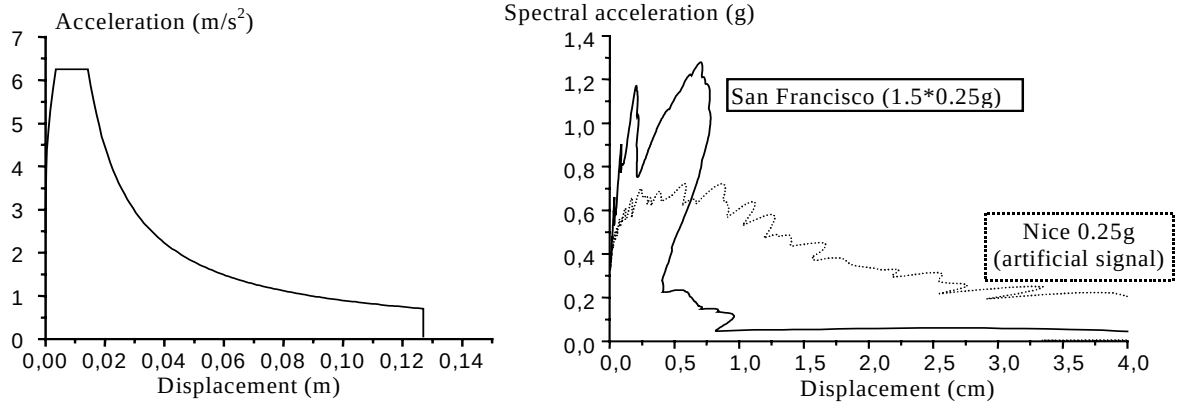
Table 2 shows the main experimental results. The comparison with Table 1 shows a quite good agreement with the predictive calculations.

Table 1: Correspondance between displacement and seismic level for 2 types of signal

Spectral displacement	*Top displacement*	*Ductility*	*Equivalent damping*	*Seismic level for Nice*	*Seismic level for Melendy Ranch*
4mm	5.6mm	<1	2%	0.14g	0.13g
9.3mm	12.9mm	1	2%	0.2g	0.26g
15mm	20.8mm	1.61	4.2%+2%	0.48g	0.54g
20mm	27.8mm	2.15	6.4%+2%	0.58g	0.95g
30mm	41.7mm	3.22	8.85%+2%	0.83g	1.67g
40mm	55.6mm	4.30	10.3%+2%	1.01g	2.14g

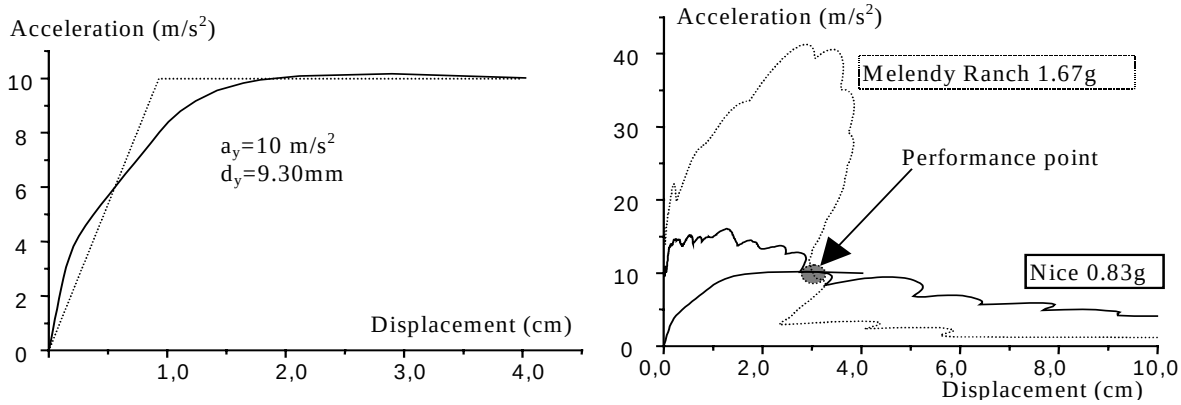
Table 2: Main experimental results

Signal	*Nice 0.42g*	*Nice 0.22g*	*Melendy Ranch 1.35g*	*Nice 0.64g*	*Nice 1.0g*
Displacement at 4th storey	/	/	18.5 mm	20.5 mm	34.6 mm
Displacement at 5th storey	7.0 mm	4.3 mm	29.2 mm	27.5 mm	47.1 mm
Top displacement (at 6th storey)	/	/	/	34.9 mm	58.8 mm



P S92 response spectra ***Response spectra of Nice et San Francisco***

Fig 5 : ADRS response spectra of the French PS92 design code and Nice and San Francisco signals



Spectral acceleration vs displacement for the wall structure Camus III ***Response spectra for a spectral displacement of 30mm (Damping=2+8.85%)***

Fig 6: Determination of the loading programme of CAMUS III seismic tests

TIME HISTORY ANALYSIS OF A 4 STOREY REINFORCED CONCRETE FRAME

Characteristics of the experimental campaign on the 4 Storey Structure

Two full-scaled reinforced concrete specimens have been tested with the pseudodynamic testing method in JRC Ispra. The 2 specimens have identical concrete frame designed essentially for gravity loads and a nominal load of 8% of its weight but have been tested in several configurations (bare frame, infilled frame before and after reparation and strengthening). Its reinforcement details were specified to be representative of buildings constructed over 4. years ago in European Mediterranean countries such as Italy, Portugal and Greece [13], [14].

The present paper concerns only the bare frame structure without strengthening. This structure has been tested with 2 signals representative of 475 years and 975 years return periods. The second tests have been interrupted in order to limit the damage in the structure and be able to strengthen it. The behavior of the bare frame structure has been mainly dominated by the stiffer column which changes of geometry and reinforcement between the 2nd and the 3rd storey. During the 975 years return period test, damage and interstorey drift concentrated in the 3rd storey (maximum value: 2.41%). Damage was limited to some slight crushing in the stiff column.

NUMERICAL MODEL OF THE STRUCTURE

The non linearities have been assumed concentrated in localized plastic hinges at the extremities of the columns and beams modeled with the non linear fiber type beam element. The length of plastic hinge has been taken equal to the column width for this study. Elastic Bernoulli beam elements with linear curvature have been used for the central part of the columns and beams. A 20000 MPa Young modulus has been considered for the elastic behaviour. Effective width of the slab has been taken equal to 2.00 m, 0.85 m and 0.25 m in the 3 different models used for this study. This assumption can strongly changes the elastic stiffness and ultimate strength of the beams and so the strength of each storey and the global failure mechanism.

For example, the 1st natural frequency depends strongly of the effective width: 1.59 Hz for the model with 2.00 m effective width vs 1.29 Hz for the model with no slab contribution.

A damping matrix proportional to the stiffness matrix has been considered with 2% damping on the 3rd natural frequency (about 7.0Hz). The damping has been reduced in the non linear elements by a factor equal to 10.

DYNAMIC ANALYSIS

Dynamic computations with the signals representative of the 475 years, 975 years and 3000 years return periods have been conducted. The main global results are given in Table 1. Whereas the displacement and drifts are quite well predicted for the 475 years return period signal, one may remark the strong underestimation of these values for the 975 years signal since the damage is not cumulated in the computations at the opposite of the tests. These difference can also be explained by the limitations of the numerical constitutive law since the global strength has also been overestimated.
The results of the dynamic computations have been analyzed by defining a local curvature demand • and a damage index D function of the ultimate and yielding curvatures:

$$\mu=\frac{\phi_{\max}}{\phi_{yielding}} \text{ and } D=\frac{\phi_{\max}-\phi_{yielding}}{\phi_{failure}-\phi_{yielding}}$$

The ultimate curvatures $\bullet_{failure}$ are given by a preliminary section analysis and correspond to the local failure criteria of 0.35% in compression (for concrete) and 5% in tension (for steel). For the model with no slab contribution, the local damage index was maximum at the 3rd storey for the criteria compression and equal to 0.168, 0.245 and 0.964 respectively for the 475, 975 and 3000 years return period signals.
The non linear computations give also the shear demand in all the structural members. Table 2 gives the shear forces and shear stresses in the stiffer column in the 1st and 3rd storeys. The shear demand has been compared to the shear strength computed with the Priestley's formula [6]. In this formulae, the shear strength is separated in the contribution of steel, of concrete and of the inclined axial force. In the table, the contribution of steel and concrete have been given together (the stirrups provide a uniform shear strength of 0.60MPa). The shear demand remains lower than the shear strength calculated for a low ductility demand. This is in accordance with the experimental results and the fact the structure sustains the 475 years return period earthquake without damage.

Table 2. Main numerical results – Bare frame (model with no slab contribution)

	475 years numerical	*475 years experimental*	*975 years numerical*	*975 years experimental* [(1)]	*3000 years Numerical*
Max. acceleration	2.18 m/s^2	2.18 m/s^2	2.88 m/s^2	2.88 m/s^2	4.27 m/s^2
Top displacement	71.5 mm	60.8 mm	77.8 mm[(2)]	116.7 mm	167.4 mm
Base shear	219 kN	209 kN	272 kN[(2)]	217 kN	304 kN
Drift at 4th storey	0.58%	0.46%	0.70%[(2)]	0.91%	1.47%
Drift at 3rd storey	0.76%	0.79%	0.91%[(2)]	2.41%	2.25%
Drift at 2nd storey	0.81%	0.73%	0.96%[(2)]	1.03%	1.76%
Drift at 1st storey	0.63%	0.44%	0.85%[(2)]	0.63%	1.86%

[(1)] The test was interrupted after 7.0s [(2)]Max. values are reached before 7.0s

Table 3. Shear demand in the column 3 (stiffer column)

	475 years	*975 years*	*3000 years*	*Shear strength (high ductility)*	*Shear strength (low ductility)*
3rd storey					
Shear forces	108.1 kN	118.1 kN	126.0 kN	(217.+53.7) kN	(100+53.7) kN
Shear stress	0.865 MPa	0.945 MPa	1.01 MPa	(1.74+0.43) MPa	(0.80+0.43) MPa
1st storey					
Shear forces	202.1 kN	184.9 kN	218.3 kN	(261+157) kN	(120+157) kN
Shear stress	1.35 MPa	1.23 MPa	1.46 MPa	(1.74+1.05) MPa	(0.80+1.05) MPa

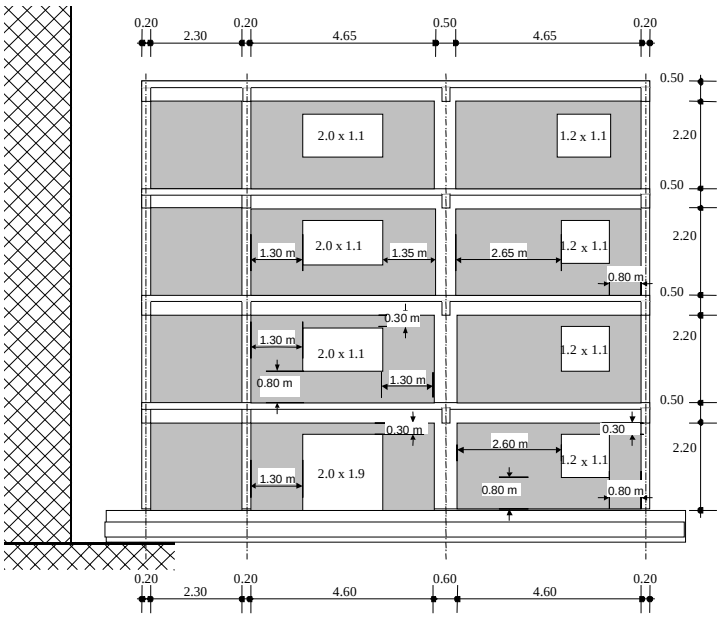


Fig. 7. Full scale 4 Storey Reinforced Concrete Frame tested at Ispra (configuration with masonry infills)

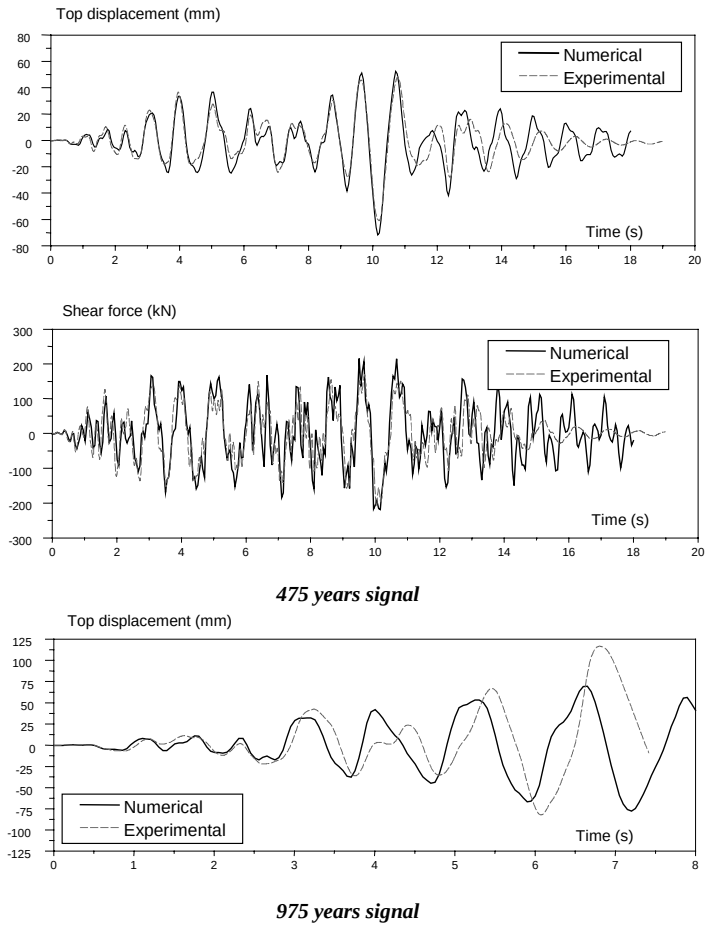


Fig. 8. Top displacement and base shear force - Model with no slab contribution

CONCLUSIONS

The present paper gives some general consideration about the application of non linear modeling to the seismic assessment of existing reinforced concrete buildings.
Push over and time history analysis have been conducted with a non linear fibre type beam element and have shown the capability of such modeling to reproduce the complex physical phenomena which can occur during a seismic loading. These non linear models allow to estimate the local flexural ductility demand and check the shear demand in the main structural members.
The good agreement between numerical and experimental results for the 2 different structures studied in the present paper must not make forbidden the difficulties in the application of such modeling to the seismic assessment of existing structure. One of the main difficulties is the definition of the damage index: what can be defined as failure ?, for which level of damage are designed the structures such as the nuclear facilities ? Furthermore the lack of common rules for the seismic assessment of nuclear facilities make

problematic the choice of some modeling parameters such as the damping matrix, the strength of the brittle failure mechanisms, the influence of the construction details on the modeling parameters, etc… Documents similar to the FEMA 273 Guidelines might be developed specifically for the nuclear facilities.
Finally the approaches applied to the study of the 2 structures rermains deterministic. A next step is to determine the fragility curve of the structure and compute a failure probability by convoluting fragility and seismic hazard. Although this approach is very attractive in term of safety analysis, one must not forget the problem carried out by the new parameters to introduce: enough experimental results are they available to determine the density of probability of local failure ?

ACKOWLEDGEMENT

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REFERENCES

1. Millard A., CASTEM 2000, Manuel d'Utilisation, CEA-LAMS Report N° 93/007, Saclay, 1993
2. Combescure D., Pegon P., Application of the local-to-global approach to the study of infilled frame structures under seismic loading, 12th World Conference on Earthquake Engineering, Auckland, New Zealand, 2000.
3. Combescure D., Sollogoub P., Bouchon M., Deleruelle F.. Local and global modelling of the NUPEC R/C Shear wall under seismic loading. 11th Earthquake Engineering European Conference, Paris 1998.
4. Pegon P., A Timoshenko simple beam element in Castem 2000, JRC Technical Note N°I.93.05, Ispra, 1993.
5. Guedes J., Seismic behaviour of reinforced concrete bridges, Numerical analysis and experimental assessment, PhD thesis, Univ. of Porto, 1997.
6. Priestley, M.J., Displacement based seismic assessment of reinforced concrete buildings, Journal of Earthquake Engineering, Vol 1, N°1, 1997.
7. Monti, G. Consistent insertion of bond-slip into beam fiber elements for biaxial bending. 12th World Conference on Earthquake Engineering, Auckland, 2000.
8. Xiao Y. & Ma R. Seismic retrofit of RC circular columns using prefabricated composite jacketing. Journal of Structural Engineering, Vol. 123, N°10, ASCE, 1997.
9. Eligehausen R. & Balazs G.L.. Bond and detailing. In Bulletin d'Information CEB N°217, Comité Euro-International du Béton, 1993.
10. Aboutaha, R. Seismic retrofit of non-ductile reinforced concrete columns using rectangular steel jackets. PhD Thesi dissertation, Austin University. 1996.
11. Sollogoub P., Combescure D., Queval JC, Chaudat Th. In plane behaviour of several 1/3rd scaled RC bearing walls. Testing and interpretation using non linear modelling. 12th World Conference on Earthquake Engineering, Auckland, 2000.
12. Griffith M.C., Pinto A., Seismic retrofit of reinforced concrete buildings. A review and case study, 12th World Conference on Earthquake Engineering, Auckland, New Zealand, 2000.
13. Calvi G.M, Recla M, Assessment of current prediction capacities of the response of existing reinforced concrete buildings, , 12th World Conference on Earthquake Engineering, Auckland, New Zealand, 2000.
14. FEMA, FEMA 273-NEHRP Guidelines for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, 1997.

Damage Detection and Assessment by Ambient Vibration Monitoring

Helmut WENZEL
Dr.- Dipl.Ing.

VCE

Vienna Consulting Engineers
Vienna, Austria

Helmut Wenzel is the President of **VCE**, an international consulting firm. He earned a Ph.D. in Bridge Construction from the University of Vienna in 1978. He operates world wise with offices in Vienna, Taiwan and Korea. Dr. Wenzel also teaches Bridge Design and Construction at the University of Vienna.

e-mail : vce@atnet.at

SUMMARY

Ageing is a fact of life. This applies for structures as well. Re-evaluation of structures therefore has become standard procedure and offers the chance to look at situations in the light of the newest technologies. Recent outstanding development in the field of traffic infrastructure can be applied to the nuclear facilities in order to improve existing methodologies. This contribution elaborates the options bridge monitoring methods offer for the assessment and seismic re-evaluation of nuclear facilities. The BRIMOS approach is based on the fact that the condition of any structure is represented in it's dynamic behaviour. The so called "Vibrational Signature" contains all information necessary for a detailed assessment and evaluation. The tools already in use comprise :

- Parametric excitation, where spectra from external sources, such as earthquakes, are compared to the structure's response
- Frequency analysis, where the response of the structure is monitored over a period of time and trends can be extracted
- Damping evaluation, where information content of the Vibrational Signature is extracted to allocated weak points and damages in the structure.
- Eigenform analysis, which allows a prediction of the behaviour of the structure under extreme loading conditions.
- Furthermore loading history, life expectancy, fatigue condition, influence of environmental loads and operation conditions are assessed.

1. GENERAL

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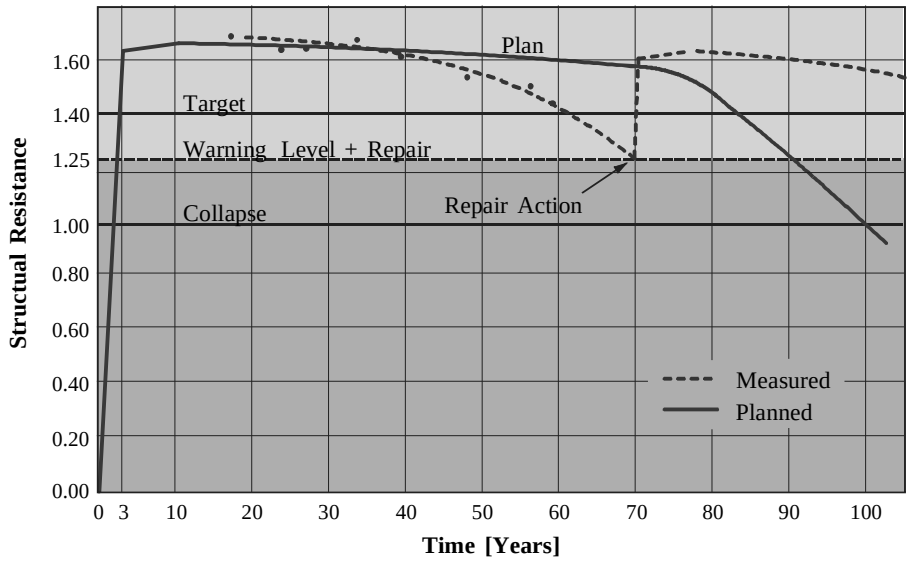


Fig. 1 Development of Resistance over time

Monitoring the quality of structures comprises a wide field of engineering tasks. The most promising recent development has been achieved with Ambient Vibration Testing and dynamic System Identification tools. Therefore this contribution concentrates on this subject.

2. BRIEF HISTORY

Mechanical engineers of the last century already predicted that the vibrational signature of structures and components contains all relevant information for assessment. Tests with simple structures, such as masts, date back to the early twenties. After WW2 the development was guided by the limited means available for

testing and calculations. All kind of strain gages were developed and other sensors tested. The seventies brought the first applications of field accelerometers with computer aided data processing. The Forced Vibration Testing (FVT), as applied by EMPA [4] in Switzerland or ARSENAL in Austria, gave remarkable results at small and medium structures. The breakthrough came with the development of powerful PC's and sensors in the nineties. This step was documented by the contributions to the 1995 IABSE Symposium in San Francisco [2,5,8]. Ambient Vibration Testing (AVT) became feasible and powerful. It becomes more and more accepted by practicing engineers and attracts research and development.

3. INTRODUCTION TO AMBIENT VIBRATION TESTING

Ambient Vibration Testing does not require a controlled excitation of the structure. The structures response to ambient excitation is recorded in a large number of points. By the application of system identification technologies the frequency response functions are determined and analyzed [6]. For large and flexible structures, such as suspension bridges, cable stayed bridges or high-rise buildings it becomes too difficult and costly to provided controlled excitation at levels which are significantly higher than the excitation provided by ambient sources. The method only requires the measurement of the response to ambient excitation which might be caused by wind, traffic, waves or micro seismic activity. It is assumed that the excitation is relatively smoothly distributed in the frequency band of interest. Than the natural frequencies and mode shapes of the structure can be identified and it becomes possible to estimate damping values associated with individual modes [7]. The main advantage of this method is that normal operation, such as traffic, does not have do be influenced or shut down during testing. Traffic is a welcome source of excitation which usually provides good wide band excitation but also deserves attention on tricky side effects as demonstrated in coming chapters. A typical layout for a monitoring system is shown in Fig. 2.

Several dynamic investigations of large complex civil structures, especially bridges, have been carried out by various research institutes. Research and development activities are recorded in parallel in Europe, America and Japan. A couple of institutions in the U.S.A i.e. the Columbia University and the University of Connecticut are developing prediction methods and are working with damage laws [2,5].

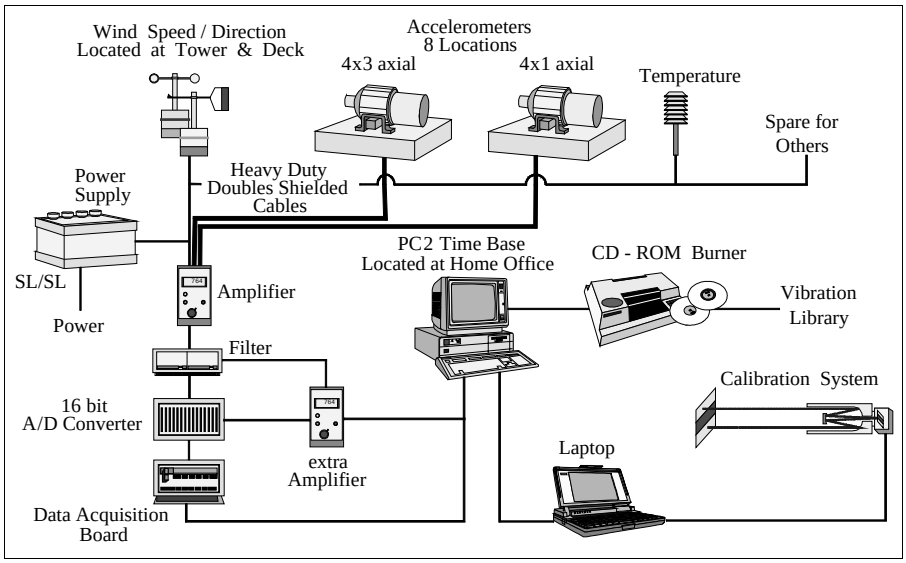
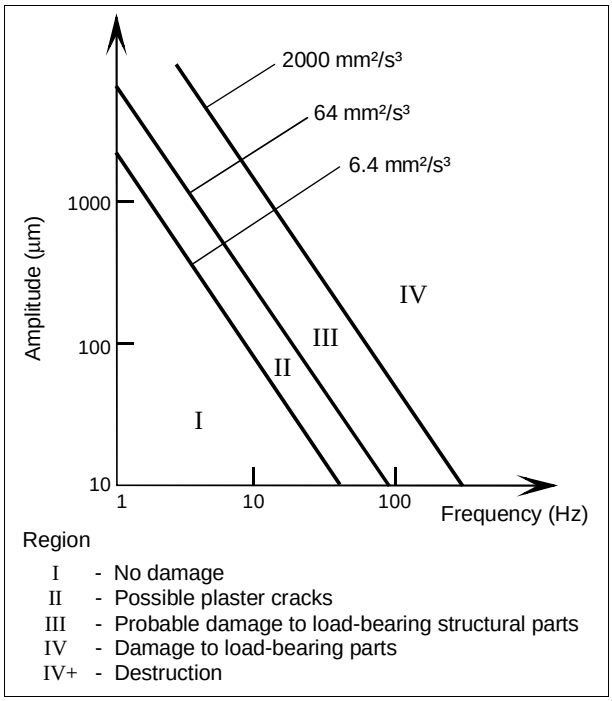


Fig. 2 Mobile system for ambient vibration testing

In Japan and the Far East large monitoring systems are very common and the Sensor technology is advanced [8]. But little has been published on the progress in system identification technologies. In Europe mobile systems have been applied successfully, so that a great number of structures were tested and the request for better system identification tools became imminent [11].



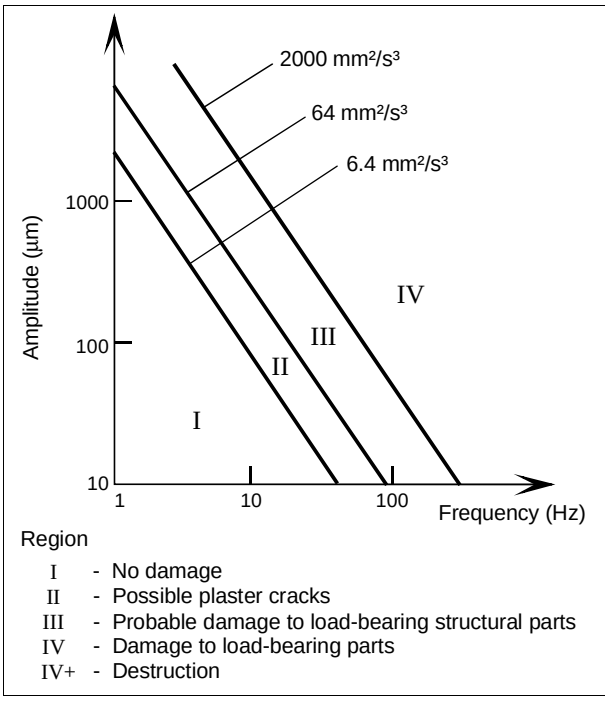


Fig. 3 Structural damage limits [3] acc. Beards "Structural Vibration"

The procedure of ambient vibration testing is straight forward. A theoretical model for the structure to be tested is created and the basic frequencies and mode shapes are determined as desired by the planer. Based on the geometry a layout of measurement points is decided and the measurement is taken under normal operation conditions [12].

Already on site the spectra are determined and compared with the theoretical values. The deeper analysis is than carried out in the home office. The result can be a comparison of desired behavior and actual behavior but also a change of behavior within a time frame, which means development of the structure over time (Fig. 1). The information content of the records is almost unlimited and in most cases the costs determine the depth of investigation.

A classification of results may be done using Fig. 3.

4. SYSTEM IDENTIFICATION TECHNOLOGIES

System identification means extracting the dynamic characteristic of bridges or other civil engineering structures from vibration data. The vibrational characteristic serves as input to model calibration and damage identification algorithms. Technical development work is carried out all over the world on this subject. For reference the BRITE-EURAM Project SIMCES (System Identification Methods for Civil Engineering Structures) is referred too. Special attention was paid to techniques making use of operational data.

4.1 Peak-picking

A first method to estimate the modal parameters of a bridge based on output-only measurements (accelerations) is the rather simple, but very effective, peak-picking method. The method is widely used and practically implemented by VCE [11], EMPA [7], BAM and others.

In this method the natural frequencies are determined as the peaks of the Averaged Normalized Power-Spectral Densities (ANPSDs). The ANPSDs are basically obtained by converting the measured accelerations to the frequency domain by a Discrete Fourier Transform (DFT). The coherence function computed for two simultaneously recorded output signals has values close to one at the natural frequencies. This fact also helps to decide which frequencies can be considered as natural.

The components of the mode shapes are determined by the values of the transfer functions at the natural frequencies. Note that in the context of ambient testing, transfer function does not mean the ratio of response over force, but rather the ratio of response measures by a roving Sensor over response measures by a reference sensor. So every transfer function yields a mode shape component relative to the reference sensor. Here it is assumed that the dynamic response at resonance is only determined by one mode. The validity of this assumption increases as the modes are better separated and as the damping is lower. The method has been used successfully at VCE and EMPA for a large number of structures.

4.2 Least Square Method

Is has been shown that, under the assumption that the system is excited by stationary white noise, correlation functions between the response signals can be expresses as a sum of decaying sinusoids. Each decaying sinusoid has a damped natural frequency and damping ratio that is identical to that of a corresponding structural mode. Consequently, the classical modal parameter estimation techniques using impulse response functions as input like Polyreference LSCE, Eigensystem Realization Algorithm (ERA) and Ibrahim Time Domain are also appropriate to extract the modal parameters from response only data measured under operational conditions. This technique is also referred to as NexT, standing for Natural Excitation Technique.

4.3 Stochastic subspace identification

Unlike the two previous methods the stochastic subspace identification method directly works with the recorded time signals. The peak-picking method requires frequency domain data while the polyreference LSCE method needs the correlation functions between time signals [10].

The method assumes that the dynamic behavior of a structure excited by white noise can be described by a stochastic state space model (this statement can be justified):

$$x_{k+1} = Ax_k + w_k \quad (1)$$

$$y_k = Cx_k + v_k \quad (2)$$

where x_k is the internal state vector; n_p is the number of poles; y_k is the measurement vector and w_k, v_k are white noise terms representing process noise and measurement noise together with the unknown inputs; A is the state matrix containing the dynamics of the system and C is the output matrix, translating the internal state of the system into observation.

The subspace method than identifies the state space matrices based on the measurements and by using robust numerical techniques such as QR-factorization, Singular Value Decomposition (SVD) and least squares. Loosely said, the QR results and a significant data reduction, whereas the SVD is used to reject the noise (assumes to be represented by the higher singular values). Once the mathematical description of the structure (the state space model) is found, it is straightforward to determine the modal parameters (by an eigenvalue decomposition); natural frequencies, damping ratios and mode shapes.

4.4 Mode Shapes

The real vibration shapes of a structure consist of the mode shapes corresponding to the natural frequencies. Therefore the mode shapes are - beside the natural frequencies - the second important quantities describing the dynamic behavior of a structure. Measurements of the global vibrations in discrete points contain the contributions of the single mode shapes to the global dynamic behavior at these locations. After identifying the natural frequencies in the ANPSD the acceleration records are transformed into displacement records by a double integration process. Transformation of these time domain displacement records into frequency domain and normalisation of the displacement
spectra leads to the displacement values for each natural frequency at each measurement location. The measured mode shapes are compared to the computes ones using MAC techniques (Modal Assurance Criteria).

4.5 Damping

Beside the natural frequencies and the corresponding mode shapes the damping coefficients are the
third factor used for describing the dynamic response of a structure. The frequency dependent damping ratios are important criteria for structural assessment due to the fact that these ratios increase significantly when the structural resistance decreases – in other words, high damping ratios are an indicator for reduced safety. Especially prestressed concrete bridges show a distinct increase of the damping ratios when the cross sections change from uncracked to cracked state (i.e. due to loss of prestressing force). The damping ratios are extracted from the measured acceleration records by using the Random Decrement Technique (RDT). This technique was developed by NASA in the early 70-ies. It is based on the idea that averaging segments of time series of the response of a stochastic loaded system describe the system properties cleaned from the traces of the stochastic load [12].

4.6 Assessment of the Structure

When dealing with wind the so called "aerodynamic derivatives" are used for assessment. It is proposed to introduced so called **“structural dynamic derivatives”**, which are based on a similar idea. The most desired system parameters are the real stiffness and the damping of the structure. The stiffness varies with time and is influenced by creaking of concrete structures. Material damping represents the actual condition of the material with respect of fatigue and lifetime development, and system damping represents the capacity of the structure to dissipate energy (i.e. friction in bearings etc). When the actual stiffness of a structure, obtained from measurement, is known, it is a simple task to introduce it into the calculation and rerun the structure. This gives a distinct value for the remaining structural capacity. It can be expressed in a percentage.

5. QUALITY ASSESSMENT AND DAMAGE DETECTION

The use of the described tools and the key-points under discussion are most usefully described in examples.

5.1 Assessment of Cables

Cable stays are exited by moderate wind and rain. Out of many cables only a few are concerned. Monitoring is able to identify those cables where the critical damping is below 0.3%. Adequate damping measures can be designed with the data and the effectiveness can be tested after installation. The method was applied at a couple of cable stayed bridges already. One of the most regent cases was the new Donaustadt - Bridge in Vienna which spans the Danube with an eccentric main span of 186 meters. 8 out of 20 cables where effected by the phenomenon and the problem was solved by installation of tiny tie ropes between adjacent cables.

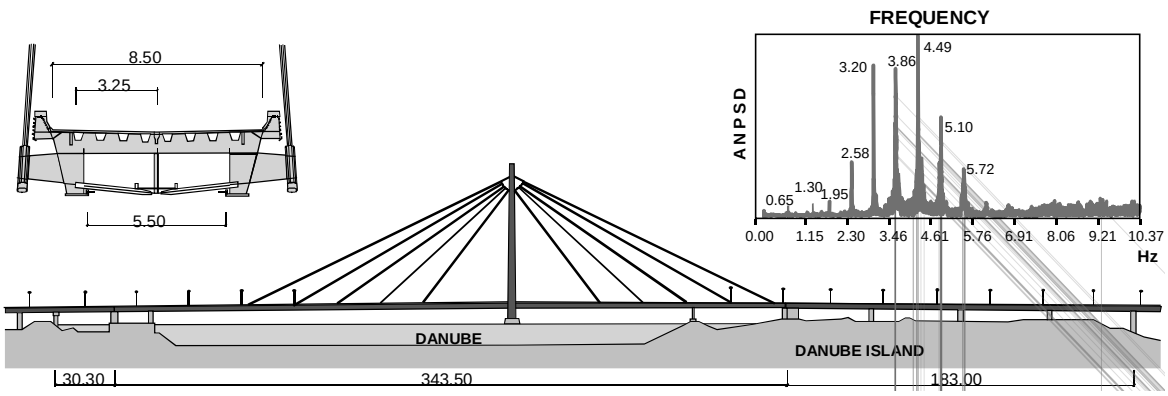


Fig. 4 Elevation, cross section and spectrum of cable 10 N

5.2 Vibration Mitigation

Two reduce noise and particular vibration emissions of railway lines mass spring systems have been developed during the past year. The vibration attenuation capacity of such systems depends on the natural frequency and the damping ratio which are controlled by the mass and the spring stiffness. The effectiveness of such systems can be tested by the described system identification technologies. Series of tests are carried out to conform the design integrity of the system. The thermal behavior of the huge concrete mass, a couple of 100 meters of concrete mass is pureed in one piece, the actual displacement under train loads and the vibrational behavior is monitored. The finite element calculation can be calibrated by the results and the transfer functions from the sources of vibration to the target structures is determined. The quality of a design idea is assessed as well as the function in reality is documented. As and example the 1,176 m long continuos mass spring system of the Zammer Tunnel for the Austrian High Speed Railways is presented (Fig. 5). It was found that the targets have been over fulfilled which provided the basis for more economic design in the 2. case where the technology has been applied.

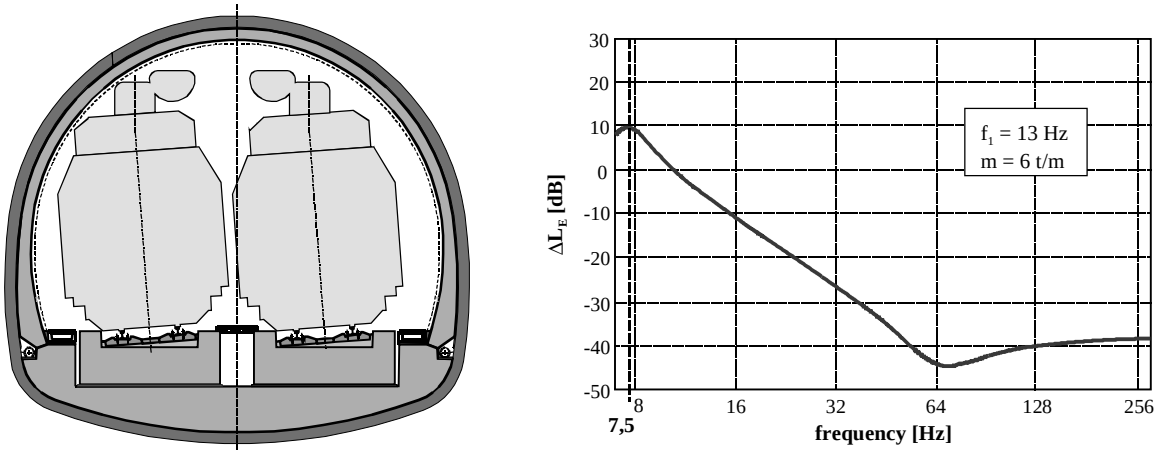


Fig. 5 Mass-spring-system of Zammer tunnel and frequency attenuation

5.3 Damage Identification

Construction joints of concrete bridges built in the 1960'ies represent notorious week points. The Großram – Bridge is a typical continuos structure built by the advancing shoring method. During earlier inspections a defect construction joint was identified. The joint opened under heavy loads exposing the prestressing tendons to moisture and salt attack. The joint was repaired and strengthened by clued fibre plates. The task for the monitoring team was to assess the quality of the strengthening work and the function of this important structure. It was demonstrated that the damage was repaired successfully and the capacity of the structure was restored. The assessment was carried out using Modal Assurance Criteria (MAC) which provide figures for modal fitting between calculations and test data. Fig. 6 shows the superimposed modes in theory and practice.

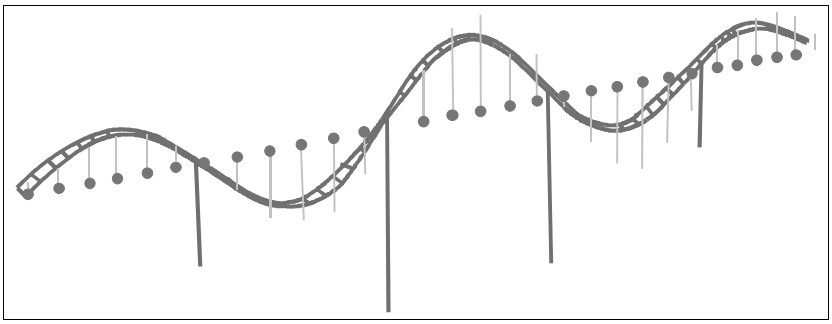


Fig. 6 Comparison between calculated and measured data

5.4 Pier Settlement

System identification is a most valuable tool for the determination of soil structure interaction phenomena. The most common simple assessment by the engineer, that piers are rigidly connected to the ground can be quantified by measurement carried out under operation.

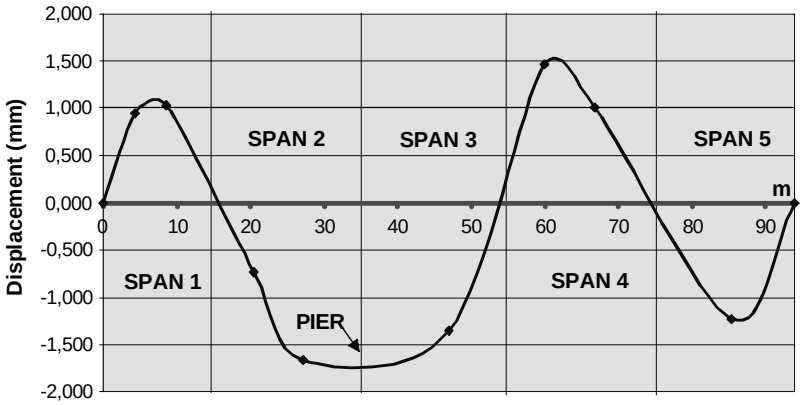


Fig. 7 Displacement of the structure under heavy traffic

The best indicator is a change in mode shape as shown in Fig. 7 for the Gurk – Bridge where a pier settles periodically with heavy loading. This settlement with a period of 20 sec. and a displacement of 8 mm indicates a beginning damage of the foundation. This findings led to immediate action by additional supports to the structure to avoid a collapse. The displacement should be very close to zero of piers in continuous structures. In Fig. 7 the settlement of the pier is obvious.

5.5 Power Plant Buildings

The dimensions of the load bearing columns and members of power plant buildings are depending on the assessment of realistic loading from wind-forces, life-load and accidental loads. When an existing system is measured and the results are compared to the design, experience can be gained for the design of future projects. Another benefit in buildings is the determination of real displacement, which has influence on the design of the cover and other structural elements. Currently under development is a method to assess precast panels of major structures on instability or their potential of failure. The comparison of the vibrational characteristic of neighboring similar panels provides an easy and quick option for assessment.

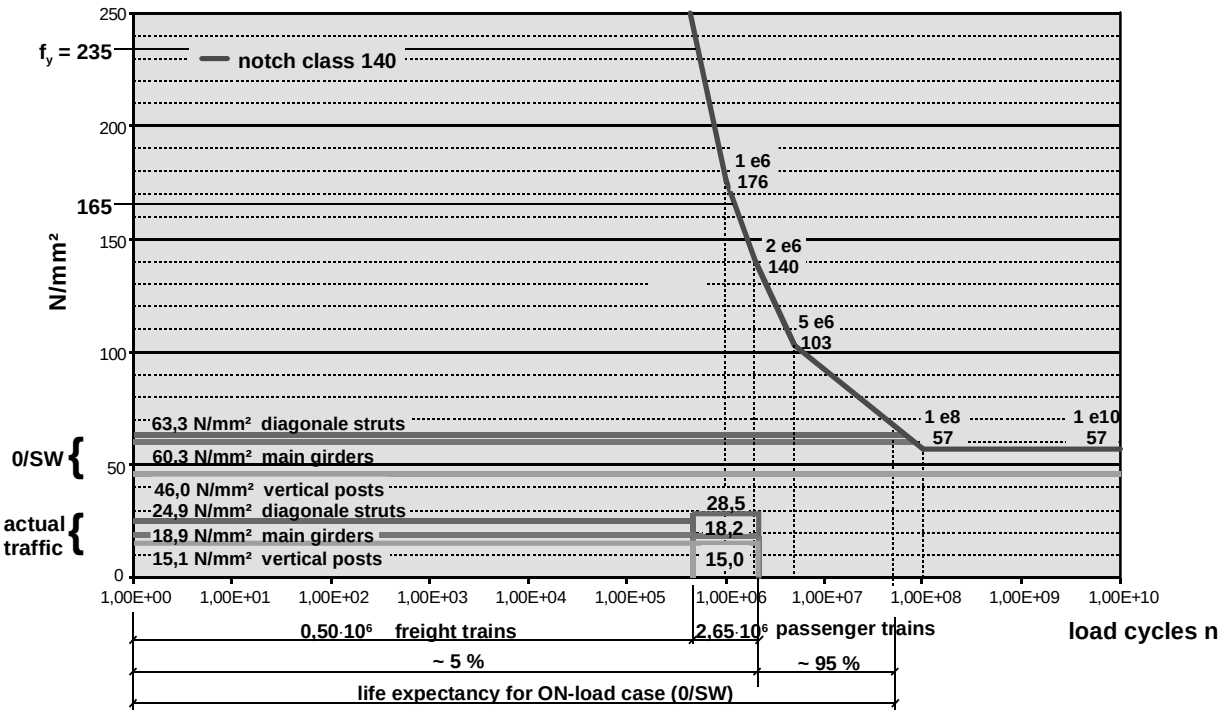


Fig. 8 ÖBB-Bridge Tulln – max. fatigue stress for superstructure 1 and life expectancy

5.6 Assessment of old Steel Structures

There are huge numbers of bridges older than 100 years in our railway systems. The desire for higher axial loads and higher speed of trains requires an assessment of these structures. The fact, that these bridges very often consist of a number of equal single span girders allows a typical qualitative comparison between each other. While monitoring the 5 span railway bridge across the Danube at Tulln differences in the vibrational characteristic could be identified as repair actions after damages during the 2. world-war. The correct acting structural system including the effect of the repair-works could be established and the finite element model now represents the real behavior of the structure.

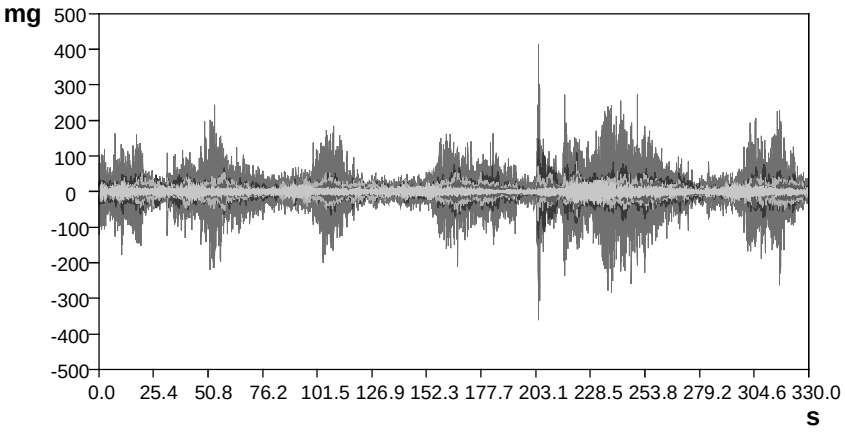


Fig. 9 Peak acceleration of 0,42 g for the cantilever tip of the Europabrücke

In addition the assessment of the remaining life-time of the structure was carried out with the use of the load collective determined from the railway-data and the comparison to typical Wöhler Diagrams. In Fig. 8 a typical life expectancy for a 100 year old railway bridge is shown. It has to be noted that a perfect assessment of the remaining life-time is impossible due to the high scatter of the fatigue tests. Anyhow a secure remaining life-time can be determined.

5.7 Assessment of Structural Elements

Structural elements of steel bridges may experience dramatic changes in loading when bridges are retrofitted or strengthened. System identification also serves for the determination of the behavior of single members within a complex structure (Fig. 9). In case of the record breaking Europa - Bridge an additional lane was introduced and the structure had to be widened by 3 m. This creates unbalanced systems for the orthotropic deck which has considerably larger cantilever arms now. The behavior of the cross-section had to be determined to find out the real loads on diagonal beams.

5.8 Structural Control by Tele-Monitoring

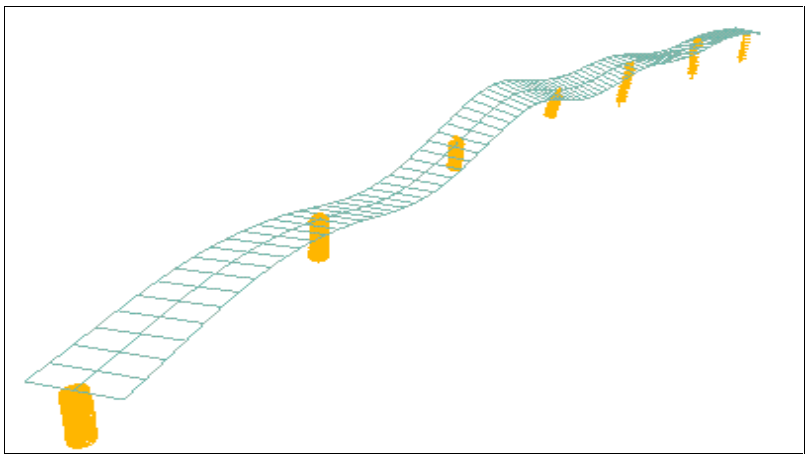


Fig. 10 Animated first mode of the St. Marxer Hochstraße

Permanent monitoring of structures enables the engineer to gather data on the long-term performance of bridges. When the system is clearly identified, each new recorded response can be classified and counted. After a reasonable amount of time sufficient information for a realistic fatigue calculation are collected. A side effect is the identification of extraordinary loads on the structure.
For this purpose a bridge in Vienna was instrumented with a Tele-monitoring system, consisting of accelerometers linked with a Video-system, that provides information on the traffic conditions at certain structural response levels. The well-known thesis, that high-loads at low speed are less harmful to the structure than standard loads at excessive speed, proved to be valid. Furthermore this type of instrumentation provides the chance to get actual traffic information via Internet.

5.9 The Effect of Temperature and Traffic Loads

It was found that the temperature effect might be dramatic in case of very rigid structures with small spans. In case of major bridges this effect can be neglected in the assessment of the vibrational characteristic. At the Schottwien – Bridge with a main span of 250 m and 12 m high girders at the support measurements were taken over a temperature range of 17°. Due to the fact that this bridge is virtually unloaded every night very clear ambient signals have been received. The change of the dominant first Eigenfrequency is within 1% over the full temperature range and can therefore be neglected. This phenomenon has been confirmed on other major structures. In minor structures the strain from temperature changes the characteristic within a range of 5%, which can not be neglected anymore. The results have to be calibrated by statistical means. Therefore it is essential to record the ambient temperature at each monitoring action. The effect of traffic loading on the bridge was tested at the Nordbrücke where 105,000 vehicles pass daily with distinct peaks at rush hour. It was demonstrated that the method can deal perfectly with different load scenarios.

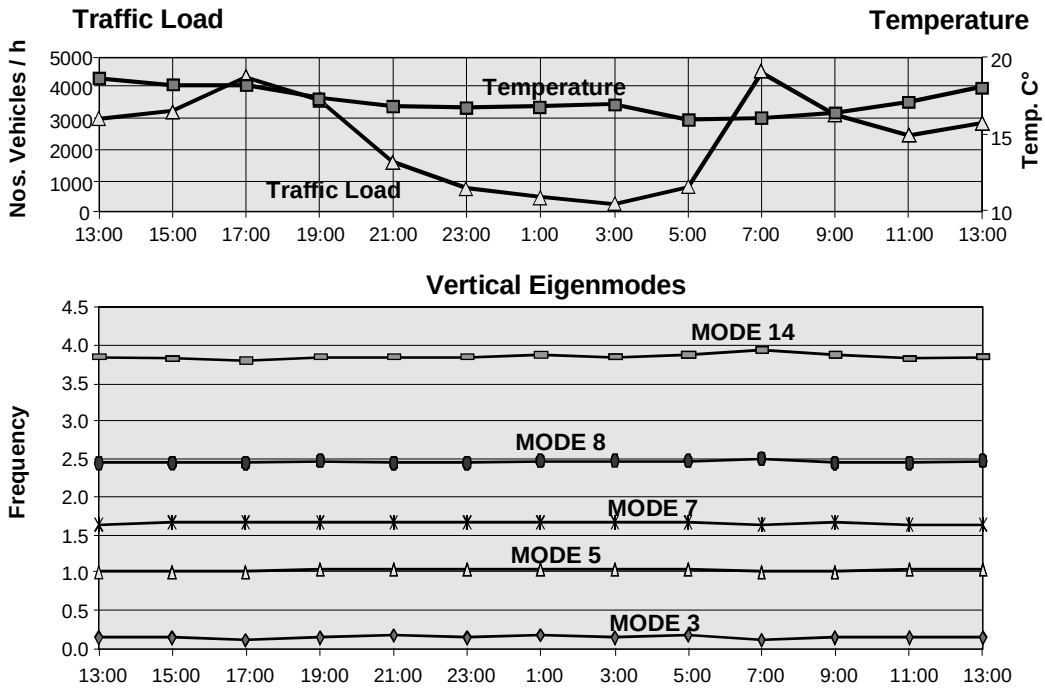


Fig. 11 Effect of vehicle load on the vertical modes (24 hours)

5.10 Quality Control of Construction

The vibrational characteristic changes with each construction stage. Monitoring instruments are able to record these changes and therefore confirm the quality of construction steps carried out. Another benefit is that major impacts are recorded which might influence the quality of the structure. In case of cable stayed bridges the stresses in the cables can be monitored and compared to the desired values. Another application is the check of the removal of temporary fixations during construction. Complex systems can so be checked easily as demonstrated in Fig. 12.

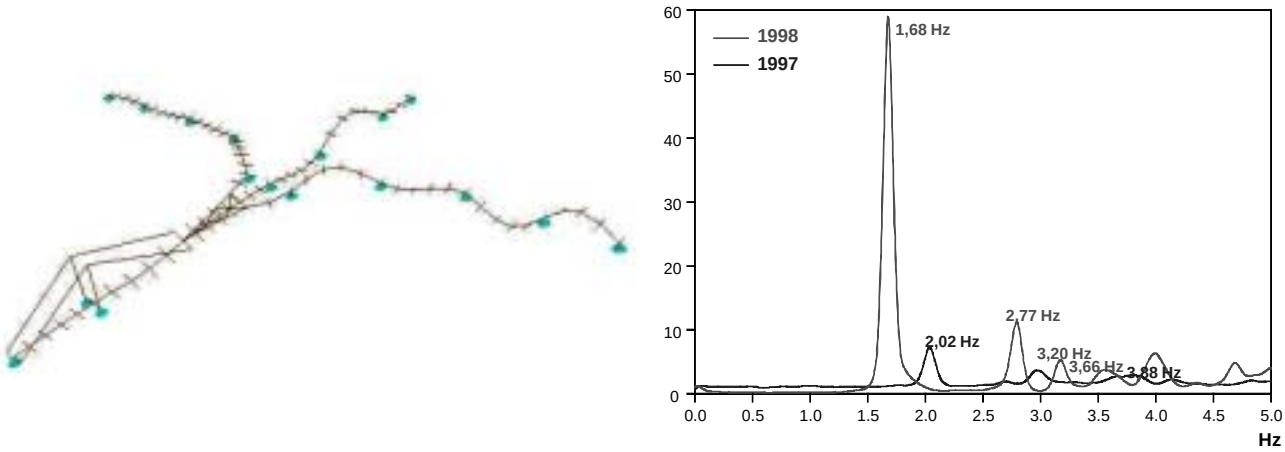


Fig. 12 Hall West Bridge, effect of the release of a horizontal restraint

5.11 Assessment of Prestressing Cables

Due to the growing importance of external prestressing assessment tools are desired for the existing structures. With the application of similar algorithms as used for stay cables the external prestressing cables can be assessed on the actually existing forces and on the global behavior of the structure, particular after retrofit measures. As an example the Mur-Bridge St. Michael is shown, where excessive displacements have been stopped by external cables. The cables have been assessed one by one by acceleration measurement and the safety of the global system was determined.

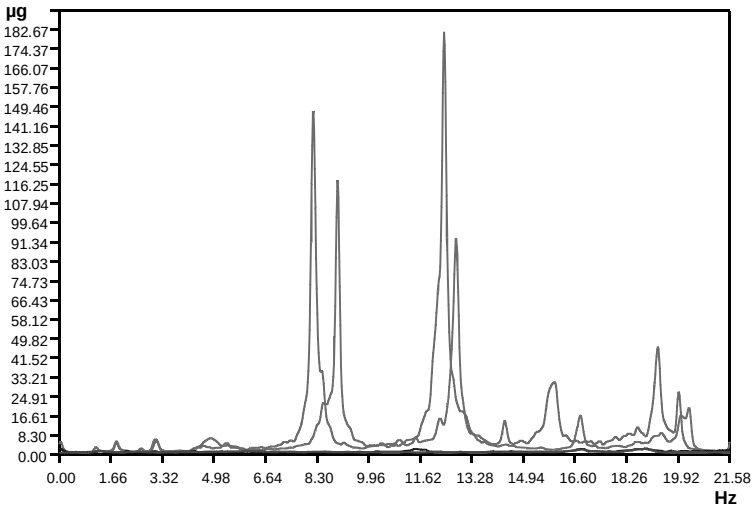


Fig. 13 Simple measurement of spectrum and related cable frequencies

6. OUTLOOK

Structural monitoring is in the transition phase from being scientific and educational only to commercial exploitation. The research and development work is still too much concentrated on basic research. In the 5th Framework program of the European Union monitoring plays a key role with the key-words on structural assessment and damage detection. Due to the end-user orientation these projects will come up with practical solutions for the engineering society. It is planed to form a cluster of projects consisting of:

- A bridge monitoring project including damage detection
- A high-rise building monitoring project including wind design optimization
- A cultural heritage monitoring project including detail assessment of surfaces

- A demonstration and data collection project providing free access to data
- A basic research project on damping and related issues

Two main facts are hindering the dissemination of the methods. First of all good equipment is rather expensive and suffers from the rough construction environment. The application requires expensive specialists which increases the costs of the action. The second problem is the absence of monitoring in the education of structural engineers and the related low level of information in the clients organizations. The enormous potential of monitoring is widely unknown.

Considering all this facts future key-actions shall concentrate on:

- The development of reliable, cable-less monitoring systems which can be purchased at reasonable costs
- The development of software-tools for application by concerned engineers without the necessity of expensive specialists
- The education of the engineering society with respect to monitoring and its potential

Considering the experience made with over 60 structures monitored, where 20% of the cases showed considerable differences between design and reality, monitoring should find its way to be a standard tool for structural engineers in future.

REFERENCES

[1] Agrati S., "Estimation of Structural Parameters from Ambient Vibration Test", *Master thesis 1994, Danish Technical University.*

[2] Aktan A.E., "Issues in Instrumented Bridge Health Monitoring", *IABSE Symposium San Francisco 1995.*

[3] Beards C.F., "Structural Vibration, Analysis and Damping", *Halsted Press, ISBN 0 470 23586 1*

[4] Cantieni R. et al., „Untersuchung des Schwingungsverhaltens großer Bauwerke", *Technischen Akademie Esslingen 1996.*

[5] Dewolf J.T., Coon P.E., O'Leary P.N., "Continuous Monitoring of Bridge Structures" *IABSE Symposium San Francisco 1995*

[6] Felber A. J, "An Introduction to Ambient Vibration Testing", *EMPA Report-No. 155'715*, Dübendorf, 1996.

[7] Felber A. J.; Cantieni R., "Introduction of a new Ambient Vibration Testing System – Description of the System and Seven Bridge Tests", *EMPA Report-No. 156'521,* Dübendorf, 1996.

[8] Liu X., "Global Monitoring System on Lantau Fixed Crossing in Hong Kong", *IABSE Symposium San Francisco 1995*

[9] Peeters B., Abdel Wahab M., De Roeck G., De Visscher J., De Wilde W.P., Ndambi J.-M., Vantomme J., "Evaluation of structural damage by dynamic system identification", *In Proceedings of ISMA 21, the International Seminar on Modal Analysis*, pp. 1349-1361, Leuven, Belgium, 1996

[10] Peeters B., De Roeck G., Pollet T., Schueremans L., "Stochastic subspace techniques applied to parameter identification of civil engineering structures", *In Proceedings of New Advances in Modal Synthesis of Large Structures: Nonlinear, Damped and Nondeterministic Cases*, pp. 151-162, Lyon, France, 1995

[11] Wenzel H.; Pichler D., "Structural Assessment of Railway Bridges by Ambient Vibration Testing", *US-Canada-Europe Workshop on Recent Advantages in Bridge Engineering*, Dübendorf and Zürich, 1997.

[12] Wenzel H., Pichler D., Schedler R., "Ambiente Schwingungsmessungen zur System- und Schadenserkennung an Tragwerken", *Bauingenieur März 1999*

Seismic Periodic Safety Review of Nuclear Power Plants in Korea

Lee, Jong-Rim[1], Ham, Young-Seoung[2], Lee, Sang-Kook[3]

1) Korea Electric Power Research Institute, KEPCO, Taejon, Korea
2) Korea Electric Power Corporation, Seoul, Korea
3) Korea Institute of Nuclear Safety, Taejon, Korea

Abstract

A periodic safety review (PSR) for Kori NPP is currently in process in Korea. Based on the research activities related to seismic re-evaluation for the existing NPPs, several technical issues are briefly introduced with countermeasures and improvements in this paper. Geological and seismological investigations, which include evaluation of capable faults and estimation of maximum credible earthquake for the NPP sites, were carried out in this study. Based on the recorded earthquake an attempt was made to develop a site-specific response spectrum. Earthquake monitoring network is installed on the outskirts of NPP sites to identify seismic activity. For reducing uncertainties of the probabilistic seismic hazard assessment (PSHA), a research work was performed. Seismic qualification of equipments for the unresolved safety issue (USI) A-46 NPPs is introduced. The post-earthquake procedure including selection of base lines and a seismic damage indicator is developed to provide guideline for better post-shutdown inspection and test. A study to evaluate tunamic risk is introduced. In the future, further implementation to resolve seismic safety issues is to be made.

1. Introduction

The Kobe earthquake in Japan not only disclosed the fact that even a low seismicity area could bring forth a large magnitude of earthquake but also made us realize the importance of aseismic countermeasures and seismic design criteria. Even though little earthquake damage has occurred throughout the 20th century in Korea, historical record reveals that earthquakes incurred significant damage in the Korean history over 2000 years. In December 1996, the earthquake with magnitude 4.5 stroke the Yongwol area located in the middle eastern part of Korea, spreading impact on the whole Korean peninsula. In June 1997, the Kyungju earthquake with magnitude of 4.3 concerned many people because its epicenter is located on the strike of the Yangsan fault, which is regarded as a potential active fault by several seismologists. Kori and Wolsung NPPs are located approximately 20 km far away from the Yangsan fault. In these regards, seismic design requirements have been strengthened to all the structures in Korea since 1999.

Over 40% of total electricity generation in Korea depends on 16 units of NPP in operation. Since the first commercial operation of NPP in 1978, there have been substantial developments in safety standards and practices, and, in technology, better analytical methods and lessons learned from operating experience in Korea.

The objective of a PSR is to determine by means of a comprehensive assessment of an operational NPP, whether the plant is safe as judged by current safety standards and practices, and whether adequate arrangements are in place to maintain plant safety. A PSR for old NPPs in Korea is in process since 2000. It is expected that the results of PSR for operating NPPs be of great use not only for feedback of the design improvement for future NPPs but also for evaluation of the old NPPs for life extension. In this paper, the present status of a PSR for NPPs is described with countermeasures and improvements. The PSR for Korean NPPs in this paper includes 1) evaluation of capable fault and maximum credible earthquake, 2) evaluation of design response spectrum 3) improvement of Soil-Structure Interaction (SSI) analysis technologies, 4) seismic qualification for equipments, 5) improvement of uncertainty in PSHA, 6) NPP response to an earthquake, and 7) evaluation of tunami risk.

2. Evaluation of Capable Fault and Maximum Credible Earthquake

The Yangsan fault trending NNE-SSW is one of the latest major faults developed in the Korean peninsula. It is about 200km long and located in the southeastern part of Korea. There have been arguments over whether the Yangsan fault is capable and can be a seismic source, thus jeopardizing the safety of NPPs. Most of the Quaternary faults are very steep and found in or near Quaternary alluvial deposits (Okada, 1999). Historical earthquake records are concentrated around Kyongju City in the central part of Yangsan fault. Kyongju was an ancient capital city about 1,500 years ago and populated area at that time (Lee, 1991). Several geologists insist that the Yangsan fault be active based on the concentration of historical earthquakes near Kyungju but they failed to provide clear evidences.

The Ulsan fault trending NNW-SSE strike is about 50km long and located next to the Yangsan fault. According to previous studies the Ulsan fault was considered as not a fault but only a long lineament because of no displacement found along it. Recently, several quaternary faults were reported in the northeastern part of the lineament. Some experts pointed out the activity of the Ulsan fault with reference to the seismic safety of the Wolsung NPP located about 20km from the fault. Studies in recent years show reverse faults in Quaternary deposit at several sites located in the eastern block of the Ulsan fault. And the results also reported two fault movements and liquefaction phenomena resulting from MMI IX earthquake (Kyung, 1997). At present there are some reports on Quaternary faults near the Ulsan fault but there is no direct evidence supporting that the Ulsan fault is a seismic fault.

These studies have only focused on age dating of Quaternary faults at specific outcrops without quantitative paleoseismic evaluation from the faults. Recently, a long-term research project has just been launched to interpret the segmentation of the fault, to analyze the development of the fault, to develop domestic active fault criteria, and to evaluate quantitative seismic potential for the NPP sites.

For the future NPP sites to be located next to the existing Kori and Wolsung NPPs, comprehensive investigations are in progress and research projects are scheduled to determine the activity of faults, site suitability, and seismic safety. Basically, lineaments will be re-evaluated through aerial photographs, satellite image interpretation and trench survey across the lineaments faults will be performed.

The paleoseismologic data from detailed observation of trench sections and age dating data for the Quaternary sediments, fault rocks and gouges will provide us information on the fault movement age and recurrence interval, slip rate, maximum displacements and so on. Various methods for age dating will be applied.

In the near future, it is expected that a capable fault criteria suitable for Korean geological environments will be recommended by Korean Institute of Nuclear Safety (KINS). Based on the new capable fault criteria, previously known Quaternary faults will be re-evaluated for the activity of faults and its effect on NPPs.

3. Evaluation of Design Response Spectrum

Due to the lack of the earthquake data, instead of using site-specific response spectra for the seismic design of NPP in Korea, the standard design ground response spectra of U.S. NRC reg. Guide 1.60 have been applied. A research work is carried out to evaluate the design response spectrum on the assumption that the geological and seismological characteristics of the Korean Peninsula are analogous to those of the stable continental regions.

Two methods, the empirical and the synthetic methods, are applied to develop the site-specific response spectra of NPP sites. For the empirical method, compatible earthquakes in the world should be selected because of the lack of domestic earthquake data. The criteria and procedures to select compatible earthquake are developed considering the source mechanism of earthquake, propagation property of seismic wave and amplification properties of the site. For the synthetic method, a synthesizing program is made and input data of seismic source are defined. With the selected and synthesized earthquake data, the

procedures of developing site-specific response spectrum are established. The compatible earthquakes detected in the stable continental regions are selected.

For the empirical method, firstly, the procedure and the method to evaluate the similarity of the earthquakes in the world are established. Secondly, the evaluating items are quantified. Lastly, several compatible earthquakes are selected and their response spectra are calculated. The response spectra developed show that the standard response spectra are conservative at low-frequency range and peak responses are indicated around 10Hz. It is known that the similar results have been discussed in the eastern USA (EPRI, 1995).

To make synthetic earthquake from the small earthquakes occurred in Korea, the method that synthesizes a large earthquake from a small earthquake with seismic source data was developed. To predict the large earthquake, the actually measured large and small earthquakes that occurred at the same source is necessary. Unfortunately, there are very few applicable earthquake events in domestic data. As a result of applying the synthetic method with one event, the response spectrum is similar to that of compatible earthquakes (See Fig. 2). It may be noted that the results of the two methods indicate a consistency.

For the practical usage of the method, a proper theory should be developed and actual earthquake data should be accumulated. Stochastic method (Boore, 1983) could be one of alternative methods to gain site-specific large synthetic earthquakes.

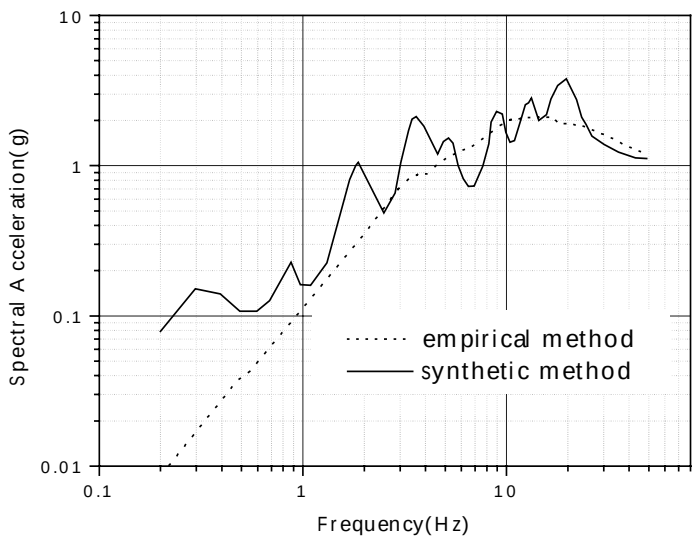


Fig. 2 Comparison of response spectra developed by empirical and synthetic methods

The earthquake parameters such as peak ground acceleration, spectral acceleration and duration time of ground motion should be obtained from the recorded data for engineering purpose. To measure strong earthquake, earthquake observatory networks have been established, which consist of eight earthquake observatory stations (excluding four stations established by KINS) in the vicinity of the four NPP sites (See Fig. 3). Individual stations are equipped with a seismic accelerometer, a velocimeter and a recorder. Recorded data from these stations are transmitted to the seismic monitoring center of Korea Electric Power Research Institute (KEPRI) and analyzed in real time basis. KEPRI plans to install five additional earthquake observatory stations near the Yangsan fault by 2002 and connect with Korea Integrated Seismic System (KISS).

This system will serve as a cornerstone significantly contributing to develop site-specific ground motion attenuation relation, design response spectrum, etc. as well as to provide useful data to evaluate

earthquake effects on the NPP. It is expected to identify the characteristics of seismic activities in the Korean peninsula.

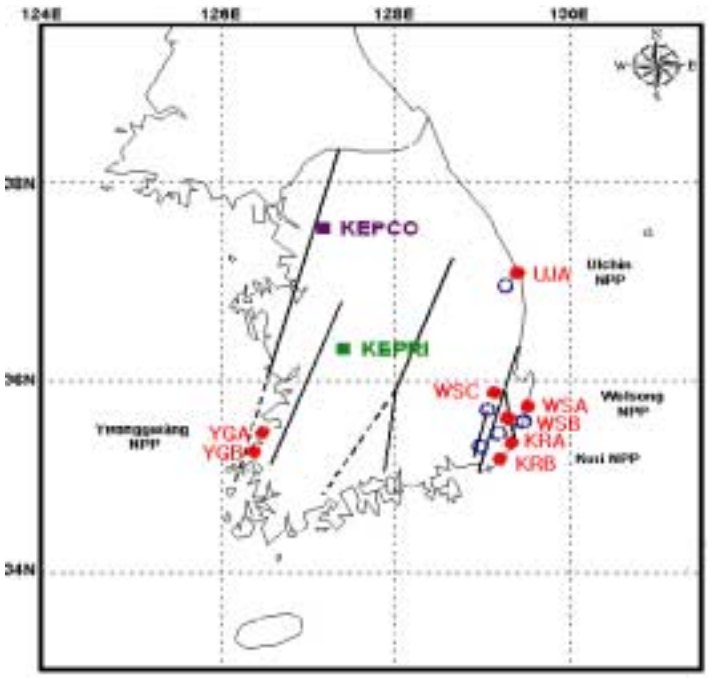


Fig. 3. Earthquake Observatory Network for NPPs sites

4. Improvement of Soil-Structure Interaction Analysis Technologies

It is difficult to accurately estimate the SSI effect because of many uncertainties of complicated soil properties. Moreover, available techniques and computer codes for SSI analysis may give quite different results depending on their different assumptions and limitations in spite of remarkable development of SSI analysis procedures and theories.

To solve these kinds of essential SSI problems, KEPRI has actively participated in the Hualien project under the leadership of EPRI and Taipower since 1990 (KEPRI, 1995). The Hualien project was initiated to obtain earthquake-induced SSI data, to identify the nonlinear soil behavior due to strong motion and near field earthquake characteristics, to verify the convolution and deconvolution methods considering soil stiffness reduction, and to define the input motion for SSI analysis. A quarter scale reinforced concrete containment model was constructed at Hualien in Taiwan where strong earthquake motions frequently occur. On the basis of various SSI analysis experiences and the obtained earthquake data, KEPRI has developed the advisory software program to guide SSI analysis and improved the probabilistic approach for SSI analysis to quantify the uncertainties of SSI input parameters (KEPRI, 2000).

To verify the developed SSI analysis program, the analytical results were compared with the actual data recorded in the Hualien large scaled seismic test (LSST) model. As shown in Fig. 4, maximum difference between analytical and recorded results is 9.4 % for the fundamental frequency and 16 % for the peak acceleration at the wall of Hualien LSST model (See Fig. 4). These results indicate that the developed SSI analysis program is reliable and accurate in comparison with the existing SSI analysis techniques.

In order to enhance the reliability of SSI analysis, KEPRI is carrying out a long-term research project funded by Ministry of Science and Technology of Korea, to evaluate shear stress-strain histories in soil directly from the free-field downhole accelerations (Elgamal, et al, 1995). KEPRI is to develop the spatial variation functions, which can provide a complete description of the statistical properties of the horizontal components of the seismic wavefield based on the recorded earthquake data of Hualien LSST model (EPRI, 1992).

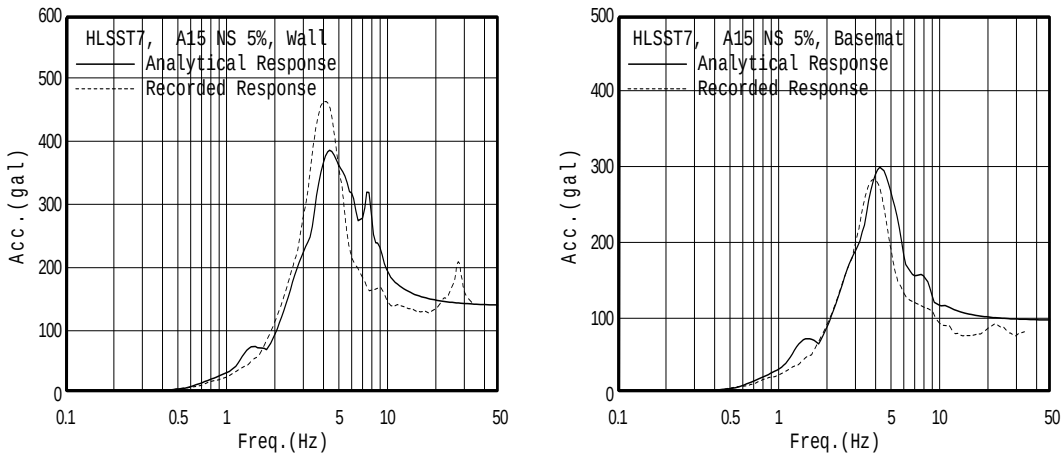


Fig. 4 Comparison of analytical results with recorded results

5. Seismic Capability Evaluation for Equipments

The seismic capability evaluation of equipments should demonstrate an ability to perform its required function during and after the time, which is subjected to the forces resulting from design basis earthquake. The IEEE 344 described the seismic test and analytical methods as seismic qualification methodologies for the mechanical as well as electrical equipments.

There are three USI A-46 "Seismic Qualification of Equipment in Operating Plants," and eight USI A-40 "Recommended Revisions to NRC Seismic Design Criteria" NPPs in Korea. For the resolution of the USI A-46, EPRI has formed the seismic qualification utility group (SQUG) and developed the generic implementation procedure (GIP), which is an alternative verification method for seismic adequacy of the equipment (SQUG, 1993). For the resolution of the USI A-40 Lawrence Livermore National Laboratory funded by U.S. NRC has performed a long-term research program, seismic safety margin research program (U.S. NRC, 1984). The current licensing basis has been developed on a basis of the results of this program. IAEA recommended that all the operating plant including A-46 plants should perform a PSR to comply with the current licensing basis.

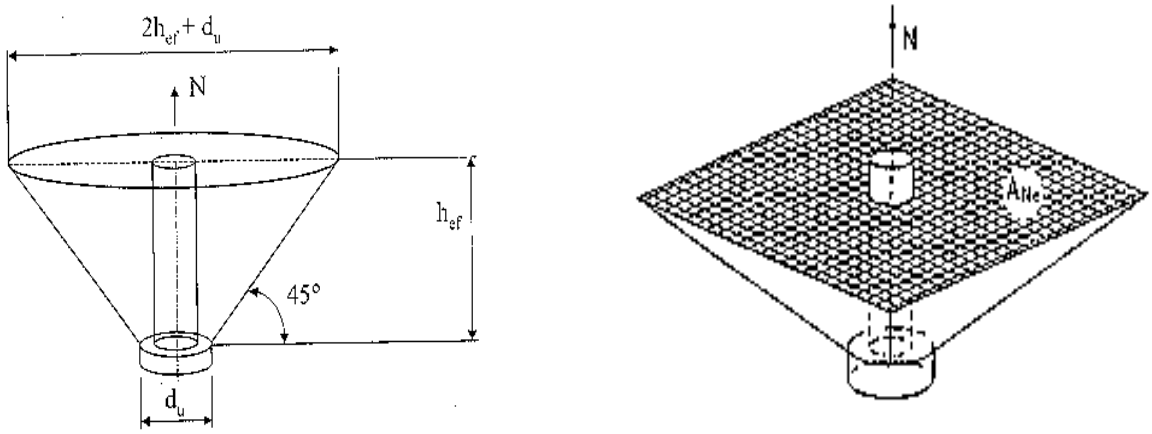
For the resolution of the seismic safety issue, the use of two approaches, that is, seismic margin assessment (SMA) and seismic probabilistic safety assessment (SPSA) are allowed (EPRI, 1991, U.S. NRC, 1985).

In order to resolve the above problems related to the seismic safety issues, KEPRI has joined the SQUG since 1998 and is developing the efficient combined methodologies to evaluate the seismic capability of equipments for the operating NPPs. The new combined methodology requires both GIP and SMA simultaneously, which are based on the seismic walkdown. The seismic walkdown is known to be the most effective tool to find seismic deficiencies of the existing plant and can achieve the cost benefit for the seismic safety review.

The design of anchor system for fastening the equipments and piping systems, etc. in Korean NPPs has been done based on the ACI 349 (See Fig. 5). Recently, concrete capacity design (CCD) method of Euro-International Committee for concrete (CEB) code, which was processed in Europe, indicates that anchor system designed by ACI 349 may not satisfy required tensile and shear capacity (Fuchs, et al, 1995). Also, a lot of papers related to this subject represent new tendency to make some amendment to ACI 349 code or new design code based on CCD method of CEB code (EPRI, 1991). According to U.S. NRC standard review plan published in 1996, NRC acknowledged that ACI 349 would overestimate the real capacity of anchor system for fastening equipments and recommended that anchor system should be designed by test

results for each case. Therefore, in order to evaluate the tensile and shear capacity of existing anchorage system, KEPRI is carrying out actual model tests.

In addition, KEPRI makes an attempt to identify the effect of high frequency on equipments, reduction of uncertainty within the seismic fragility and develop the generation of generic equipment ruggedness spectrum (GERS) for the GIP.



(a) ACI 349 Code (b) CEB Code

Fig. 5 Failure modes for fastenings under tensile loading

6. Reduction of Uncertainty in Probabilistic Seismic Hazard Analysis

Several research works were performed to reduce the excessive uncertainty involved in the previous PSHA by improving credibility of main PSHA input data such as ground motion attenuation relations, earthquake catalogs, and seismic source characterization. A new PSHA methodology is established not only to incorporate the state-of-art technologies of PSHA but also to be compatible with revised regulation requirements.

Revised PSHA inputs are obtained by employing comprehensive technical approaches of data collection and analysis, experiments, theoretical study, and etc. to explicitly account for and reduce the uncertainty in scientific knowledge related to the PSHA input data. Using these new PSHA inputs, a PSHA is performed as a case study on the Wolsung NPP site to demonstrate the improved seismic hazard results.

The temporal and spatial characteristics of seismicity in the Korean Peninsula were analyzed and compared with the neighboring intraplate regions. A close temporal correlation was identified among Korea, China, and Japan.

A strong motion attenuation equation was estimated by using the intensity data of small to medium size earthquakes and the formula for the Eastern North America. The coda Q value for the southeastern part of Korea was estimated by using the small earthquakes recorded recently. Crustal structure of the Korean Peninsula was identified with limited broadband teleseismic and deep-focus earthquake data.

EQHAZARD software package developed by EPRI (EPRI, 1989) was modified to implement the state-of-art technologies and revised regulations related to PSHA. Seismic source interpretations were collected for the region of Korea for inputs to PSHA. A team approach to the EQHAZARD code was adopted.

Using revised PSHA inputs and EQHAZARD code obtained from this research, the PSHA on Wolsung NPP site was performed considering the incompleteness of the Korean earthquake catalog (Yun, K.H. and Lee, J.R., 1999). The new PSHA result shows reduced seismic hazard level over the previous PSHA result, which has the effect of significantly reducing the level of core damage frequency (See Fig. 6). Although this study has solved some issues related with PSHA, there still exist problems to be solved in the future. Acquisition of strong motion data is needed and studies on seismology, geology, and geotectonics are required to further reduce uncertainties.

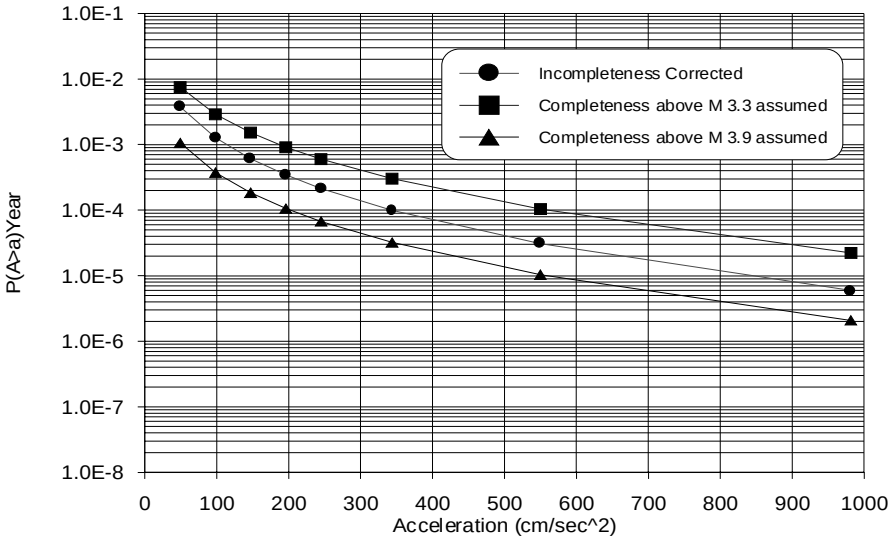


Fig. 6 Seismic hazard results according to the methods considering the incompleteness of the earthquake catalog

7. NPP Response to an Earthquake

Small earthquakes may exceed the OBE spectrum in the high-frequency range, without causing damage. An analytical criterion for determining when an OBE is exceeded should be set up. However, a comprehensive plan for plant response must include provisions for assessing plant physical condition and readiness for shutdown, shutdown decision criteria, and procedures for plant evaluation and restart.

A comprehensive plan is made and procedures for NPP response to an earthquake are developed in order to determine 1) the effects of an earthquake on physical conditions of Korean NPPs, 2) if shutdown of the plant is appropriate based on observed damage to the plant or due to an exceedance of the OBE, and 3) the readiness of the plant to restart following a shutdown due to the earthquake.

Guidelines for nuclear plant response to earthquakes enable operators to quickly evaluate the need for post-earthquake plant shutdown and provide procedures for evaluation of earthquake effects on the plant, as well as criteria for plant restart. The guidelines consist of short-term actions, post-shutdown inspections and tests, and long-term evaluations. The short-term actions, which could be completed within four to eight hours of the earthquake, consist of operator walkdown inspections, evaluation of ground motion records, and pre-shutdown inspections. The long-term evaluations would normally be performed after the NPP has returned to service. Restart of the plant following shutdown due to an exceedance of the OBE would be based on the results of the post-shutdown inspections and the successful completion of surveillance tests and operability tests

The NPP should be shutdown for inspections and tests prior to a return to power if the earthquake exceeds the OBE. The OBE at the plant is considered to have been exceeded if the computed CAV from the earthquake record is greater than 0.16g-sec. However, the CAV criterion should be determined considering the seismic and structural characteristics of the plant.

An experimental study using shaking table is conducted in this study to evaluate intensity of CAV criterion. Appropriate level of CAV is evaluated based on the test results using the developed SDI model (See Fig. 7).

Fig. 7 The developed Seismic Damage Indicator

The model consists of stacked acrylic cylinders and is developed to behave consistently for each directional seismic load. The result of the experimental study indicates that the CAV criterion of 0.16g.sec is conservative enough to be applied to Korean NPPs since the CAV value of the seismic input motion of the Korean standard NPPs ranges from 0.3 to 0.5 g-sec (KEPRI, 2000). The developed SDI is expected to be useful not only in easily determining OBE exceedance but also in evaluating earthquake damage quantitatively to provide guidelines for better post-shutdown inspection and test.

8. Evaluation of Tsunami Risk

A safety review for Ulchin NPP against tsunami was made on the basis of maximum earthquake magnitude 7 $^{3}/_{4}$ and available tsunamigenic earthquake fault parameters (KOPEC, 1986). But, recently, based on the seismic gap theory some geologists and seismologists warned that the earthquakes with larger magnitude than were expected might occur in the East Sea region (Ishikawa, 1994).

For the safety review, an explicit finite difference model is employed based on the linear and nonlinear shallow water equations. Also, using earthquake fault parameters, the vertical sea bottom displacement at the event of earthquakes are estimated. It is assumed that the initial profile of tsunamis is equal to the sea bottom displacement without any modification by hydraulic effect because the horizontal scale of tsunami source area is large enough to water depth in most cases.

In Korea, the maximum run-up height of tsunamis was observed at the Imwon Harbor located about 20 km northward from the Ulchin NPP in 1983 East Sea Tsunami (See Fig. 8). Therefore, the 1983 East Sea Tsunami is simulated and compared the calculated water surface profile with the observed wave heights. Finally, the water level rise and fall will be evaluated at the Ulchin NPP site, with the numerical model developed in this study.

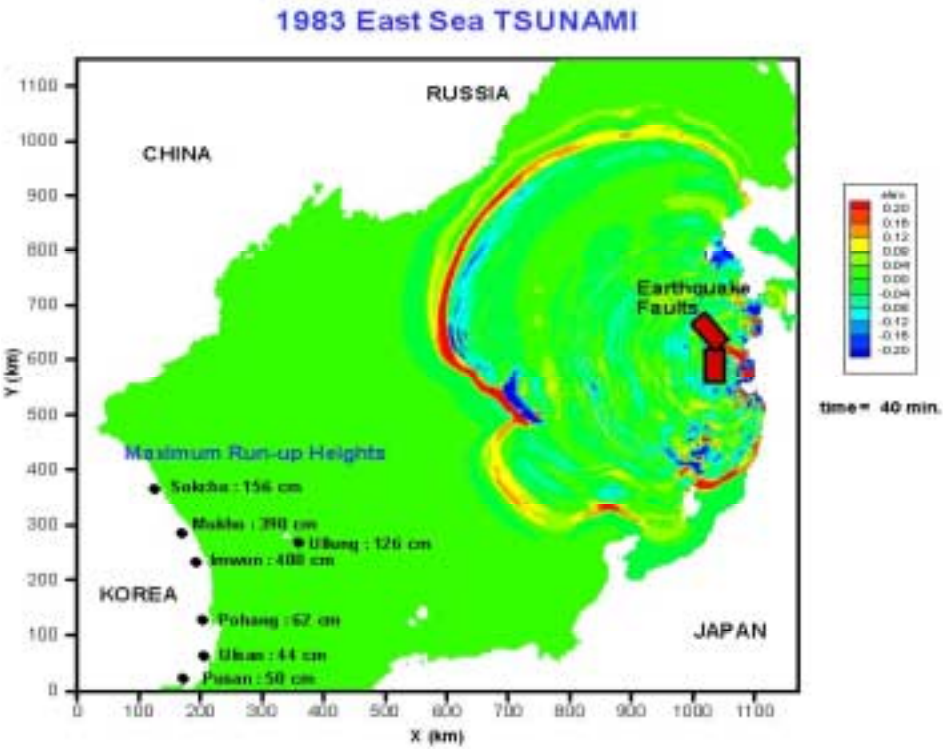


Fig. 8. Snapshot of tsunami wave propagation and max. run-up heights in 1983 East Sea Tsunami

9. Future and Conclusions

PSRs for the Korean NPPs have been implemented since 2000. Based on the research activities related to seismic re-evaluation for the existing NPPs, several technical issues are briefly introduced with countermeasures and improvement. For evaluation of capable faults and estimation of maximum credible earthquake for NPP sites, it is required to develop a capable fault criteria suitable for Korean geological environments. Based on the new capable fault criteria, previously known Quaternary faults will be re-evaluated for the activity of faults and its effect on NPPs. Historical earthquakes should be reevaluated to estimate maximum credible earthquake.

Based on the measured earthquake recorded, an attempt was made to develop a site-specific response spectrum. It is expected that earthquake observatory networks installed on the outskirts of NPP sites will be of great use to identify seismic activity. For reducing uncertainties of the PSHA, a research work was performed. However, uncertainties of the input parameters for PSHA should be reduced. For better PSHA an attenuation curve based on the strong earthquakes is required. Seismic qualification of equipments for the USI A-46 NPPs is being undertaken. GERS for Korean NPPs are required and anchor system for equipments should be reevaluated with seismic walkdown. The post-earthquake procedure including selection of base lines and a seismic damage indicator is developed to provide guideline for better post-shutdown inspection and test. Further investigation is required to develop a guideline for long-term evaluations.

It is expected that the results of PSR for operating NPPs be of great use not only for feedback of the design improvement for future NPPs but also for evaluation of the old NPPs for life extension.

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References

1. Astumasa Okada, Yangsan and Ulsan Active Fault Systems in the Southeastern Part of Korean Peninsula", Korea-Japan Geomorphological Conference Proceedings, pp. 13-15 (In Japanese), 1999.
2. Lee, K., Jin, Y.G., Segmentation of the Yangsan Fault System: Geophysical Studies on Major Faults in the Kyeongsang Basin, Jour. Geol. Soc. Korea, Vol. 27, pp. 434-449, 1991.
3. Kyung, J.B., Paleoseismological Study on the Mid-northern Part of Ulsan Fault by Trench Mehtod, Jour. Eng, Geol., Vol. 7. Pp. 81-90. (In Korean), 1997.
4. KEPRI, A Study of Major Seismic Safety Issues for Nuclear Power Plants – Hualien Large Scale Seismic Test, KRC-90H-J05, 1995.
5. KEPRI, Development of Seismic Safety Assessment Technology for Containment Building, TM.99NE02.P2000.135, 2000.
6. Elgamal, A.W., Zeghal, M., Tang, H.T. and Stepp, J.C., Lotung Downhole Array. I: Evaluation of Site Dynamic Properties, *Journal of Geotechnical Engineering, Vol. 121, No. 4*, 350 – 362, 1995.
7. EPRI, Spatial Variation of Earthquake Ground Motion for Application to Soil–Structure Interaction, TR-100463, 1992.
8. SQUG, Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment (Rev. 2A), 1993
9. U.S. NRC, A Research Program for Seismic Qualification of Nuclear Plant Electrical and Mechanical Equipment, NUREG/CR-3892, 1984.
10. EPRI, A Methodology for Assessment Nuclear Power Plant Seismic Margin (Rev. 1), NP-6041, 1991.

11. U.S. NRC, Simplified Seismic Probabilistic Risk Assessment: Procedures and Limitations, NUREG/CR-4331, 1985.
12. Fuchs, W., Eligehausen, R and Breen, J.E., Concrete Capacity Design (CCD) Approach for Fastening to Concrete, *ACI Structural Journal, Vol. 92, No. 1*, 73 – 94, 1995.
13. EPRI, Seismic Verification of Nuclear Plant Equipment Anchorage (Rev.1), NP-5228-SL, 1991.
14. EPRI, Engineering Characterization of Strong Ground Motion Recorded at Rock Sites (EPRI TR-102262), 1995.
15. Boore, D.M., Stochastic simulation of high-frequency ground motions based on seismological models of the radiated spectra, Bull. Seism. Soc. Am., 73, 1865-1884, 1983
16. Yun, K.H. and Lee, J.R., Reduction of the Seismic Hazard Considering the Incompleteness of the Korean Earthquake Catalog, 15th Int. Conf. on Structural Mechanics in Reactor Technology, Seoul, 1999
17. EPRI, Seismic Hazard Methodology for the Central and Eastern United States, 1989.
18. KEPRI, Development of Post-Earthquake Guideline and Evaluation Methodology for Nuclear Power Plants, 2000
19. KOPEC, A study of tunami impact on the nuclear power plant unit 9&10 (in Korean), 1986
20. Ishikawa, Y., Remained seismic gaps, Gekkan Kaiyo 7 (in Japanese), 1994

Reassessment Philosophy for the Seismic Safety of NPPs in Japan

Takaaki Konno
Secretariat of Nuclear Safety Commission
Cabinet Office of Japan

ABSTRACT

After the JCO accident that occurred at the uranium processing facility in Tokai-mura, the Japanese Nuclear Safety Commission (NSC) decided to do review and reassessment in the subsequence regulation to improve the regulatory effectiveness. The basic cause of the JCO accident was the unsafe work procedures along with violation of regulations. This accident also led to criticism of the overseeing authorities, for their not noticing the incorrect activities of the operator over such a long period of time. Safety design, construction and operation should be performed without any degrading the safety performance from the plant-specific safety assurance defined in the siting permit. To confirm plant safety into the future, reassessment of the plant safety performance is essential and the results should be reflected in the subsequence regulation to help safety management. Seismic Probabilistic Safety Assessment (PSA) is a suitable method for reassessment of seismic safety performance. Seismic PSA should be conducted following a three-phase procedure according to the status of plant design and operation conditions. In the first phase, the seismic safety assessment scenario is confirmed from the seismic design specification in the safety design assessment report submitted for siting license application. In the second phase, the seismic PSA is performed with the detailed design and construction data before the operation permit. In the third phase, the seismic PSA should be performed based on a walk-down inspection of the plant operation. Development activities of seismic PSA methodology are on-going in Japan, to be utilize in a risk-informed safety management not only concerning prevention of severe accident but also concerning shutdown management, on-line maintenance, in-service inspection, and in-service testing. The NSC is still discussing of the reassessment methodology and how to reflect the reassessment results in the regulation. This paper present a philosophy of the reassessment of the seismic safety performance of NPPs using seismic PSA, to be reflected in the subsequence regulation.

INTRODUCTION

Usually the Japanese NSC conducted a nuclear safety assessment only in the first phase of the regulation that is required as a double check for the siting permit. After the JCO accident that occurred at the uranium processing facility in Tokai-mura, the NSC decided to review and reassessment the regulations to improve regulatory effectiveness. The basic cause of the JCO accident was the unsafe work procedures and violation of regulations. This accident also led to criticism of the overseeing authorities, for their not noticing the incorrect activities of the operator over such a long period of time.

Reassessment is important for ensuring the seismic safety of NPPs that are designed, constructed, operated and maintained in such a way that their impact on public health and safety is as low as reasonably achievable. Such reassessment must consider not only the dynamic interaction of earthquake hazard and reactor system but also consideration of dynamic interaction among human, technology, and environment.

The safety design, construction and operation should be performed without degrading the safety performance from the plant-specific safety assurance that was specified in the first phase permit. To confirm the validity of the safety assurance, reassessment of the plant safety performance is essential and the results should be reflected in subsequent regulation to help safety management. Seismic PSA is a suitable method to do reassessment of the seismic safety performance.

Recent inland earthquake experiences such as 1995 Hyogo-ken Nanbu Earthquake Mj=7.2 or 2000 Tttori-ken Seibu Earthquake Mj=7.3 have increased public concern about nuclear safety. In the public opinion, prompt review of the current Seismic Safety Guideline is needed, based on the recent seismic technology and the performance reassessment of the seismic safety of existing NPPs. The damage belt of the 1995 Hyogo-ken Nanbu earthquake Mj=7.2 that struck the densely populated city of Kobe demonstrated to the public the dangerous power of the near-fault inland earthquakes. Many buildings including wooden houses and structures such as bridges, port facilities, subways and rail lines were badly damaged.

Recently, the 2000 Tottori-ken Seibu earthquake Mj=7.3 occurred at a previously unknown active fault for which the epicenter was located approximately 45 km south-east from an existing NPP. This caused discussion about the sufficiency of the blind fault earthquake magnitude that is required; commonly a magnitude 6.5 earthquake at the hypocenter distance of 10 km is used for seismic design of every NPPs.

These recent earthquake experiences awoke public concern about the seismic safety that the current guideline considers necessary for review based on the state of the art knowledge of seismology, geology, and earthquake engineering to ensure the seismic safety performance of NPPs. Recent advances in seismic design technology brings the impression among some seismic engineering experts that the current guideline is outdated. They consider that further advancement in the Guideline is necessary based on the recent seismic technology and on recent earthquake experiences.

Development activities of seismic PSA methodology are ongoing within the Japan Atomic Energy Research Institute (JAERI), Nuclear Power Engineering Corporation (NUPEC), electric power utilities, and other organizations in Japan. This methodology is being applied to risk-informed safety management not only concerning prevention of severe accident but also concerning shutdown management, on-line maintenance, in-service inspection, and in-service testing.

The NSC still be discussing the reassessment methodology and how to reflect the reassessment results to the regulations. This paper presents a philosophy of the reassessment of the seismic safety performance of NPPs using seismic PSA to be reflected in the subsequent regulation.

PHILOSOPHY OF NPP SEISMIC SAFETY

The objective of reactor safety is that reactors will be built and operated to pose no undue risk to public health and safety. The current seismic design guideline interpreted the reactor safety philosophy to the seismic safety of NPPs that reactor safety should not be compromised by any possible earthquake ground motions at the site. This philosophy of the seismic safety requirement is converted to the seismic design criteria of reactor facilities and categorization of the design earthquakes to meet adequate engineering solutions to actual or potential geologic and seismic effects at the proposed site. To meet this seismic safety requirement, design basis earthquakes are categorized into four classes, from large magnitude/low probability to small magnitude/high probability, in accordance with the reactor facility's functional

classification from the most important, important, moderate important, and non-nuclear facilities. This seismic design requirement is intended to consider sufficient range of earthquakes to assure reactor safety for any potential earthquake shaking.

However current deterministic design practice to meet this seismic design requirement does not suitable to explain thoroughly the relationship between reactor safety performance and the strengths of structures, equipment and components. This is because uncertainties both of the seismic hazard among earthquake magnitude, occurrence, ground motion attenuation and of the seismic safety performance within the complex functional classification of reactor facilities.

A probabilistic approach to seismic safety assessment is important from the point of view of "How safe is safe enough." Such an approach takes into account the ground motion from the full range of earthquake magnitudes, allowing explanation of the relationship between the reactor safety performance and the strengths of structures, equipment, and components considering the uncertainties within the seismic hazard and the safety performance system. The probabilistic approach to seismic hazard characterization is very compatible with current trends in earthquake engineering and the development of building codes, which have embraced the concept of performance-based design. The objective of the performance-based design is to clarify how reactor safety performance is degrading with the increasing magnitude of earthquakes.

In the current Guideline, the Design Earthquakes are categorized in accordance with the reactor facility's classification to range from extremely large magnitude/low probability of occurrence to small magnitude/high probability of occurrence as follows: extreme design earthquake (S2), maximum design earthquake (S1), 1.5 times a non-nuclear facilities design earthquake, and non-nuclear facilities design earthquake. The extreme design earthquake S2 is determined based on earthquakes from active faults both of the frequent activity fault and less frequent activity fault whose recurrence interval is shorter than 50,000 years. This S2 earthquake hazard is based on the seismo-tectonic structure and on possible blind faults, considering that an earthquake of magnitude up to 6.5 could take place at any inland location in Japan. The maximum design earthquake S1 is determined based on historical earthquakes, earthquakes due to active faults with frequent activity whose recurrence interval is shorter than 10,000 years.

Current deterministic evaluation method of the design earthquake ground motion is often criticized from the reason that uncertainties incorporated within the earthquake simulations are not explicitly considered. It has been recommended that modern method for evaluation of design earthquake ground motions for reactor facilities should consider fault rupture process and directivity of the wave propagation in the near-fault earthquake from the point of recent knowledge on earthquake ground motions.

One approach to account more realistically for these effects in ground motion models is to include them in empirical models by using a large number of predictive parameters related to source, path and site conditions. Another approach is to use seismologically-based ground motion models that take account of the specific source, path and site conditions.

If numerical simulation methods are used to estimate the ground motions, then the spatial distribution of slip on the fault and the time function of slip on the fault also needs to be characterized. Irikura proposed a recipe for prediction of scenario earthquake strong ground motion caused by active fault by means of numerical analysis (Irikura, 2000) [1]. Strong ground motions in the near-source area are controlled by heterogeneous source processes. Source characterization includes source effects such as those due to rupture directivity or the orientation of fault, and the effects of deep structure such as sedimentary basins, basin edges, and buried folds and faults therefore is one of key issue for more precise strong ground motion prediction. The new, high-quality data recorded in the near-source region of recent large earthquakes are useful to evaluate source characterization such as spatial variations of slip, slip velocity, or rupture velocity for accomplish precise strong motion prediction by modern earthquake ground motion evaluation technology.

A probabilistic approach to characterizing the ground motion that a given site will experience in the future is very compatible with the current trends in earthquake engineering and the development of building codes towered performance design. Probabilistic evaluation method of the design earthquake ground motion based on the seismic hazard analysis is thought to be a suitable method to handle theoretically and quantitatively the uncertainties among earthquake magnitude, occurrence and ground motion attenuation.

If a probabilistic approach is used, then the ground motions from a large number of possible earthquakes are considered, and their frequencies of occurrence are key parameters in the analysis. Probabilistic analysis requires an important category of additional information that the expected frequencies of occurrence of earthquakes of various magnitudes on each potential seismic source. The site specific hazard curves, from which the requested sets of Uniform Hazard Spectrum may be obtained, should also accommodate uncertainty in the site specific dynamic material properties as well as local and regional seismicity and attenuation characteristics. One of the objectives in developing seismic design spectra is to achieve approximate uniformity of seismic risk for structures, equipment, and components designed to those spectra, across a range of seismic environments, annual probabilities, and structural frequencies.

Seismic safety performance of the structures, equipment and components are required to prove that the ultimate strengths have the capacity not to cause severe reactor accident due to the any earthquake event. Seismic capacity data of passive components, active components and electrical equipment on Japanese standard plants are accumulating. For the active components and the electrical equipment whose failure modes are functional, they are evaluated on the basis of data from shaking table tests performed in Japan and from engineering judgment. The capacity data of active components and electrical equipment are determined by both capacities due to structural failure and functional failure mode.

Seismic PSA is a suitable method to confirm the seismic safety of NPPs that are designed, constructed, operated and maintained in such a way that their impact on public health and safety is as low as reasonably achievable, with the consideration of not only dynamic interaction of earthquake hazard and reactor system but also dynamic interaction among human, technology, and environment.

Safety science concerning dynamic interaction among human, technology, and environment has long been important vehicle to drive properly the nuclear safety in the public risk and health. Safety management based on the risk assessment by the seismic PSA, the plant operation, maintenance, inspection and severe accident managements are expected to be rationally improved considering the effect of human factors. Human factors are represented from individual, group, and organizational factors. From the JCO criticality accident on 30 September 1999, it was clear that the accident was a typical organizational accident, emphasizing importance of the organizational factors for safety management.

Seismic PSA should be conducted following a three-phase procedure to fit the Japanese regulatory system, according to the progress of plant design and operation conditions; in the first phase, the seismic safety assessment scenario is confirmed depending on the seismic design specification in a safety design assessment report submitted for siting license. In the second phase, the seismic PSA is performed with the detailed design and construction data before the operating permit. In the third phase, the seismic PSA is certified by reassessment based on the plant walk-down inspection due to the plant operation.

Usually seismic PSA focuses on evaluation of core damage frequency. However, safety management concerns not only prevention of severe accidents but also concerns shutdown management, on-line maintenance, in-service inspection, and in-service testing. Risk-informed safety management should be developed based on defense-in-depth philosophy as one of the ways to strengthen public confidence in the regulatory system. Safety management structures a software barrier system of NPPs. Multiple barriers both of hardware systems and software systems are useful ensuring nuclear safety assurance.

THE IMPORTANCE OF HUMAN FACTORS

After the JCO accident, the NSC recognized the importance of the human factors in risk analysis. It is further important that nuclear safety experts continue to improve their understanding of the risks from nuclear reactors and communicate that information to the public. The technical and individual human side of accident causation has since long been in the center of attention of safety sciences (Vuuren, 1999) [2].

The JCO accident suggesting that the development and improvement of hardware barriers is not yet sufficient to ensure the safety assurance, and that the creation and materialization of software barriers are also important from the defense-in-depth philosophy. Various types of incidents appeared recently to have

inherent problems. Comparison analysis of these incidents and evaluation of root cause of them are required.

In any organization, people tend to give priority to inner logic, such as importance of economy or quality assurance of products, and if a safety level of operation is maintained, safety activities and importance on safety criteria will deteriorate gradually. This kind of tendency is an open invitation for internal errors or violation of procedures. New regulatory policies considering safety culture and safety evaluation processes for constructing the system to promote safety culture and for checking internal errors such as deviations or negligence will be highly required.

The concept of a safety culture has not yet found a commonly shared understanding. This concept can serve as an important vehicle to foster comprehensive concerns with individual, group, organizational and managerial as well as inter-organizational factors relevant to safety in high hazard industries. Theoretically-based assessment techniques are important for the need to equip NPP staff with practical methods for self-assessment and regulators for external assessment of nuclear operations. Practical issues emerge particularly in the domain of organizational development, i.e. the goal-oriented efforts to change the structures and the functioning of nuclear operations in such a way that the desired outputs in terms of safety and reliability result in a sustained fashion.

Respect of inner logic in the organization has a tendency to oppose the spirit of respect for safety culture. Respect for safety culture should be developed and the procedure to evaluate it should be included in safety regulation. The NSC has the duty to evaluate safety culture and also to construct the structure to keep it intact. Only a close cooperation among scientists from various disciplines and of practitioners holds the promise of adequately understanding and use of organizational factors in further improving the safety record of nuclear industry.

Reassessment of nuclear installations with the consideration of not only dynamic interactions of earthquake hazard and reactor system but also dynamic interactions among human, technology, and environment in the after the siting phase based on the detailed data of plant design, construction and operation is important ensuring the seismic safety that are designed, constructed, operated and maintained in such a way that their impact on public health and safety is as low as reasonably achievable.

Ensuring the safety assurance, discussion of the following items are important to propose effective countermeasures: What is safety, How safe is safe enough, What is the methodology to prevent the degradation of safety culture. Development of the methodologies for technology transfers, probabilistic safety assessments, human reliability analysis, root-cause analysis, and documenting techniques are also important.

CONSEPT OF REASSESSMENT

Reassessment should be conducted for the following two phases of NPP progress: 1) detailed design and construction phase, and 2) the operation phase. In the plant detail design and construction phase, the seismic PSA should be performed based on the detailed design and construction data. In the plant operation phase, the seismic PSA should be performed based on the plant walk-down inspection during plant operation.

Safety assessment is performed to answer the questions from the point of verification, validation and certification on the evaluation depth in high hazard systems such as nuclear installations (Rasmussen, et al., 1994) [3].

- Verification is an assessment of the degree to which the results meet the requirements of the design specification. Verification is supposed to answer the questions: Is the design right? Does the product meet the design intentions?

- Validation is an assessment of the degree to which the design achieves the original system objectives. Validation is thus supposed to answer the questions: Does the product meet the needs of the end user? Is it the right design?
- Certification is a particular type of validation with a focus on the constraints around the original system objectives. This explicit focus is particularly important when advice systems are introduced, which are based on heuristic rules, as in expert systems. While it is practically possible to validate the systems within the design basis, it is very difficult to certify that the response of heuristic rules to unpredicted situations outside the design basis will not have unacceptable side effects. Thus, certification of software and hardware will very become a major concern for regulatory bodies.

In the recent review of the evaluation problem in high hazard systems such as nuclear power plant, Tanabe emphasized the need to explicitly evaluate the potential side effects of system functions during abnormal operational conditions (Tanabe,1991) [4]. That is, in the validation of system objectives, explicit considerations are necessary for certification.

The effectiveness of hardware systems can be evaluated in a design phase but the thorough evaluation of the effectiveness of software systems need to be performed in operation phase. To evaluate the safety management, a performance-based approach is necessary for regulatory body to define the effectiveness and efficiency. A comprehensive set of Performance Indicators for regulatory should be developed.

REASSESSMENT IN CONSTRUCTION PHASE

Review and reassessment in the plant construction phase is intended to confirm the safety assurance that was specified in the site license. For the reassessment of the seismic safety assurance, seismic PSA in the plant construction phase is performed based on the detailed design and construction data before the operation permission.

Characteristic analyses of the seismic design system, construction system, and inspection system before operation should be performed to answer the question from the point of the verification and/or the validation in the construction phase. The characteristic analyses of production method and the procedure will be performed based on the actual procedure. The reassessment results are utilized to the safe operation standard manual as the issues to be concerned.

In the safety review of the basic design, the following should be considered as preventive measures: Consideration of the possibilities, if any, of deviating from these conditions when these are put to use; Required implementation of specific safety designs against not only wrongful operation but also intentional error while taking into consideration the potential event. Returning to the original concept of “Defense-in-depth” fail safe, fool proof, and preventive measures for intentional error should be taken at the facilities.

Inspections considering software aspects is important for revealing the quality of operability and utility. Inspection systems to check safety activities of facilities by regulations should have resident inspectors, periodic safety reviews, systems for qualification of shift supervisors, qualification of engineers for nuclear fuel material handling, guidance for performing probabilistic risk assessment studies and preparing severe accident management measures, and other features.

It should be required to prove the seismic safety performance of the structures, equipment and components. Seismic capacity data for component fragility evaluation based on the structure failure models were obtained for many component categories of Japanese standard plants; for examples, pressure vessel and its support, primary loop recirculation system, reactor core internal structures, control rod drive hydraulic unit, tanks, gas insulation switchgear, and so on.

Seismic safety reliability-proving tests that have been conducted by the Nuclear Power Engineering Corporation (NUPEC) provide essential data for evaluation of fragility curves of the structures, equipment and components. Seismic safety analyses based on the nonlinear response, 3-D response and ultimate

strength of the total system of structures, equipment and components are desirable to confirm the seismic performance into ultimate conditions.

REASSESSMENT IN THE OPERATION PHASE

In the plant operation phase, the technical ability and the safety operation management that were confirmed to have adequate safety assurance in the siting license phase should be certified by periodic inspection and reassessment in subsequent regulation.

A reassessment methodology for the procedural capabilities for plant normal operation, shutdown management, severe accident management, on-line maintenance, in-service inspection, and in-service testing should be developed to keep a high level capability based upon a safety culture.

Risk-informed reassessment methodologies for the normal operation management, shutdown management, severe accident management, on-line maintenance, in-service inspection, and in-service testing should be developed as one of the ways to strengthen public confidence in the regulatory system.

The performance of the seismic PSA based on plant walk-down inspection concerning to the reactor safety function are maintained properly to act surely in the earthquake, the deterioration of the low level safety function does not injured the high level safety function due to plant operations is essential for the reassessment of the seismic safety assurance in the plant operation phase.

For the seismic safety in the operating phase, the performance of operator support systems under seismic condition is important. Operator should be provided with operational guides related to prioritized systems and components based on safety shutdown path for preventing core damage, considering that some of them may be inoperable due to seismic motion. The prioritization means the order of systems and components that operators should use to prevent core damage. The method of prioritization is as follows: 1) systems actuate automatically, 2) systems less dependent on support systems, and 3) systems with sufficient seismic capacity.

According to the results of many seismic PSAs, the situations which may happen due to an earthquake could be classified into the following; 1) Continue plant operation, with some inspection or surveillance of systems and components are necessary, 2) Reactor scram occurs, 3) Loss of off site power (LOSP) as well as reactor scram occurs, and 4) Small loss of coolant accident (LOCA) in addition to reactor scram occurs.

DEVELOPMENT OF SEISMIC PSA

Development activities of seismic PSA methodology is ongoing within JAERI, NUPEC, electric power utilities, and other organizations in Japan. This methodology will be utilized in a risk-informed safety management not only concerning prevention of severe accident but also concerning shutdown management, on-line maintenance, in-service inspection, and in-service testing.

A case study of seismic PSA and seismic margin analysis (SMA) for a Japanese standard BWR was performed as an objective to establish seismic PSA methodologies for Japanese plant [5]. In the study, an emphasis was made on the following points: first, to study a procedure of integrating expert opinions in the seismic hazard evaluation; second, to develop fragility database for domestic plant components; third, to study dominant contributors to core damage frequency. The seismic PSA results showed that dominant contributors were common cause random failure and seismic failure of several components and that random failure had a large portion of contribution to core damage frequency (CDF). On the other hand, the SMA results showed that dominant contributors were seismic failure of several components. The

dominant contributing seismic acceleration region was around (0.6-2.5) times S2 in seismic PSA, while the SMA results gave a plant capacity of High Confidence Low Probability of Failure (HCLPF) value about 2.5 times S2, which was larger than the contributing acceleration region in seismic PSA. Dominant accident sequences obtained were almost the same in seismic PSA and SMA.

Seismic margin analysis (SMA) for Japanese ABWR was performed for a seismic risk assessment project that was recently started in a number of Japanese BWR utilities [6]. The objective of this analysis is: to demonstrate earthquake-related safety margin of the plant by a probabilistic assessment methodology; to gain new insights to make the NPP even safer. The study was consisted from four tasks: 1) event tree development, 2) fault tree development, 3) fragility evaluation for structures, systems and components, and 4) seismic margin analysis for accident sequences. High Confidence Low Probability of Failure (HCLPF) value of Japanese ABWR has been evaluated to be about 1.8 times of the peak ground acceleration of design basis earthquake, and this value indicates that this plant has large seismic margin against design basis earthquake. The accident sequences in which the margin is relatively low, is the station blackout sequence, and the major critical components are as follows; Air conditioning duct, Piping of RCW, RSW, and RCIC, and inside containment, RCW pump, Valve for containment venting.

Recognizing the potential importance of operator support system under seismic conditions, the Japan Atomic Energy Research Institute (JAERI) has started a feasibility study to develop a concept of an Operator Support System under Seismic Conditions (OSSC) as one of seismic risk management strategies. A conceptual design of operator support system under seismic conditions was proposed utilizing the results and findings from seismic PSA [7]. If a large earthquake occurs near a nuclear power plant, it may cause abnormal situations to the NPP such as occurrence of multiple initiating events and failures of mitigation systems. Many difficulties may arise in diagnosis of the plant status and actions operators and technical support staff of the NPP due to highly stressful conditions. In the case of the 1995 Hyogo-ken Nanbu earthquake, many thermal power plants, substations, and transmission and distribution facilities near the epicenter suffered various types of damages from the earthquake. As many alarms and annunciators sounded in unison at that time, it was difficult for operators to move to approach the panels or consoles to confirm and stop them [8]. Considering that the stress may be induced to the operators by earthquake and some of the engineered safety systems actuate automatically in short term, it is important to provide operators with guides for confirmation of the systems to be started automatically and manipulation which operators ought to perform in a short term. On the other hand, for the middle and long terms, it is important to provide operators with guides for assisting diagnosis of the plant status and for responding to multiple failures in the plant. The primary aim for assist function provided to the operators should be changed depending on the level of severity of earthquake motion at the plant.

The evaluation of fragility is important factor in the seismic PSA. Seismic performance of the structures, equipment and components are required to prove that the ultimate strengths have the capacity not to cause sever reactor accident due to the any earthquake events. Seismic capacity data for components fragility evaluation based on the structure failure models were obtained for many component categories of Japanese standard plants; for examples, pressure vessel and its support, primary loop recirculation system, reactor core internal structures, control rod drive hydraulic unit, tanks, gas insulation switchgear, and so on [9]. The capacity data of passive components were evaluated on the basis of the safety factor method using the results of seismic design analyses. For the active components and the electrical equipment whose failure modes are functional, they were evaluated on the basis of data from shaking table tests performed in Japan and from engineering judgment. The capacity data of active components and electrical equipment were determined by both capacities due to structural failure and functional failure mode. The value of capacity for major components in Japanese LWR plants generally proved to be relatively high.

The Nuclear Information Center of Central Research Institute of Electric Power Industry (CRIEPI) serves utilities by providing safety-, and reliability- related information on operation and maintenance of the nuclear power plants, and by evaluating the plant performance and incident trends. As a result of these evaluations, a nuclear component reliability data system has been developed for estimating failure rate of major components for use in PSA. Internet-Web client at the utilities can access this data system. The users can select component, plant system and time period, and then compute the failure rate by the data system.

Data of component failure are then continuously being collected from utilities and transferred into the data system for the estimation of component reliability within Japan. A set of domestic component reliability data on 49 Japanese LWRs from April 1, 1982 to March 31, 1997 was reported [10].

CONCLUSION

A philosophy of review and reassessment of the seismic safety performance of Japanese NPPs using seismic PSA was discussed herein. This philosophy is to be reflected in subsequent regulations.

Reassessment is important for ensuring the seismic safety of NPPs that are designed, constructed, operated and maintained in such a way that their impact on public health and safety is as low as reasonably achievable. Such reassessment must consider not only the dynamic interaction of earthquake hazard and reactor system but also consideration of dynamic interaction among human, technology, and environment.

Conventional seismic design by deterministic method is not appropriate for thorough, quantitative determination of safety margins of NPPs when considering the uncertainties in the potential high hazard system.

A probabilistic approach to seismic safety assessment is important from the point of view of “How safe is safe enough.” Such an approach takes into account the ground motion from the full range of earthquake magnitudes, allowing explanation of the relationship between reactor safety performance and the strengths of structures, equipment, and components considering the uncertainties within the seismic hazard and the safety performance system.

Reassessment should be conducted for two phases (detail design and construction phase and the operation phase) according to the progress of the NPP. In the plant design and construction phase, the seismic PSA is performed based on the detailed design and construction data. In the plant operation phase, the seismic PSA is performed based on the plant walk-down inspection during plant operation.

Usually seismic PSA focuses on evaluation of core damage frequency. However, safety management concerns not only prevention of severe accidents but also concerns shutdown management, on-line maintenance, in-service inspection, and in-service testing. Risk-informed safety management should be developed based on defense-in-depth philosophy one of the ways to strengthen public confidence in the regulatory system.

Safety management structures a software barrier system of NPPs. Multiple barriers both of hardware systems and software systems are useful ensuring nuclear safety assurance. The effectiveness of hardware systems can be evaluated in a design phase but the thorough evaluation of the effectiveness of software systems need to be performed in operation phase.

To evaluate the safety management, a performance-based approach is necessary for regulatory body to define the effectiveness and efficiency. A comprehensive set of Performance Indicators for regulatory should be developed.

Development activities of seismic PSA methodology is considerably on going in Japan to be utilize in a risk-informed safety management not only concern prevention of severe accident but also concerns shutdown management, on-line maintenance, in-service inspection, and in-service testing.

DISCLAIMER

The views expressed in this paper are those of author and should not be construed to reflect the official Japanese NSC position.

REFERENCES

1. Irikura, K., "Prediction of Strong Motion From Future Earthquakes Caused by Active Faults-Case of the Osaka Basin," Proc. Of the 12WCEE, Auckland, 2000

2. Vuuren, van W., "Organizational Failure: Lessons from Industry Applied in the Medical Domain," Safety Science, 33, 13-29, 1999

3. Rasmussen, J., Pejtersen, A.M., Goodstein, L.P., "Cognitive Systems Engineering," Jhon Wiley & Sons, Inc. 1994

4. Tanabe, F., "Issues in Man-Machine System Evaluation," SMiRT 11th Post-Conference Seminar on Probabilistic Safety Assessment Methodology, August 1991

5. Sakagami, M. and Hirano, M., "Comparison of Trial Seismic PSA and Seismic Margin Analysis for a BWR" Proc. of PSAM5, Osaka, 2000

6. Miyata, K., Zama, T., Yoneyama, M., Visweswaran, S., Fujii, M. and Hisamochi, K., "Seismic Margin Analysis for Japanese ABWR Plant" Proc. of PSAM5, Osaka, 2000

7. Oikawa, T., Muramatsu, K., Kasahara, T., Kawamata, K. and Morota, H., "Conceptual Design of Operator Support System under Seismic Conditions" Proc. Of PSAM5, Osaka, 2000

8. Kansai Electric Power Company, Restoration Record in the Southern Hyogo prefecture earthquake, June 1995 (in Japanese)

9. Nishihata, M., Shibuya, A. Sakagami, M. and Mizumachi, W., "Evaluation of Component Fragility for Seismic PSA on Japanese LWRs," Proc. Of PSAM5, Osaka, 2000

10. Kirimoto, Y., Emukai, F. and Sasaki, A., "Component Reliability Data from 1982 to 1997 on 49 Japanese LWRs Accumulated in the Nuclear Component Reliability Data System" Proc. of PSAM5, Osaka, 2000

METHODOLOGY AND TYPICAL ACCEPTANCE CRITERIA

FOR SEISMIC RE-EVALUATION OF VVER-TYPE EQUIPMENT COMPONENTS AND DISTRIBUTION SYSTEMS

Rudolf MASOPUST

STEVENSON AND ASSOCIATES, Office in Czech Republic

1 INTRODUCTION

The purpose of this paper is to describe the methodology and typical acceptance criteria for seismic re-evaluation of equipment components and distribution systems of operating VVER-type NPPs. The plants of this type are located and operated in several European countries (Czech and Slovak Republics, Hungary, Bulgaria, Ukraine, Russia).

The seismic re-evaluation of structures, systems and equipment components of VVER-type NPPs is affected by the following principal conditions:

(a) the seismic hazard was significantly underestimated in the original design of these NPPs,
(b) almost no national standards, codes and other similar documents exist for these purposes,
(c) there has been and still is a significant and valuable attention of IAEA to seismic re-evaluation of these NPPs.

The paper does not deal with procedures and rules used to select essential systems and equipment components to be evaluated for seismic effects. Also procedures to determine the proper seismic input (in-structure seismic response spectra and corresponding seismic anchor movements) are not described herein.

2 MAIN STANDARDS, CODES AND GUIDES

(a) IAEA guides

- IAEA Safety Series 50-SG-D15 “Seismic Design and Qualification for Nuclear Power Plants”, IAEA, Vienna, 1992 [1],
- Technical Guidelines prepared by IAEA for individual NPPs (as a rule upon the request of the corresponding National Regulatory Authority),

(b) Internationally well-recognized standards, codes and guide documents

- ASCE 4-86 “Seismic Analysis of Safety-Related Nuclear Structures. ASCE, 1986 [2],
- ASCE 4-98 “Seismic Analysis of Safety-Related Nuclear Structures. ASCE, 1998 [3],
- ASME BPVC Section III, Division I, Subsections NCA, NB, ND, NF and Appendixes, Edition 1992 [4],

- ASME QME-1 “Qualification of Active Mechanical Equipment Used in Nuclear Power Plants. 1994 [5],
- IEC 980-89 “Recommended Practices for Seismic Qualification of Electrical Equipment of the Safety System
  or Nuclear Generating Stations. IEC, 1989 [6],
- A Methodology for Assessment of Nuclear Power Plants Seismic Margin. Report NP-6041, Revision 1.
  EPRI, Palo Alto, 1991 [7],
- Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Power Plant Equipment Revision 2A. SQUG, 1992 [8],
- Use of Seismic Experience and Test Data to Show Ruggedness of Equipment in Nuclear Power Plants. Revision 4.0. Prepared by SSRAP for SQUG, 1992 [9],
- Seismic Evaluation Procedure for Equipment in U.S. Department of Energy Facilities. DOE RpT 0545. U.S. Department of Energy, March 997 [10],

(c) Other related documents

- Criteria for Seismic Evaluation and Potential Design Fixes for VVER Type Nuclear Power Plants. Prepared for IAEA by J.D. Stevenson, 1996 [11],
- Seismic Verification of Mechanical and Electrical Equipment Components Installed on WWER-Type Nuclear Power Plants by the GIP-VVER Procedure. Document rem10-00.iae. Prepared by S&A-CZ for IAEA, 2000 [12],
- Anchorage of Equipment – Requirements and Verification Methods with Emphasis on Equipment of Existing and Constructed VVER-Type Nuclear Power Plants [13].

3 USED WORK APPROACHES

Generally, the seismic re-evaluation of an each equipment component or an each distribution system consists of the following three main steps:

- evaluation of the seismic margin capacity of equipment components and distribution systems as built,
- assignment of the relevant seismic upgrading measures, if necessary,
- evaluation of the seismic margin capacity of upgraded equipment components and distribution systems.

<u>The Seismic Margin Assessment (SMA) is used to determine the High Confidence Low Probability of Failure (HCLPF) seismic margin capacity of components and systems (as built and also when upgraded) in combination with the modified Generic Implementation Procedure (GIP) called as GIP-VVER.</u>

<u>It is, of course, assumed, that the evaluated equipment components and distribution systems were properly designed against all non-seismic loads and effects.</u>

4 METHODS AND TYPICAL ACCEPTANCE CRITERIA USED FOR SEISMIC RE-EVALUATION OF VVER-TYPE EQUPMENT COMPONENTS AND DISTRIBUTION SYSTEMS

Table 1Methods and Typical Acceptance Criteria Used for Seismic Re-evaluation of VVER-Type Equipment
Components and Distribution Systems

EQUIPMENT	***Seismic Resistance Elements***	***Evaluation Method***	***General Acceptance Criteria***
Reactor	strength of reactor internal parts	HCLPF calculation	ASME BPVC Section III (ed. 92) NB and/or NG, Appendix F, Service Level D, ductility - Table 4
	insertion of control rods	testing	insertion time should be within the prescribed limits
	strength of upper structure	HCLPF calculation	ASME BPVC Section III (ed. 92) NB and/or NG, Appendix F, Service Level D, ductility - Table 4
	strength and stability of rector vessel supports	HCLPF calculation	ASME BPVC Section III (ed. 92) NB and/or NG, Appendix F, Service Level D, ductility - Table 4
	strength of reactor nozzles	HCLPF calculation	ASME BPVC Section III (ed. 92) NB and/or NG, Appendix F, Service Level D, ductility – Table 4
Primary circuit components, pressurizer	strength of component body	HCLPF calculation	ASME BPVC Section III (ed. 92) NB and/or NC, Appendix F, Service Level D, ductility - Table 4
	strength of supports	HCLPF calculation	ASME BPVC Section III (ed. 92) NF, Appendix F, Service Level D, ductility - Table 4
	strength of anchorage	HCLPF calculation	specific approach, capacity limits in GIP-VVER
	strength of essential nozzles	HCLPF calculation	ASME BPVC Section III (ed. 92) NB and/or NC, Appendix F, Service Level D, ductility - Table 4
	seismic interactions	walkdown only	GIP-VVER

Table 1 Continued

EQUIPMENT	***Seismic Resistance Elements***	***Evaluation Method***	***General Acceptance Criteria***
Active mechanical components	functionality	tests, GIP-VVER	ASME QME-1 GIP-VVER
	strength of component body including internal parts	HCLPF calculations	ASME BPVC Section III (ed. 92) NC and/or ND, Appendix F, Service Level D, ductility - Table 4
	strength and proper function of supports	HCLPF calculation + walkdown	ASME BPVC Section III (ed. 92) NF, Appendix F, Service Level D, ductility - Table 4
	anchorage of supports	HCLPF calculation	specific approach, capacity limits in GIP-VVER
	strength of essential nozzles	HCLPF calculation	ASME BPVC Section III (ed. 92) NC and/or ND, Appendix F, Service Level D, ductility - Table 4
	seismic interactions	walkdown only	GIP-VVER
Passive mechanical components	strength of component body including internal parts	HCLPF calculation	ASME BPVC Section III (ed. 92) NC and/or ND, Appendix F, Service Level D, ductility - Table 4
	strength and proper function of supports	HCLPF calculation + walkdown	ASME BPVC Section III (ed. 92) NF, Appendix F, Service Level D, ductility - Table 4
	anchorage of supports	HCLPF calculation	specific approach, capacity limits in GIP-VVER
	strength of essential nozzles	HCLPF calculation	ASME BPVC Section III (ed. 92) NC and/or ND, Appendix F, Service Level D, ductility - Table 4
	seismic interactions	walkdown only	GIP-VVER
Hot and large bore pipes	strength of pipe runs, elbows and T-elements	HCLPF calculations	ASME BPVC Section III (ed. 92) NC and/or ND, Appendix F, Service Level D, ductility - Table 4
	strength and proper function of pipe supports	HCLPF calculation + walkdown	ASME BPVC Section III (ed. 92) NF, Appendix F, Service Level D, ductility - Table 4
	anchorage of supports	HCLPF calculation	specific approach, capacity limits in GIP-VVER
	seismic interactions	walkdown only	GIP-VVER
Small bore pipes, HVAC ducts	strength of pipe runs, elbows and T-element	GIP-VVER	GIP-VVER

Table 1 Continued

EQUIPMENT	***Seismic Resistance Elements***	***Evaluation Method***	***General Acceptance Criteria***
Buried pipes	strength of pipe elements subjected to seismic wave effects	HCLPF calculation	ASCE 4-86 (98), ASME BPVC Section III (ed. 92), NC, ND, Appendix F, Service Level D, ductility - Table 4
Electrical equipment components	functionality	tests, GIP-VVER	IEC 980, GIP-VVER
	strength of housing structure and supports	HCLPF calculation	stress limits according to national standards for design of steel structures
	anchorage	HCLPF calculation	specific approach, capacity limits in GIP-VVER
	seismic interactions	walkdown only	GIP-VVER
HVAC equipment components	functionality	tests, GIP-VVER	IEC 980, GIP-VVER
	strength of the component body and its supports	HCLPF calculation	ASME BPVC Section III (ed. 92) NC and/or ND, NF, Appendix F, Service Level D, ductility – Table 4
	anchorage of supports	HCLPF calculation	specific approach, capacity limits in GIP-VVER
	strength of essential nozzles	HCLPF calculation	ASME BPVC Section III (ed. 92) NC and/or ND, Appendix F, Service Level D, ductility - Table 4
	seismic interactions	walkdown only	GIP-VVER
I&C equipment components	functionality	tests, GIP-VVER	IEC 980, GIP-VVER
	strength of housing structure and supports	HCLPF calculation	stress limits according to national standards for design of steel structures
	anchorage	HCLPF calculation	specific approach, capacity limits in GIP-VVER
	seismic interactions	walkdown only	GIP-VVER
Essential relays and other sensitive instruments	functionality	tests	IEC 980-89, IEC 255-21-3

Table 1 Continued

EQUIPMENT	***Seismic Resistance Elements***	***Evaluation Method***	***General Acceptance Criteria***
	attachment to cabinet	walkdown only	GIP-VVER
Cable supporting structures and tubings	strength of structure elements	HCLPF calculation	specific approach, stress limits from national standards for design of steel structures, ductility factors according to Table 4
	anchorage	HCLPF calculation	specific approach, capacity limits in GIP-VVER
Cable penetrations	general resistance	walkdown only	GIP-VVER
Cranes, containers, supporting platforms	seismic stability	HCLPF calculation (simplified)	safety at least 1.50 against overturning and at least 1.20 against sliding (using conservative friction coefficients)

Table 2Seismic Load Combinations and Load Factors

Equipment	***Seismic Load Combinations and Load Factors***
Passive and active equipment components	1.0 D + 1.0 L + 1.0 P + 1.0 RLE
Equipment nozzles	1.0 D + 1.0 L + 1.0 T + 1.0 P + 1.0 RLE
Equipment supports and anchorage	1.0 D + 1.0 L + 1.0 T + 1.0 RLE
Pipes	1.0 D + 1.0 L + 1.0 P + 1.0 RLE
Pipe supports and their anchorage	1.0 D + 1.0 L + 1.0 T + 1.0 RLE
Cable structures, supporting platforms etc.	1.0 D + 1.0 L + 1.0 T + 1.0 RLE

Notes: 1) D = dead load under normal operating conditions (NOC)
L = live load under NOC, if any
T = loads due to restrained temperature expansion under NOC, if any
P = internal pressure under NOC, if any
RLE = seismic loads due to Review Level of Earthquake (RLE) (no less than Safe Shutdown Earthquake)
I. Inertia effects of RLE should be combined with the corresponding seismic anchor movements, when important and must be considered, using the SRSS rule.

The following methods are used to calculate seismic responses:

- response spectrum method, using rules for combining modal and spatial responses in accordance with ASCE 4-86(98) or ASME BPVC Section III (ed. 92) Appendix N requirements,
- equivalent static method, using rules for combining spatial responses in accordance with ASCE 4-86 (98) requirements,
- linear and non-linear time-history method, using rules for combining spatial responses in accordance with ASCE 4-86 (98) requirements.

Tab. 3 Acceptable Damping Values

Equipment	***Acceptable Damping Values***		
	for HCLPF calculations	***for design of seismic upgrades*** *(see note 1)*	
		stress level 1	***stress level 2***
bolted supporting structures	7 %	4 %	7%
welded supporting structures	5%	2%	5%
pipes (all parameters and all diameters)	5%	2%	5%
anchored mechanical components	5%	3%	5%
electrical and I&C cabinets and panels	5%	3%	5%
cable supporting structures (note 2)	5 – 10 – 15%	2 – 6 – 10%	5 – 10 – 15%
tanks -- impulsive mode -- convective mode	 5% 0.5%	 3% 0.5%	 5% 0.5%

Notes: 1) Stress levels 1 and 2 means about 50% and 100% of the bearing capacity respectively.
I. Use these three values for structures loaded by cables up to 10, 50 and 100% of their nominal capacity respectively.

Tab. 4 Acceptable Ductility Factors (to be used in HCLPF elastic calculations only)

Equipment	***Ductility Factor***
- equipment components that must remain functional	1.00
- equipment components with brittle failure modes	1.00
- adequately anchored passive components with welded connections	1.50
- welded pipelines (basic pipe material and welded connections)	1.50
- welded nozzles	1.25
- flange nozzles and flange pipe connections	1.00
- threaded pipes	1.00
- components made of cast iron	1.00
- pipe and component supports and their anchorage (brittle failure modes)	1.00
- pipe and component supports and their anchorage (ductile failure modes)	1.50 [1)]
- steel columns of platforms with predominant bending failure modes	1.50
- steel columns of platforms with predominant shear or compression failure modes	1.00
- steel beams of platforms with predominant bending failure modes	2.00
- steel beams of platforms with predominant shear or tension failure modes	1.25
- ductile connections of steel platforms	1.25
- brittle connections of steel platforms	1.00

Notes: 1) This value should be used carefully only for really ductile failure modes.
2) The elastic seismic capacity expressed as a certain multiple of PGA (RLE) should be multiplied by these
factors to obtain the HCLPF value.

5 GIP-VVER PROCEDURE

The purpose of this section is to briefly describe the modified GIP titled as GIP-VVER which was created and can be used to verify seismic adequacy of the most important classes of mechanical and electrical equipment components and also distribution systems of operating or constructed VVER-type NPPs, namely VVER-440/213 and 1000 type NPPs.

The procedure GIP-VVER has been prepared using the following background:

- public available information contained in SSRAP, GIP, U.S. DOE, LLNL and MCEER documents [8,9,10,15,16],
  information extracted from the documents prepared in a frame of the IAEA Benchmark Study for the Seismic Analysis and Testing of WWER-Type Nuclear Power Plants [17,18,19,20,21],
- information extracted form the results of available seismic tests performed mostly in Czech Republic during the last about 15 years and collected systematically and studied by S&A-CZ,
- experience taken from various many seismic walkdowns, evaluations and analyses of VVER-type NPPs equipment performed by S&A-CZ during the last eight years for these NPPs located in Czech, Slovakia and Hungary,
- information extracted from other related papers and documents.

The scope of equipment covered by the current version of the GIP-VVER procedure includes, similarly as the original GIP, the following twenty classes of mechanical and electrical equipment:

(1) Motor Control Centers,
(2) Low Voltage Switchgears,
(3) Medium Voltage Switchgears,
(4) Transformers,
(5) Horizontal Pumps,
(6) Vertical Pumps,
(7) Fluid-Operated Valves,
(8) Motor and Solenoid –Operated Valves,
(9) Fans (ventilators),
(10) Air Handlers
(11) Chillers,
(12) Air Compressors,
(13) Motor Generators,
(14) Engine Generators,
(15) Distribution Panels,
(16) Batteries on Racks,
(17) Battery Chargers and Inverters,
(18) Instruments on Racks,
(19) Temperature Sensors,
(20) I&C Panels and Cabinets.

European and particularly <u>VVER-type relays, switches, transmitters and electric penetrations are significantly different from those included into the original GIP databases. These two classes of equipment are not included into the GIP-VVER procedure and their seismic verification shall be based on testing.</u>

In addition to twenty classes listed above, the GIP-VVER procedure also includes guidelines for simplified analytical seismic evaluation of the following classes of equipment:

(23) Cable Supporting Structures (based mainly on the EPRI methodology [23]),
(24) Tanks, Heat Exchanger, Filters (based mainly on the documents [24]),
(25) Pipelines and HVAC Ducts (based on the public available documents [22,25]).

GIP-VVER also includes two special guidelines to verify adequacy of anchorage [13] and seismic adequacy of non-bearing masonry walls.

GIP-VVER is primarily a screening and walkdown procedure. However, if an equipment item is classified as an outlier, rigorous approaches as testing on shaking table, deep study of input data, sophisticated analysis etc. may be used to verify its seismic adequacy. Generally, four major steps of this GIP-VVER procedure when applied to twenty main classes of equipment 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 the system engineers, plant operation personnel, experienced and professionally trained seismic capacity engineers, and also personnel to identify and evaluate essential relays (if necessary).

The criteria to verify seismic adequacy of an equipment item during the screening walkdown are (see also Figure 2):

- seismic capacity greater than seismic demand (by comparison of the corresponding $ISRS_{RLE(SL2, SSE)}$ or $GRS_{RLE(SL2, SSE)}$ to the Bounding Spectrum (Figure 1, Table 5),
- similarity to the equipment in the seismic experience data bases (checking of caveats, based on walkdowns and information extracted 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).

The GIP-VVER procedure uses two bounding spectra (BS):

(a) BS attached to PGA = 0.33 g (the same as introduced by SSRAP and used by GIP),

(b) BS attached to PGA = 0.50 g (1.5 times SSRAP BS) for selected VVER equipment classes, which are evidently
robust and rugged.

The following sheets are used for seismic verification and walkdowns:

- Screening Verification Data Sheet (SVDS)
- Seismic Evaluation Work Sheet (SEWS)
- Seismic Walkdown Sheet (SWS)
- Outlier Seismic Verification Sheet (OSVS)

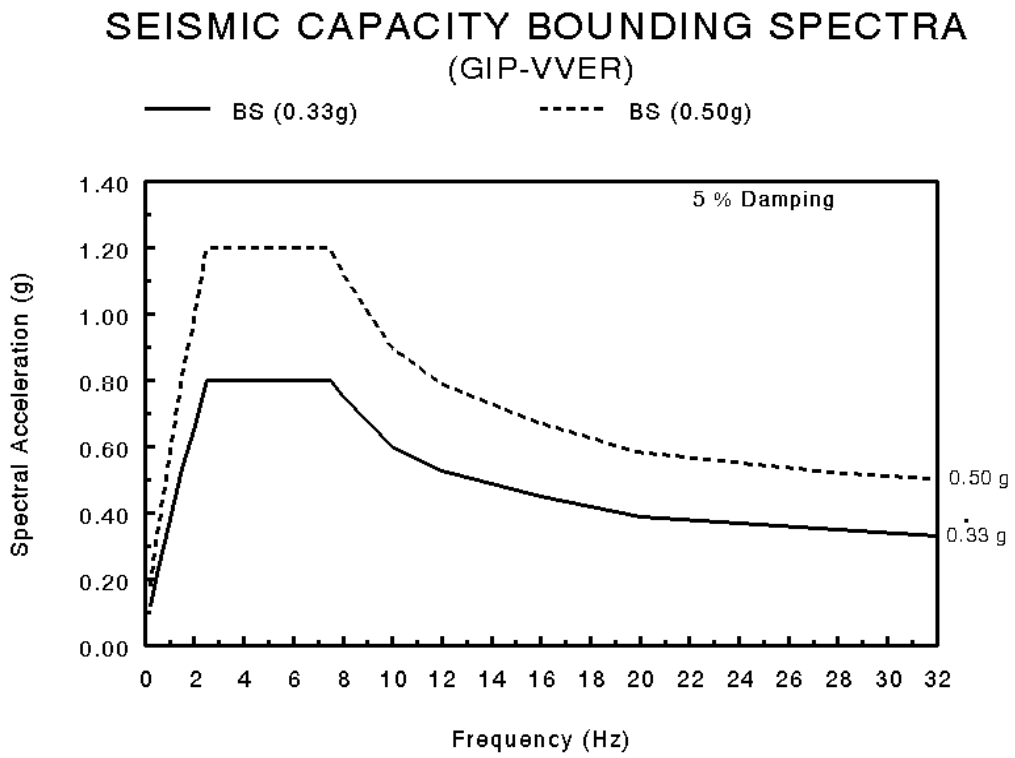


Figure 1 GIP-VVER Seismic Capacity Bounding Spectra

Figure 2 Screening Verification and Walkdown Procedure GIP-VVER

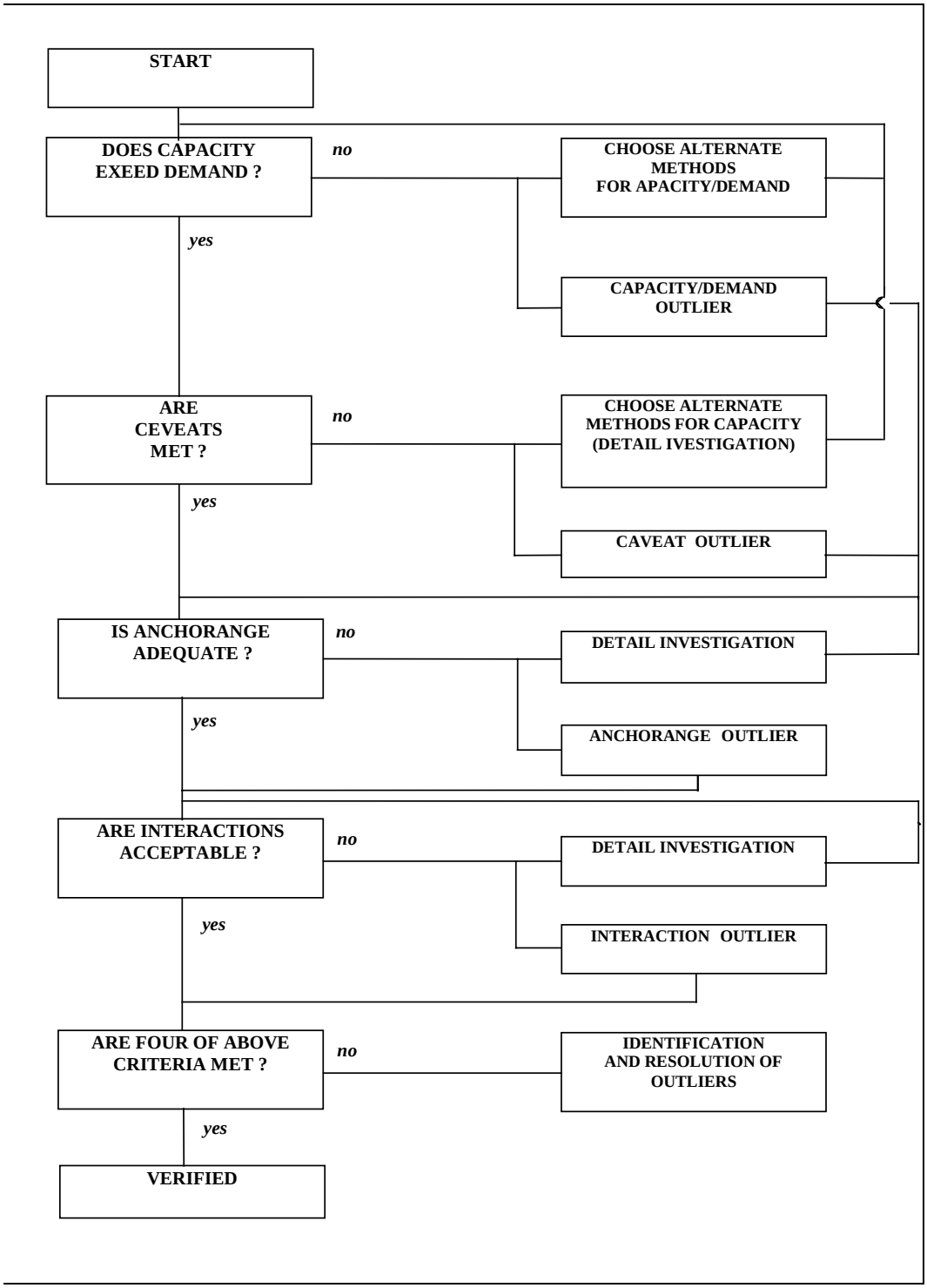


Table 5Criteria of Comparison Seismic Capacity to Seismic Demand [1)]

A. Comparison with RLE (SL2, SSE) Ground Response Spectra (GRS) [2)]
This can be used when the equipment item is mounted below about 12 m above the effective grade and when the natural frequency of equipment is greater than 12 Hz [3)] **BS ≥ $GRS_{RLE\ (SL2,SSE)}$ (5% damping)** [4)]
B. Comparison with RLE (SL2, SSE) In-Structure Response Spectra (ISRS)
1.5 x BS ≥ realistic (median, mean, best estimated) $ISRS_{RLE\ (SL2,SSE)}$ (5% damping) [4)]

Notes: 1) Apply at least one of these two rules, which applicable.

I. The criterion A can be used only with the well rigid building structures as the lower concrete parts of the
VVER-440 reactor building, the VVER-1000 reactor building, the VVER–1000 diesel-generator buildings
etc. Do not use this criterion with such flexible building structures as the longitudinal and transversal
galleries of the VVER–440 reactor buildings, the VVER-440 and also VVER-1000 auxiliary buildings, the
VVER-440 diesel-generator buildings etc.

II. Do not apply the 12 Hz limit for equipment mounted on piping systems (valves, valve operators etc.).

4) These criteria shall be met for all three orthogonal spatial directions.

The seismic capacity needs only to envelop the seismic demand spectrum for frequencies at and above the conservatively estimated lowest natural frequency of the equipment item to be evaluated. Also narrow peaks in the seismic demand spectrum may exceed the seismic capacity spectrum under the conditions specified in the corresponding user manual. It should be also noted that it is permissible to use seismic demand spectra without broadening for this comparison, however when doing it, uncertainty in the natural frequency of the building structure should be taken into account by corresponding shifting of the seismic demand spectrum at these peaks.

6 SIMILARITY OF VVER-TYPE EQUIPMENT TO EQUIPMENT INCLUDED IN THE SQUG DATABASES - PRINCIPLES

Similarity of VVER-type equipment to equipment included in the SQUG databases [9] is the most important keystone of practical application of the GIP-VVER procedure. Generally, the principal of similarity is based upon comparison of equipment dynamic and physical characteristics. The procedure to establish similarity within an each equipment class includes the following comparisons:

- most probable modes of malfunction (based on recognized behavior of all critical devices) ,
- predominant resonant and critical frequencies and mode shapes,
- critical damping,
- most important physical equipment characteristics

-- *equipment size, mass and position (vertical, horizontal, inclined etc.),*
-- *general making, quality of making, age of equipment,*
-- *location of the center of gravity, presence and location of cantilevered parts,*
-- *implementation of heavy and / or moving internal parts,*
-- *implementation of supports and anchorage,*
-- *implementation of attached lines, substructures, devices etc.*
-- *presence of devices (mechanical or electrical) sensitive to vibrations and shocks.*

7 SEISMIC INTERACTIONS

The four seismic interaction effects that are considered are:

- proximity (impacts of adjacent equipment or structures on safety-related equipment due to their relative motion during an earthquake),
- structural failure and falling of overhead or adjacent structures, systems, or equipment components),
- flexibility of attached lines and cables,
- flooding due to earthquake induced failures of tanks or vessels.

Interaction examples typical for VVER-type NPPs are as follows:

- unreinforced masonry walls adjacent to safety-related equipment may fall and impact safety-related equipment or cause loss of support of such equipment,
- fire extinguishers may fall and impact or roll into safety-related equipment, inadequately anchored or braced equipment as vessels, tanks, heat exchangers, cabinets etc. may overturn, slide and impact adjacent safety-related equipment,
- equipment carts, chains, air bottles, welding equipment etc. may roll into, slide, overturn, or otherwise impact safety-related equipment,
- storage cabinets, office cabinets, files, bookcases etc. located, for instance in control rooms, may fall and impact adjacent safety-related equipment,
- too flexible piping, cable trays, conduits, and HVAC ducts may deflect and impact adjacent safety-related equipment,
- anchor movement may cause breaks in nearby piping, cable trays, conduits, HVAC ducts etc. that may fall or deflect and impact adjacent safety-related equipment,
- emergency lights and lower ceiling panels can fall down and damage safety-related equipment,
- free crane hooks may bang the safety-related equipment in their vicinity.

8 Conclusion

The methodology and acceptance criteria described above have been used and still are used by S&A-CZ for seismic re-evaluation of equipment components and distribution systems installed on operating VVER-Type NPPs in several countries (Czech and Slovak Republics, Hungary). 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.

References

[1] *IAEA Safety Guide 50-SG-D15 Seismic Design and Qualification for Nuclear Power Plants, IAEA, Vienna, 1992.*

[2] *ASCE 4-86 Seismic Analysis of Safety-Related Nuclear Structures. ASCE, New York, 1986.*

[3] *ASCE 4-98 Seismic Analysis of Safety-Related Nuclear Structures. ASCE, New York, 1998.*

[4] *ASME BPVC Section III, Division 1, Subsections NCA, NB, NC, ND, NF and Appendices. ASME, 1992 Edition.*

[5] *ASME QME-1 Qualification of Active Mechanical Equipment Used in Nuclear Power Plants, ASME, 1994.*

[6] *IEC 980-89 Recommended Practices for Seismic Qualification of Electrical Equipment of the Safety System for Nuclear Generating Stations. First Edition. IEC, 1989.*

[7] *A Methodology for Assessment of Nuclear Power Plant Seismic Margin. Report NP-6041, Revision 1. EPRI, Palo Alto, 1991.*

[8] *Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Power Plant Equipment, Revision 2A. SQUG, 1992.*

[9] *Use of Seismic Experience and Test Data to Show Ruggedness of Equipment in Nuclear Power Plants. Revision 4.0. Prepared by SSRAP for SQUG, 1992.*

[10] *Seismic Evaluation Procedure for Equipment in U.S. Department of Energy Facilities. DOE RpT 0545. U.S. Department of Energy, March 1997.*

[11] *Criteria for Seismic Evaluation and Potential Design Fixes for VVER Type Nuclear Power Prepared for IAEA by J. D. Stevenson. Cleveland, 1996.*

[12] *Seismic Verification of Mechanical and Electrical Components Installed on WWER-Type Nuclear Power Plants by the GIP-VVER Procedure. Report No. rem10-00.iae, Revision 0. Prepared by S&A-CZ for the IAEA.*

[13] *Masopust, R., Anchorage of Equipment – Requirements and Verification Methods with Emphasis on Equipment of Existing and Constructed VVER-Type Nuclear Power Plants. In: Bezpe•nost jaderné energie, No. 5/6, pp. 146–158, 1999.*

[14] *IEC 255-21-3 Electrical Relays. Vibration, Shock, Bump and Seismic Tests on Measuring Relays and Protection Equipment. Seismic Tests. First Edition. IEC, 1993.*

[15] *Practical Equipment Seismic Upgrade and Strengthening Guidelines. Report UCRL-15815. Lawrence Livermore National Laboratory, Livermore, California, 1986.*

[16] *Seismic Reliability Assessment of Critical Facilities: A Handbook, Supporting Documentation, and Model Code Provisions. Technical Report MCEER-99-0008. MCEER, Buffalo, 1999.*

[17] *Coman, and Stevenson, J.D., Experience Database of Romanian Facilities Subjected to the Last three Vrancea Earthquakes. Working Material. Benchmark Study for the Seismic Analysis and Testing of WWER-Type Nuclear Power Plants. Volume 5A. IAEA, Vienna, 1996.*

[18] *Campbell, R.D., The December 7, 1988 Armenia Earthquake Effects on Selected Power, Industrial and Commercial Facilities. Working Material. Benchmark Study for the Seismic Analysis and Testing of WWER-Type Nuclear Power Plants. Volume 5B. IAEA, Vienna, 1996.*

[19] *Jurukowski, D., Seismic Functional Qualification of Active Mechanical and Electrical Component Based on Shaking Table Testing. Working Material. Benchmark Study for the Seismic Analysis and Testing of WWER-Type Nuclear Power Plants. Volume 5B. IAEA, Vienna, 1996.*

[20] *Summary of Earthquake Experience Database. EQE International. Working Material. Benchmark Study for the Seismic Analysis and Testing of WWER-Type Nuclear Power Plants. Volume 5C. IAEA, Vienna, 1999.*

[21] *Experience Database of Romanian Facilities Subjected to the Last Three Vrancea Earthquakes - Final Report. Stevenson and Associates (three papers). Working Material. Benchmark Study for the Seismic Analysis and Testing of WWER-Type Nuclear Power Plants. Volume 5B. IAEA, Vienna, 1996.*

[22] *Eder, S.J. et al., Walkthrough Screening Evaluation Field Guide. Report UCRL-ID-115714, Revision 2. Lawrence Livermore National Laboratory, Livermore, California, 1993.*

[23] *Cable Tray and Conduit System Seismic Evaluation Guidelines. Prepared by EQE Engineering. Report NP-7151-D. EPRI, Palo Alto, 1991.*

[24] *Bandyopadhyay, K. et al., Seismic Design and Evaluation Guidelines for the Department of Energy High-Level Waste Storage Tanks and Appurtenances. Report No. BNL 52 361 (Rev. 10/95). Brookhaven National Laboratory, Upcon, New York, 1995.*

[25] *Procedure for the Seismic Evaluation of Piping Systems Using Screening Criteria. Prepared by G.A. Antaki et al. for the US Department of Energy. Report WSRC-TR-94-0343, Revision 1. Westinghouse Savannah River Company. June 1995.*

[26] *Valves for Equipment and Pipes of NPPs. General Technical Requirements OTT-87). Issued by Gosatomenergonadzor and also Interatomenergo, Moscow, 1987.*

[27] *O' Rourke, M.J. and Liu, X., Response of Buried Pipelines Subject to Earthquake Effects. MCEER, University at Buffalo, 1999.*

SESSION 2:
COUNTERMEASURES/STRENGTHENING
Chairman: Dr. T. Katona - Paks NPP (HU)

Methods and practice of seismic revaluation for nuclear power plant structures

V.Beliaev. Research Center of Capital Construction, St. Petersburg, Russia

(For technical reasons, the first 6 figures at the end of this paper could not be reproduced)

ABSTRACT

Nowadays Russia's specialists perform a huge amount of works to revaluate the NPP safety. These works are certain to include refinement of NPP safety assessment under the effects of specific dynamic loads, earthquake effects included. It should be noted, that a number of Russian NPPs now in operation had been designed either with no account of these loads, or under the requirements which are underestimated as compared with the modern requirements on the external load composition and rate. Revaluation of NPP seismic safety is based on the results of the works taken under orderly sequence on assessment of (1) seismic input and ground effects; (2) structure response and state; (3) equipment and pipelines response and state.

Therewith we assume that ground motion predicted behavior at the structure basement has been preset for the SSE and OBE conditions and the effects of soil-structure interaction, including the possible soft soil liquefaction. The need to determine both the reaction of a construction and its state as a whole as well as its elements reaction, to evaluate their bearing capacity and failure zones formation makes it necessary to develop a detailed (usually finite-element) structural model. Since seismic revaluation is to be performed for the existing structures, characteristics of which can substantially differ from the design ones, revealing the actual state of this structures becomes critical. If the real values of physical and mechanical properties of the structure materials, connections of elements etc. are used as initial data in a structural model this permits to increase the design assessment credibility and reliability. Comparison of the results of seismic structural analysis with the design and real characteristics of material and the state of connections in several cases illustrates a pronounced effect of wear on their reaction to a seismic input. It is precisely these data that are to be taken as the initial data for seismic revaluation of equipment and pipelines reaction and state.

INTRODUCTION

The increasing seismicity level in several areas of Russia makes it necessary to solve the following problems for existing NPP:

- to perform special works on seismic strengthening of NPP buildings and equipment in order to bring their seismic stability to the level required;
- to carry out design evaluation of the possibility of NPP safety provision in case of considerable damages of buildings and structures.

Solving the problem of seismic strengthening of buildings and structures has been the objective of much research, all of this was reflected in standard material and procedure specifications, in construction and renewal procedures. By this is meant that most of existing types and structural configurations of buildings have practically testified designs for seismic strengthening. However, real financial limitations give no way to do our utmost in the shortest possible time. Therefore an in-site safety assurance evaluation in case of considerable damages of buildings, first, will allow to receive a realistic prognosis of earthquake consequences, second, it will serve as the basis for the feasibility study for carrying out the works on the new requirements on seismic stability of buildings and structures. Thus, design evaluation can be performed independently or in parallel with operations on seismic strengthening.
Analysis of the results of seismic revaluation of existing NPP buildings and structures permits to state that standard designs, which are used in engineering practice, seem to be insufficient to evaluate expected damage degree of building supporting elements under the design basis earthquakes. More often than not it is required to design on the basis of real (or synthesized) records of ground motions and complicated 3D models of buildings to judge reasonably the possibilities of a required safety level provision. Furthermore the major factor of credibility of the design evaluation received is the quality and completeness of the data on examination of the existing buildings actual state, which are to be taken into account when designing. Thus, building foundation differential settlement causes considerable stresses in the walls, these stresses (even if the monolithic nature of masonry is preserved under the normal operation conditions) at earthquakes will result in considerable supporting element damages. Furthermore, the background seismicity and other impacts (transport vibrations, industrial explosions etc.) create a damage accumulation effect in supporting elements and change the real ability of buildings to resist a seismic input.

1. Application of tool examinations to evaluate the real state of NPP building structures

Nowadays the most commonly used nondestructive methods of determination of physical and mechanical characteristics of building materials and structures (mechanical ones: rebound, plastic deformation, chip, explosion, etc, ultrasonic characteristics) have substantial drawbacks: they require an access to the structure surface under testing and permit to determine the required material characteristics at shallow depths (usually up to several mm). The exception is provided by a through ultrasonic testing, but it could be performed only given the access to two building structure surfaces, which is not always possible. When inspecting the building structures which are under operation these methods are frequently hindered or inefficient for the following reasons:

- the structure surface may have coverings (for example, plaster or metal isolation) and their removal and further recovery involve huge expenditures;
- the shallow layer characteristics of the material structure may differ substantially from the characteristics of the body of masonry as a result of carbonization, weathering, frost action, effect of high temperature or corrosive media etc;
- the building could be a multilayer structure.

To examine such structures we need special methods, enabling both to determine the characteristics of structural materials at great depths, furthermore, it is desirable to be on a large (several meters) base and without coverings removing, as well as to assess the conditions of connecting the structural components among themselves.
We had developed a combined method of evaluating the mechanical characteristics of materials, which are used for various structural elements, and the nature of their fixing (types of connections), the method is based on vibrations and wave dynamics theory. The evaluation procedure consists of two parts:

- the procedure of integrated evaluation of state and pattern of fixing the separate elements, structures or buildings as a whole on the basis of analysis of natural oscillation, induced by pulse shock load;
- the procedure of determination of acoustic characteristics of structural material at the area under testing and on the basis of these results the procedure of determination of elastic characteristics (shear modulus and elastic modulus), as well as the strength of material using the correlation dependencies.

The procedure in question is based on the analysis of shallow wave parameters, which are excited by pulse shock load.

Procedure of integrated evaluation of state and fixing pattern of the structural elements is based on exciting the structure natural vibrations by a shock pulse load, measuring these vibrations in various points of the structure, detecting vibrations at various frequencies by spectral analysis methods and analyzing these vibrations (defining the frequencies and vibration decrements, constructing their diagrams, i.e. dependencies of the amplitude on the measurement point coordinates).

Giving no consideration to the peculiarities of natural vibrations excitation and record, the processing and interpretation results, I'd like to note that the schemes of excitation and measurement of vibrations, modes of identification and detection of various vibration modes had been developed in details, these schemes enable us to determine frequencies with sufficient accuracy and to construct the diagrams for two or three (sometimes even more) first modes of structural vibrations.

Using this procedure permits to find a sound structural model, this model in its turn is the basis to defining elastic material characteristics of a structure (if its dimensions are known), which are averaged through the entire structure or its elements, as well as large defects which affect the form of diagrams of the vibration modes revealed. Examples of vibration diagrams by several forms are presented in Figures 1 and 2. Henceforward the examples presented had been received during inspection of the Leningrad NPP masonry buildings.

Procedure of determination of acoustic characteristics of structural material is based on excitation of shallow waves by shock pulse load, measurement of these waves on the structure surface and analysis of their parameters.

It is known, that the velocity of shallow waves, Rayleigh waves as well, is related to its dependence on the length (variance curve), which is determined by wave propagation velocity distribution in structural material (by dependence of a shear wave velocity and Poisson's ratio on the distance to the surface). Shear wave velocity in its turn is closely related to the shear modulus, and taking into account the Poisson's ratio it is related to the elastic modulus. Thus it is believed that given the variance curve, a shear modulus of various structure layers is determinable.

Such inverse problems in seismic acoustics have no unambiguous solution. However when inspecting the building structures the problem is frequently simplified by the fact that we usually dispose information on geometrical characteristics and material of various structural layers, which permits to receive unambiguous solution.

The purpose of the investigations conducted was to analyze for a possibility to construct shallow wave variance curves from the realizations, derived due to pulse effect (impact). With this aim in view a mathematical modeling had been conducted, in which a multilayer plate response to pulse effect in a point was simulated. The problem has been solved as a plane one. These versions had been simulated: (1) uniform half space; (2) uniform plate; (3) one- and two-layer plate on the half space at various relations of layer thickness and longitudinal wave velocities, and Poisson's ratios in layers and half space; (4) two- and three-layer plate at various relations of layer thickness and longitudinal wave velocities and Poisson's ratios in layers.

The first two versions were simulated to compare the results of variance curve construction with the theoretical data. The obtained realizations of vibrations in various points of surface were filtered through several band filters with 0,5 octave band pass over a wide range of frequencies, after that the filtered-out realizations were used to determine the coordinates of variance curve points. For all the versions variance curves had been constructed, furthermore, for the half space and for the uniform plate the results were in complete agreement with the theoretical data.

In more complicated cases the variance curve form does not entirely coincides with the forms in [1]. The difference is in that variance curves, constructed due to pulse processes, as a rule are free of branches, they are the result of averaging the parameters of waves, propagating in different layers. The one exception is provided by the versions of "soft layer on the rigid base" and "rigid interlayer in the soft soil" at velocities of waves in the layers, which differ several times, where two branches of the variance curve are observed at short waves.

Thus, mathematical modeling had shown that construction of variance curves for shallow waves phase velocities from the pulse realizations creates no problems, and their construction accuracy permits to assess acoustic characteristics of building structure materials (multilayer structure included) with reasonable resolving power. The algorithms developed were used at processing the real objects inspection data, and the modeled variance curves were used to analyze the object variance curves.
Using this procedure permits to assess acoustic and elastic characteristics of materials of various elements of building structures or their separate sections. Figures 3-6 show an example of variance curve construction and its analysis. Thin lines in Fig. 6 show the theoretical variance curves for a plate of 0,25 m in thickness (the thickness of the wall surveyed) at various values of Rayleigh wave velocity at the masonry surface V_R.
The procedures described above could be used independently or in combination. In case of their complex use the elastic characteristics and the material strength are assessed from the shallow waves, and the structural model is selected so that its design dynamic responses (frequencies and forms of resonant vibrations diagrams) most closely agree with the data which had been received experimentally for natural vibration. As an example of such a complex use of this procedures let's consider some results of examination of brick building walls of machine hall of Leningrad NPP.

2. Methodical bases of seismic reevaluation of existing NPP buildings and structures

Evaluation of the building state after design earthquake is carried out by method of direct dynamic load design with the use of real earthquakes records, analog or synthesized accelerogramms. In this condition the actual geology of foundation soils (according to the microseismic zoning data) and their capability to transform a seismic input are taken into account. We had developed a procedure in which soil mechanical behavior during the process of interaction with seismic waves is described on the basis of viscoelastic models (for the magnitude 6 as per the MSK-64 scale and below) and on the basis of elasto-plastic hysteretic models or visco-elasto-plastic models for the magnitudes more than 6. The validity of the procedure developed had been confirmed when solving the problem of verification of parameters of seismic inputs once they had passed through the foundation soils when designing buildings and structures of nuclear power engineering objects.
When evaluating the same building design models with increasing degree of complexity and details are used in succession. The need for such approach stems from the numerous facts that there is no direct dependence of building damage degree (including the same-type buildings) at earthquakes on such parameters as number of stories, structural scheme, etc., therefore successive improvement of the design model will allow to reveal the influence of various factors on building behavior under seismic effects with due regard to damages. The principal condition of design evaluation is accounting for the changes in model states during the earthquake effect under limiting states of separate elements or the building as a whole. In this condition it is necessary to consider the following possible changes of the model state:
•). Building separation into large parts, this separation is marked by through cracks which pierce the entire building in vertical or horizontal directions (adjacent walls separation, building separation and displacement along its foundation, displacement of floor disks and building separation into parts within the stories, etc).
It is evident, that building separation into large parts has a pronounced effect on its natural vibration frequencies. Hence, according to the in-situ measurement data the lateral-direction natural period values in brick buildings (being 0,2 s before Gazly earthquakes) had increased up to 0.4 s after these earthquakes.
At present solving the problem of description of building separation into large parts during seismic input involves the problems related, mainly, to absence of limiting state criteria and proper design models. According to the above procedures in the first stage a building dynamic load design with the use of 3D finite-element model without regard for failures will be performed. Then the building zones in which internal forces (stresses or deformations) exceed the limiting values for the code on aseismic building are to be determined. The design model is supplemented with the elements, which work only up to a certain level load, following which they are eliminated and dynamic load design is performed once again. If new

zones of possible failures arise the further model improvement is to be performed. For example, if in the first design stage we find the possibility of vertical cracking and end building wall separation from the spine walls because tensile stresses exceed their limiting values, then at the sites of their junction the elements are introduced, which are to be eliminated provided that:

$$\sigma_P \geq R^H_{P,}$$

where R^H_P - standard tension resistance for the masonry.

b). Changing the pattern of load perception by building supporting structures due to failure (if some components show a notable loss of bearing capacity) or changing the conditions of element junctions (horizontal cracking in upper and/or lower part of partition walls, diagonal cracking in the blind walls within a story etc).

If such a mechanism of building failure is realized in its design model then either changing the element connection conditions (unilateral constraint forming, sliding friction, etc) is performed, or stress-strain modulus alters locally, or the elements which correspond the destroyed part of structure are eliminated from the design model. In all the conditions specified a change of building stiffness and natural vibration frequency takes place. Once a number of elements had been eliminated from the design model a calculation of the building changed state under static loads is to be performed and the criteria of limiting state are to be specified.

c). Changing the masonry monolithic nature, which, as a rule, decreases the natural vibration frequencies of buildings, that is, their stiffness. These changes are related to numerous internal processes in the masonry under dynamic load input, which are associated with formation of tiny cracks and its continuity disturbance. It is to be noted, that masonry is a non-uniform elastic plastic body which consists of solution-filled stones and joints. It is obvious that in this case the choice of integrated parameter which accounts the entire set of factors will be a proper one. Numerous investigations use as such a parameter a stress-strain modulus of masonry, multiplied by degradation factor, dependent on the stressed-deformed state in building structures.

Under compressing stresses in masonry ($\sigma_P \geq 0.3R_{cm}$) the current stress-strain modulus is recommended to be determined as follows

$$\bullet_1 = \alpha\,(R_{cm} - 0.91\sigma).$$

where (α - masonry elastic characteristic according to Russian norms and standards,

R_{cm} - standard compression resistance of masonry.

Similar analyses for determination of masonry degradation factors due to various conditions are under investigation in many countries. The paper [2] shows the analysis for the most commonly used degradation models and comparison of design evaluations with research data on wide and narrow partition walls.

3. Practice of seismic recertification of NPP building structures

As example of the procedures above is the design results of a relatively simple NPP water fire extinguishing pump (WFEP) building for SSE of magnitude 7 as per the MSK-64 scale. During dynamic load design a 3D finite-element building model was used. As a result we received the displacements of all the model units and the forces in building structural elements. The designs carried out gave the evaluation of additional loads influence on meeting the strength conditions, the loads arising due to seismic input to building structures. The standard criteria of strength used at building model evaluation allowed to determine the possibility of its separation into large parts and to reveal the zones of localized failures.

WFEP building overall view is presented in Fig. 7. This building plan view is a rectangle which measures 20.51•10.2 m. Building height is 8.6 m at the 0.0 m level. In design the WFEP building includes three parts which differ in their stiffness and inertial properties: modular and monolithic reinforced concrete foundation from the building base up to 0.0 m level; masonry walls above the 0.0 m level, and the building roof, which is made from reinforced concrete panels and is at the + 8.3 m level.

Development of a finite-element WFEP building model was carried out from the operating drawings. In doing this all building design features (windows and doorways, brick masonry of various thickness, floor plates scheme etc) were reproduced in the model. In the places of possible building separation (in the

places of outside walls junctions among themselves and with internal partitions, as well as in the zone of building walls connection with foundation) the design model was added with the elements, which were eliminated as soon the limiting value of forces had been achieved. Besides, the changes in the masonry stress-strain modulus when achieving the limit value of compressing stresses was taken into account.

Analysis of building structural configuration had shown that absence of symmetry planes in plan view does not result in substantial displacement of building centre of gravity relatively its geometrical centre, this displacement was equal ~0.03 m in lateral direction and ~0.24 m in longitudinal direction. Building stiffness in longitudinal and lateral directions with due regard to ground foundation stiffness was $2.287 \cdot 10^9$ n/m and $0.912 \cdot 10^9$ n/m respectively.

Analysis of the static design results had shown that the stresses acting in brick masonry sections (compression, shear, etc) and deformation are substantially lower than allowable standard values, this agrees with the building examination data.

As the design seismic input we took the accelerogramms of 9.03.1949 Holister earthquake. The parameters of seismic input at the building foundation had been determined due its passing through the foundation soils, the structure and physical characteristics of which were determined from the microseismic zoning data. Both accelerogramms and velocity and soil-surface displacement curves, as well as response spectra corresponding to the design basis earthquake are shown in Figure 8.

The displacement diagrams show that at earthquake the soil surface is rocking with amplitude ~2 cm relatively the common non-compensated soil displacement, equal to ~2 cm in horizontal and ~4 cm in vertical direction. As for their value the hard soil surface displacements correspond those observable at earthquakes of magnitude ~ 7 (• SK-64 scale). Analysis of the effect spectra permits to state that:

- the heaviest rate of seismic input is displayed within the range of frequencies from 3 to 15 Hz both in vertical and horizontal directions, where the dynamic coefficient of the effect is ~4.
- for the horizontal effect component the presence of one peak of acceleration in the spectrum at frequency ~ 5 Hz (4.8 m/s^2) is typical;
- for the vertical effect components in the spectrum two peaks of approximately equal rate (~ 1.7 m / •2) at frequencies 4 and 9 Hz are observed.

From the results of WFEP building dynamic load design for the earthquake of magnitude ~7 (MSK-64 scale) due to Holister accelerogramms we had determined:

- failure of the building brick part connections with reinforced concrete foundation takes place practically immediately, this serves the decrease of wall loads;
- maximum displacements of brick part of WFEP building relatively the foundation are: 1.5 cm along the short building side and 1.1 cm along the long building side;
- strength of brick WFEP building walls at the preset seismic input is obtained with the exception for the partition, wall crosswise the building, in its upper part the conditions of lateral shear strength are disturbed and a crack is formed, however the partition doesn't separate from the outside building walls at its whole height;
- reducing the building stiffness due to change in masonry deformation module, related to its continuity disturbance and tiny cracks formation, doesn't exceed 5%.

At present the works on seismic recertifications of Russian NPP structures and buildings on the basis of the above procedure are in progress. The results received are used at NPP safety analysis based on the results of seismic revaluation of equipment and pipelines response and state.

References

1. Jones R, Non-Destructive Testing of Concrete. Cambridge University Press, 1962
2. Djaalali F., Afra H., Berrah M.K. Seismic Damage Assessment for Masonry Structures. // 11-th European Conf. on Earthquake Engineering, Rotterdam, 1998.

Figure1. Vibration diagrams in vertical plane.

Figure 2. Vibration diagrams in horizontal plane.

Figure 3. Initial realizations

Figure 4. Filter realizations (filter transmission frequency is 1 kHz)

Figure 5. Filter realizations (filter transmission frequency is 500 Hz)

Figure 6. Variance curve.

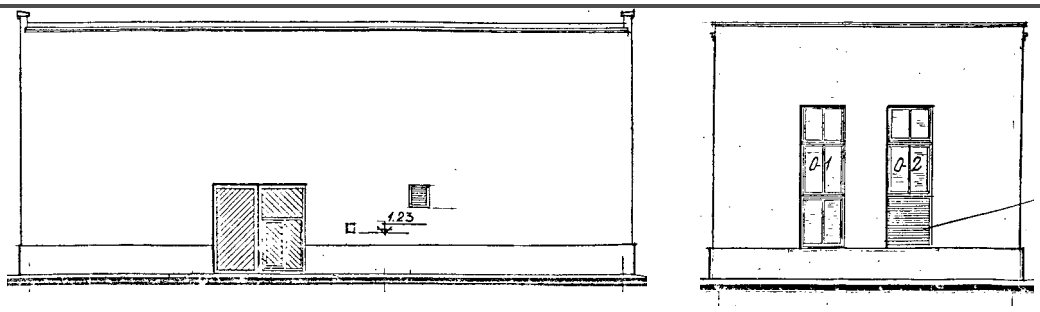


Figure 7. Pump building overall view

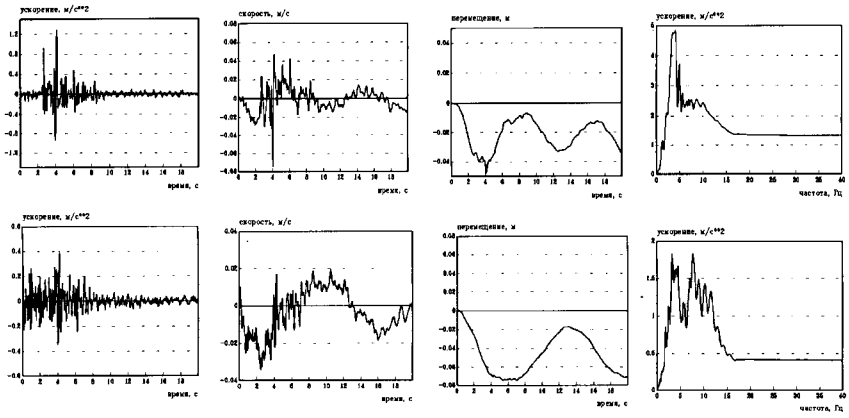


Figure 8. Design seismic impact .

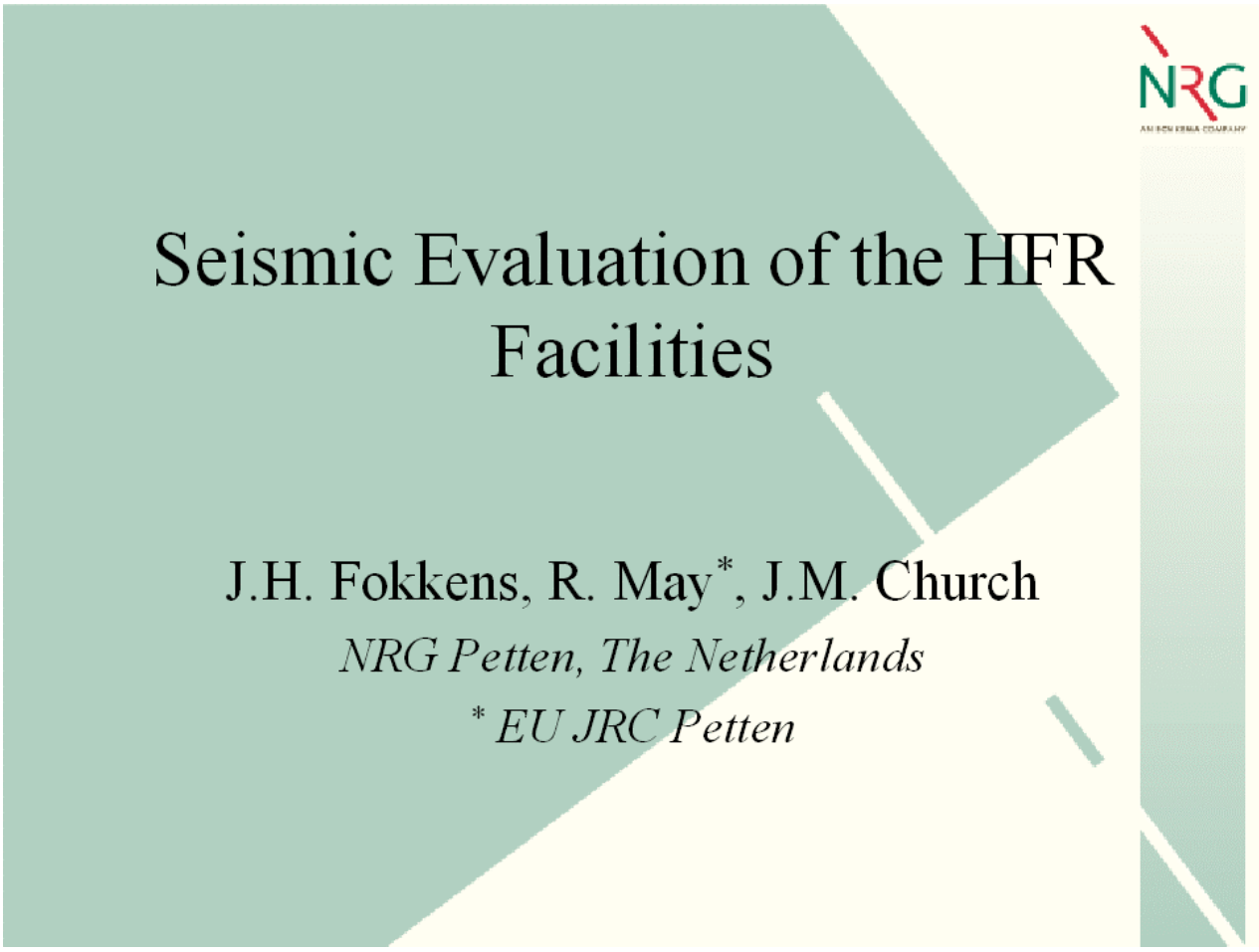
Seismic Evaluation of the HFR Facilities

J.H. Fokkens

Abstract:

NRG has more than 35 years experience in the nuclear field. Amongst others, this includes safety and integrity assessments and operation of the 50 MWth High Flux Reactor (HFR) owned by the European Commission. At the request of the Dutch Nuclear Regulatory Body a safety re-evaluation was performed for the HFR facilities. Special attention was paid to seismic evaluation in view of recent gas and oil exploration in the vicinity of the HFR site and the potential for induced earthquakes.

Initially, a Basic Design Earthquake spectrum was defined for the HFR site on the basis of available seismic data and regulatory requirements. For the various HFR components acceptance criteria and requirements were agreed upon with the Dutch Nuclear Regulatory Body. The earthquake resistance of the HFR components was evaluated using finite element analyses, with strengthening modifications proposed for those components that did not meet the acceptance criteria.



Introduction

--- Applied Mechanics Services ---

- **50 MW High Flux Reactor (HFR)**
  - *Research reactor sited in Petten, The Netherlands*
  - *Owned by the European Commission*
  - *Operated by NRG*
  - *Used for materials research and isotope production*

Background to Analysis

--- Applied Mechanics Services ---

- **HFR cited in an area of low seismic activity**
  - *Seismic analysis not previously performed*
  - *Several minor tremors (3.8 Richter) during 1994 in surrounding area*
  - *Potential for earthquake with epicentre close to HFR*
  - *Earthquakes attributed to gas exploration in area*
  - *Nuclear Regulator asked NRG to consider seismic assessment as part of licensing renewal*

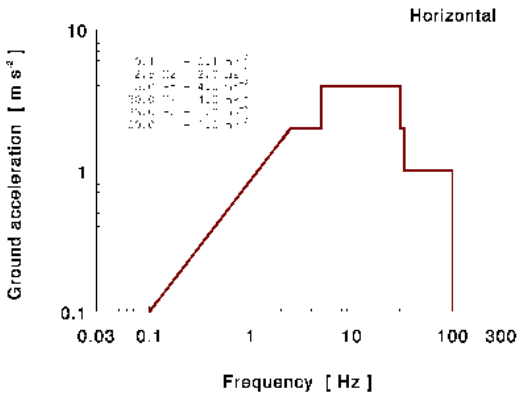
Reference Seismic Loadcase

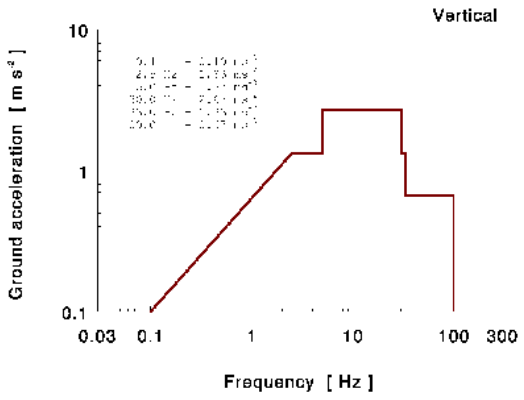
--- Applied Mechanics Services ---

- **No previous assessment, therefore no established loadcase**
  - *Comparable geological sites in coastal areas of Germany for which earthquake response spectrum defined (German Guideline KTA 2201.02)*
  - *Royal Netherlands Meteorological Institute (KNMI) provided estimate of maximum likely ground accelerations near site (gas exploration) based on bore hole measurements*
  - *Maximum acceleration 4 ms^{-2}*
  - *Frequency range 5 -30 Hz*

Reference Seismic Loadcase

--- Applied Mechanics Services ---





Components & Structures

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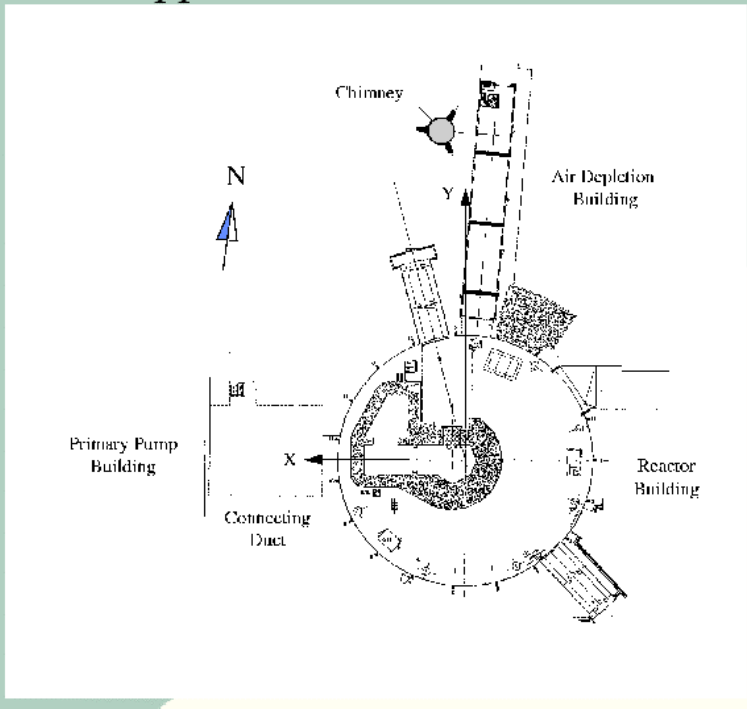
- **Primary cooling system**
  - *Line element F.E. model. ASME sub-section NC design loadcase and level A and B service conditions*
- **Primary pump building**
- **Reactor containment building**
- **Reactor building foundation structure**
- **Air depletion chimney**

HFR Facility Layout

--- Applied Mechanics Services ---



Primary Pipework System

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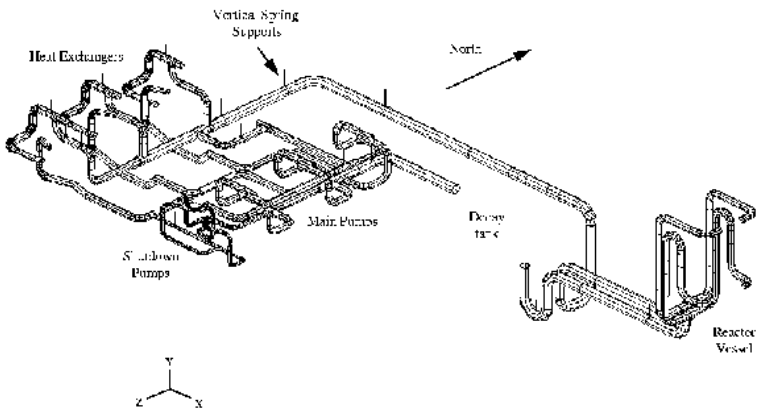

Reactor Containment Dome

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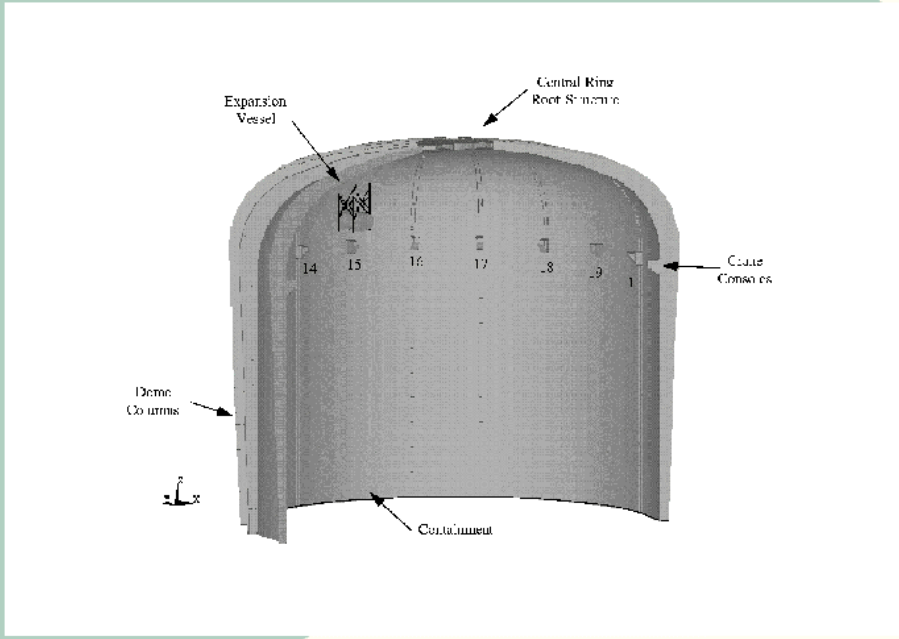
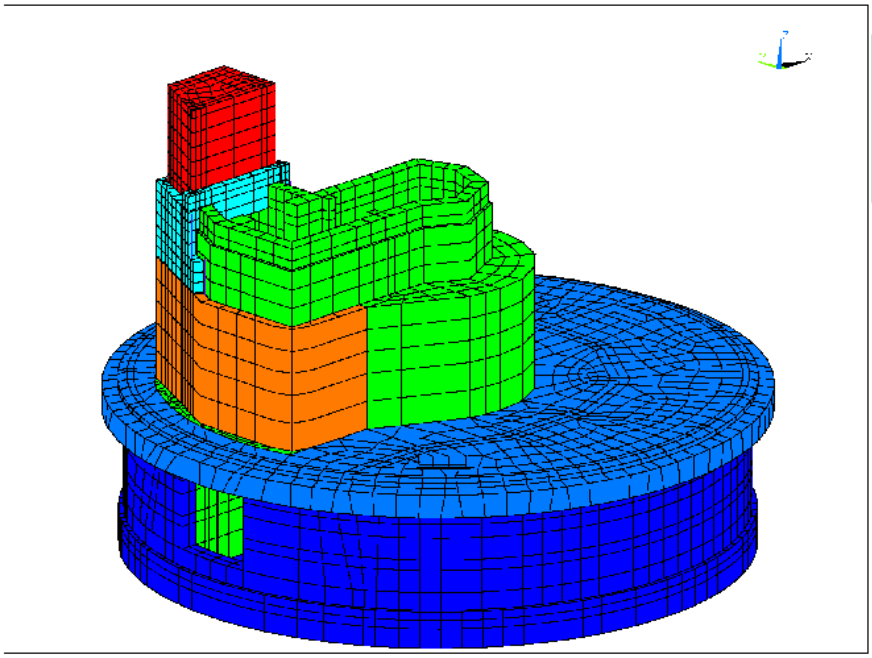


Concrete Foundation

--- Applied Mechanics Services ---



Chimney Model

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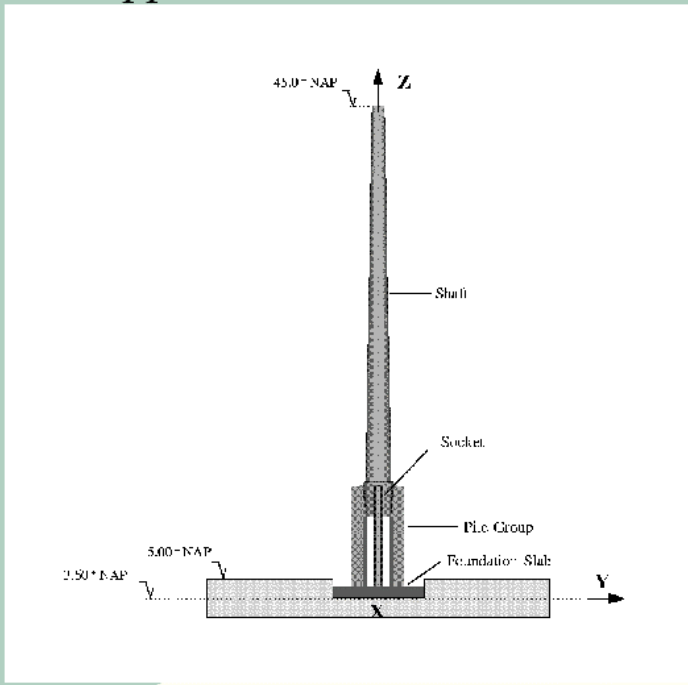

Summary of Results

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- **Primary cooling system**
  - *System satisfied ASME requirements, But slight over stress at 2 hanger locations, based on ASME 1995 due to thermal expansion. Spring design adapted*
- **Primary pump building** - *Satisfactory*
- **Reactor containment building** - *Satisfactory*
- **Reactor building foundation structure** - *Satisfactory*
- **Air depletion chimney** - *insufficient reserve*

Chimney Assessment

--- Applied Mechanics Services ---

- **Requirement 25% reserve on stress**
  - *Reserve requirement agreed with Nuclear Regulator*
  - *2 compression piles (1990 kN) : 100% reserve*
  - *1 tension pile (1043 kN) : 10% reserve*
- **Chimney failure unlikely given conservative assumptions made during analysis**
- **De-design scenarios considered**
  - *Full reinforcement of Pile Group*
  - *Partial reinforcement of Pile Group*

Pile Group: Partial Reinforcement

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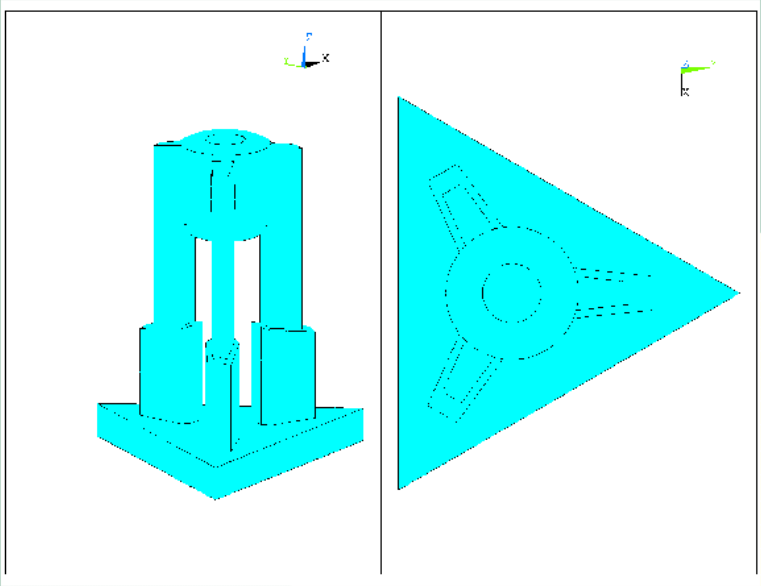

Pile Group: Full Reinforcement

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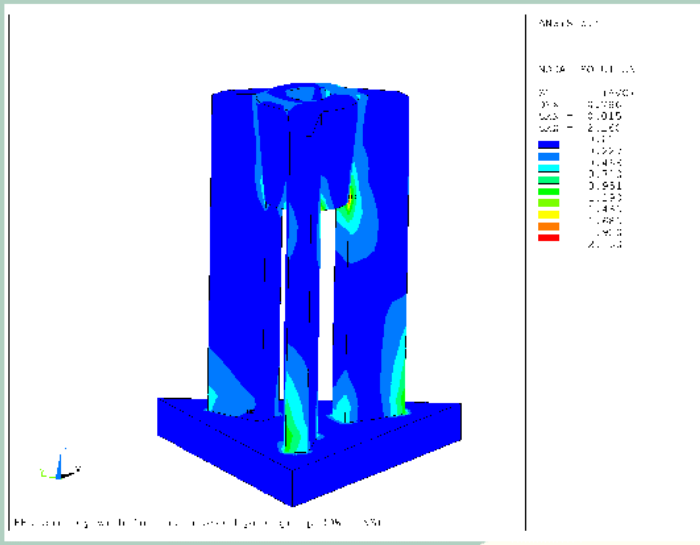

Minimum reserve factor = 0.99

Nodal average maximum principal stress profile

Pile Group: Partial Reinforcement

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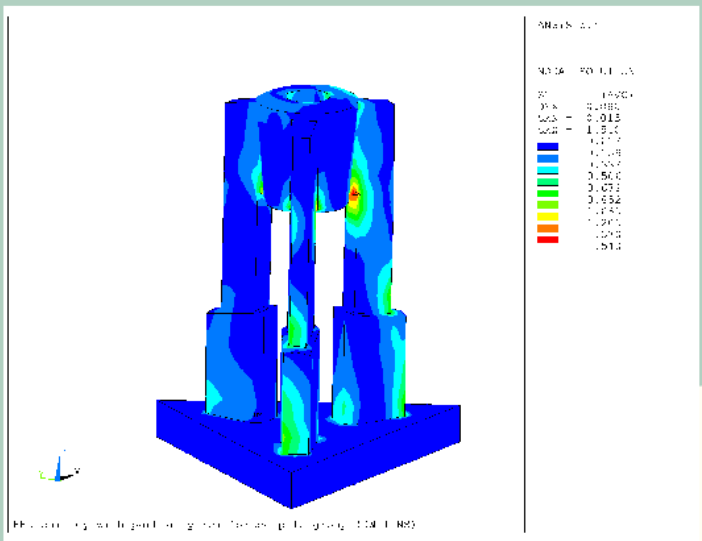

Minimum reserve factor = 1.29

Nodal average maximum principal stress profile

Chimney Assessment

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- **Recommendation**
  - *Modification of Pile Group based on partial reinforcement to improve earthquake resistance*

General Conclusions

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- **Seismic analysis of the HFR facility undertaken as part of on-going safety assessment and license renewal**
- **All major components deemed acceptable, with exception of Chimney**
- **Reserve strength factor for chimney pile group below regulatory requirement of 25%**
- **Re-assessment of the chimney pile group undertaken**
- **Minimum reserve factor increased to 29.9% on basis of partial reinforcement**

RE-ASSESMENT OF SEISMIC SAFETY OF TR-2 RESEARCH REACTOR BUILDING

AyhanAlt•nyollar
Turkish Atomic Energy Authority, Ankara-Turkey

Polat Gülkan
Middle East Technical University, Ankara-Turkey

Abstract
The TR-2 research reactor at Çekmece Nuclear Research and Training Center, operated by the Turkish Atomic Energy Authority, is located 25 km west of •stanbul in Turkey. It was built during the period 1957-1962. In keeping with the practice of the time, a site specific design basis earthquake was not considered in the original design of the reactor building. Following concerns raised following the magnitude-7.4 17 August, 1999 earthquake near •zmit in the province of Kocaeli, a distance of some 85 km from the reactor building, the seismic qualification of TR-2 reactor building was re-assessed Using new methodologies and experience. During the Kocaeli Earthquake, the acceleration time history at the base of TR-2 research reactor building was recorded.
The structural model of the TR-2 research reactor building was developed and a dynamic soil-structure interaction analysis was performed using the SASSI computer code. Generic time history (obtained from NUREG/CR-0098) and the time history recorded at the site during the Kocaeli Earthquake, were used in the analysis. The soil degradation that occurs during an earthquake was considered in the analysis. The soil structure interaction analysis of the model was carried out using SASSI, and the static analysis of the reactor building was performed using the SAP2000 computer code. Seismic safety of the TR-2 research building was evaluated using the results of both analyses.
The main structural elements to be checked in the building against earthquake loads are the shear walls surrounding the building at the base, the frames in the superstructure, and the infill brick walls.
The acceleration response spectra and time histories of points on which the safety equipments are located, were also generated for the use of seismic qualification of these equipment.

1. Introduction

At the present time, there are numerous programs underway in many countries around the world for the evaluation of seismic safety of research reactors and for their upgrading to increase seismic safety. Research reactors are usually located at or near city centers. This is different than nuclear power plants. The seismic safety of research reactors located in high seismic areas but close to urban settlements is a vital issue.

Many areas in Turkey are exposed to seismic hazard. For example, a devastating earthquake occurred in Turkey at the eastern end of Marmara Sea on August 17 1999 (03:02 Local Time, Latitude: 40.70N, Longitude: 29.91E, Depth: 15.9 km, Magnitude: 7.4). The epicenter was located near Gölcük, a town located about 4 km from •zmit City in the Kocaeli province. Field surveys confirmed that the western part the North Anatolian Fault Zone had caused the main shock. A few buildings also collapsed in residential parts of Avc•lar district in •stanbul, about 5 km away from the TR-2 research reactor building. Seismic instruments at the reactor-building basement and at the base of another surface building produced records during earthquake. The peak acceleration values of the time histories recorded at the basement of reactor and the other building were 0.12g and 0.18g, respectively. The TR-2 reactor has been designed for a base shear force of 0.1 times its weight. Inspection of the reactor building by engineers after the earthquake showed that there was no damage on the reactor building, systems and components.
The TR-2 research reactor was built between 1957-1962, and at that time a site specific design basis earthquake was not considered in the design of the reactor building. Considering the exposure of the facility to damaging ground motions, the seismic qualification of the TR-2 reactor building was re-

assessed. This paper summarizes the findings. Mainly, the following headings were considered in the studies:

- Methods, regulations and techniques used for the re-assessment of the seismic safety of the TR-2 Research Reactor,
- Earthquake hazard at the TR-2 reactor site,
- Site characteristics,
- Determination of dynamic response of the reactor building,
- Evaluation of the seismic safety of the reactor building.

2. Description of the Reactor Building

The reactor building is a reinforced concrete frame structure. The plan dimensions of building are 23.54x32.58 m. The building has an embedment to a depth of 9.12 m. The height of the building is 11.25 m from the grade level. The structure has a thick base slab and embedment part is covered by shear walls acting as retaining walls. The embedment part of the structure consists of four levels. Above grade level, the superstructure consists 10 one-way frames supported directly on reinforced basement walls. The superstructure supports a moving crane.

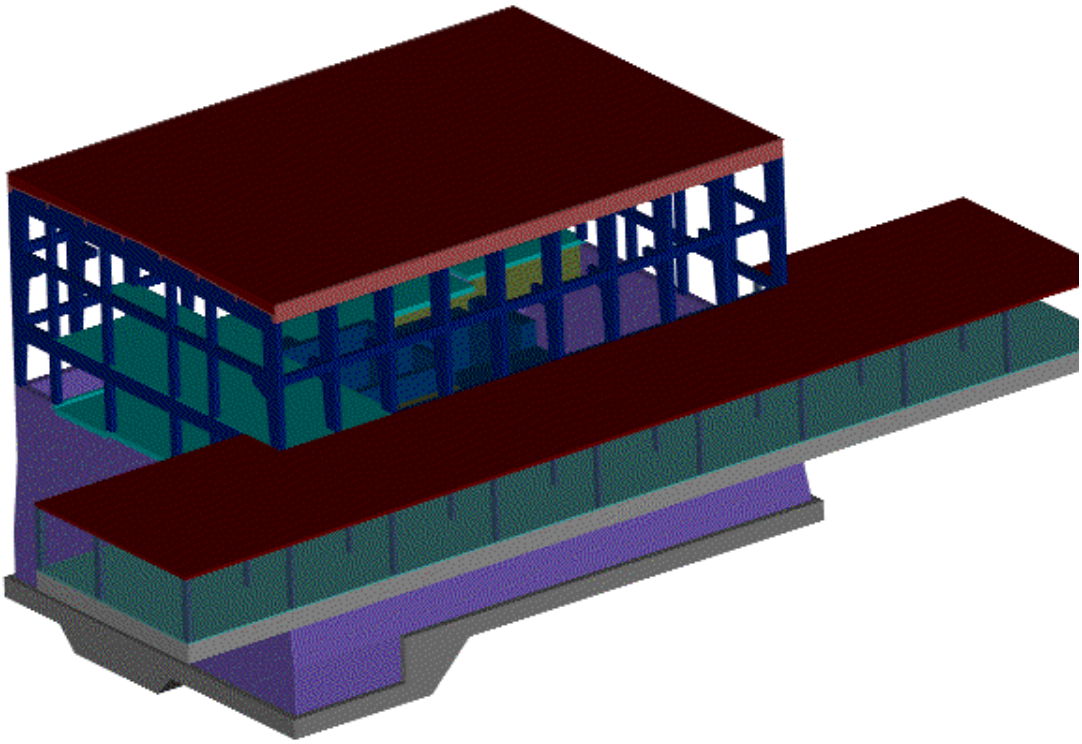


Figure 1. TR-2 reactor and auxiliary building

3. Description of Analysis Input

3.1 Artificial Time History

Review Level Earthquake (RLE) has been established to be a ground motion with 0.40 g peak ground acceleration (PGA) in horizontal direction (Ref. 1). The 0.40g PGA is assumed to be a mean estimate for a 500 year of return periods (Ref. 2). The time history trace obtained from NUREG/CR-0098 was normalized to 0.40g (Figure 2) and used for time history input for capacity check purpose in the analysis.

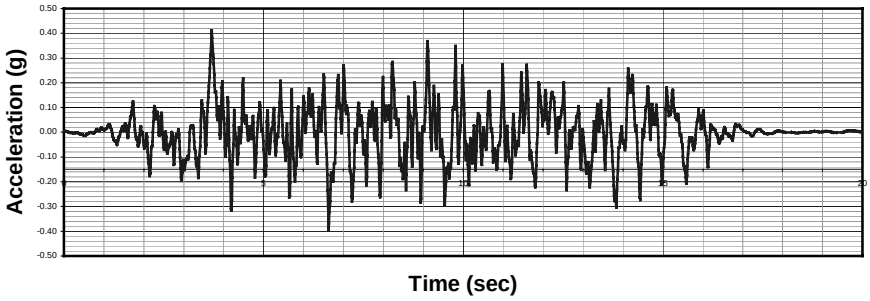


Figure 2. CR0098 Time History, 50th Percentile - Soil, Horizontal Component (N-S)

3.2 Time histories recorded at TR-2 site

There are two acceleration traces recorded at the site during the Kocaeli Earthquake. One of them was recorded at one of laboratory buildings at the site. This record will be considered as a free field in the analysis because the building is not heavy and it has no embedment (Figure 3). The maximum acceleration values of the free field record are 0.18g, 0.13g and 0.059g for horizontal components 1, 2 and the vertical component, respectively. The other trace was recorded at the basement of TR-2 research reactor building (Figure 4). The maximum acceleration values of this record are 0.12g, 0.09g and 0.05g for horizontal components 1, 2 and vertical component, respectively. These will be used for verification purpose.

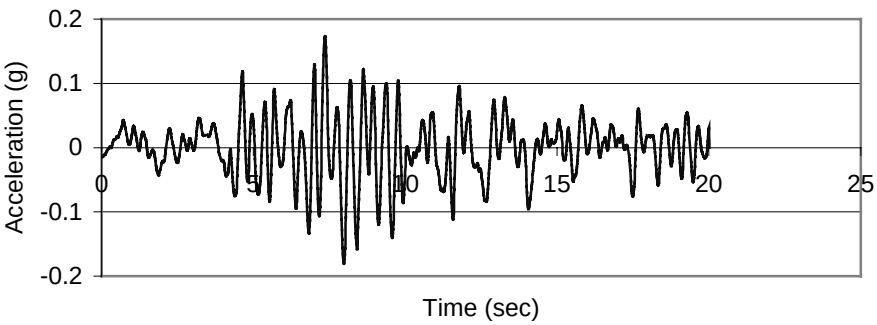


Figure 3. Recorded time history at site (free field), horizontal component (N-S)

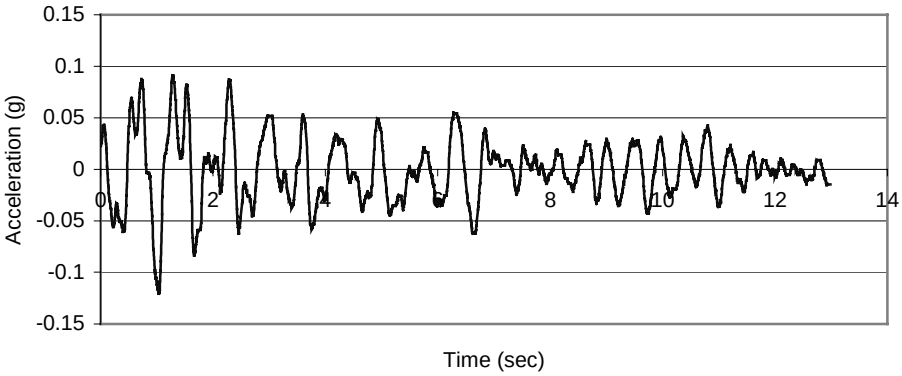


Figure 4. Recorded Time History at the base of reactor building, component (N-S)

3.3 Soil properties of theTR-2 site

Detailed soil investigation studies were conducted at the TR-2 site in 1984 (Ref. 3). Soils underlying the TR-2 reactor building are stiff to very stiff clays and silts down to considerable depths. Bedrock was not encountered within the depth of 75 m borehole exploration. Shear wave profile of the upper soil layers is given in Figure 5.. The groundwater table is stabilized at about 36.20m.

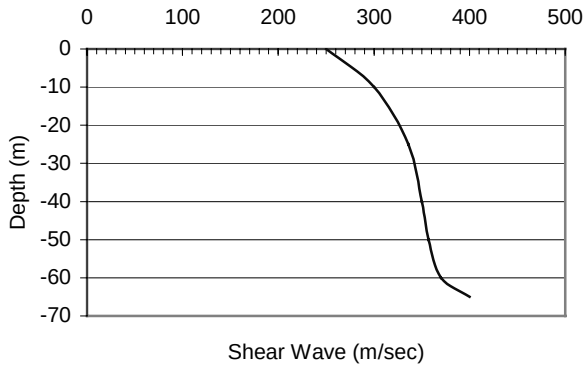
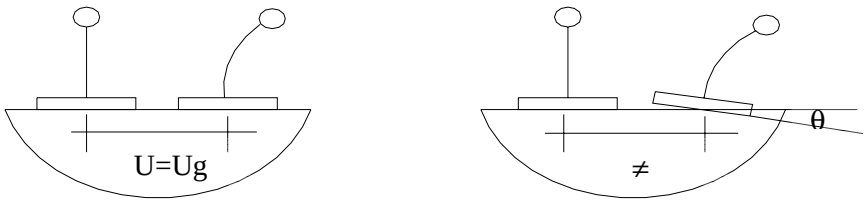


Figure 5. Shear Wave Profile of TR-2 site

4. Soil-Structure Interaction

Soil-structure interaction results from the scattering of waves from foundations and radiation of energy from the structure due to structural vibrations. The dynamic response of structures depends on the characteristics of the underlying soil medium. If the soil condition is soft, the coupled soil-structure system will exhibit a peak structural response at a lower frequency compared to a fix base structure.
Regular earthquake analyses ignore soil structure interaction effects. Free field displacements are applied as is to the base of structure. Soil structure analysis includes interaction effects between soil and structure.



Regular Earthquake Analysis *Soil-Structure Interaction Analysis*
Ug : Free field displacement, U : Displacement at the base of structure, θ : Rocking angle

Figure 6. Response of structure under the earthquake

The reactor building structure, foundation and soil were modeled and analyzed by SASSI computer program in which flexible volume sub-structuring method is used (Ref. 4). The method is formulated in the frequency domain using complex response method and the finite element technique.
In the SASSI program environment, the soil-structure interaction problem reduces to three main steps for each frequency as follows:

- Solution of the site response problem to determine the free-field motions within the embedded part of the structure,
- Solution of the impedance problem to determine the dynamic stiffness of the foundation at the interaction nodes,
- Solution of the structural problem.

5. Structural Model of Reactor Building

5.1. Static Model

A 3D finite-element model of the TR-2 reactor building was generated using appropriate solid, shell, beam and spring elements. The numbers regarding the static model are as follows:

Number of nodes : 1000

Frame elements: 654

Shell elements : 685

Solid elements : 112

Water in the pool is modelled as an equivalent system of masses and springs as described in Ref. 5. The elastic in-plane stiffness of a solid unreinforced masonry infill panel prior to cracking is represented with an equivalent diagonal compression strut in the model (Ref. 6). Heavy equipment, such as tanks, pump, reactor etc., are included as masses. The cranes are also included. This model is shown in Fig. .

5.2. Soil-Structure Interaction Model

Soil-Structure interaction model of the reactor building was generated for SASSI adding embedment soil part to the model. The Embedded soil part was modelled by solid elements with material properties of soil in the corresponding levels. The maximum size of elements was decided according to soil properties and

frequency range of input motions. In the SASSI program, there are also spring elements which are neccessary to represent water in the pool. The numbers representing the model regarding the soil-structure interaction model were as follows:

Number of nodes : 1531

Frame elements : 638

Spring elements : 16

Shell elements : 685

Solid elements (Structure): 112

Solid soil elements (Soil): 611

6. Dynamic Response Analysis Results

Dynamic response of the reactor building was obtained in the form of time histories of motionparameters (displacements, velocities and accelerations). Input time histories for two horizontal and the vertical direction were assumed to be statistically independent. In the analysis, these are applied simultaneously and the results in each direction at any point of the structure contain the influence of all input motions. The time history and maximum values of the internal forces and moments are evaluated.

6.1 Maximum Accelerations

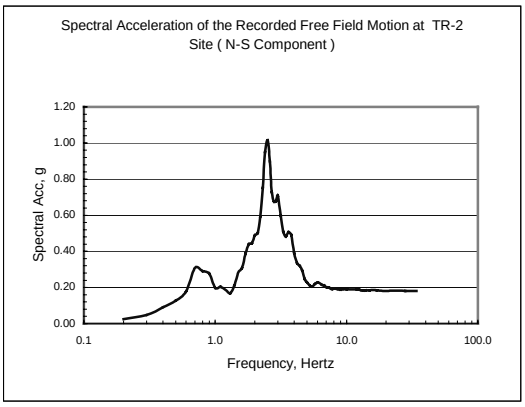
The maximum acceleration values at some nodes due to recorded free feild time history in three components are given below. The nodes on the basement walls, the nodes where accelerograph was located, and the node at the top of the building have been selected. The results are tabulated as follows:

Maximum Acceleration at the nodes due to free field time history in X direction (PGA 0.18g)						
Nodal Points	Max Acc. - X	At time	Max Acc. - Y	At time	Max Acc. - Z	At time
Along the basement wall (below grade level)						
1	0.158	7.96	0.0015	7.02	0.0108	7.37
61	0.1599	7.96	0.001	7.02	0.0109	7.37
349	0.1627	7.96	0.0007	7.15	0.0113	7.37
699	0.1647	7.96	0.0006	7.46	0.012	7.37
979	0.1655	7.96	0.0012	7.56	0.0133	7.37
At node where the accelerograph was located						
104	0.1602	7.96	0.0003	7.45	0.0017	19.99
At top of the building						
1513	0.008	9.92	0.3914	9.06	0.0066	9.26

Table 1. Maximum accelerations at selected nodes due to recorded time history

6.2 Acceleration Response Spectra

Acceleration response spectra for the recorded free field motion are given below. Acceleration response spectra where the point accelerograph located obtained from analysis results and recorded one for three component and 5 percent damping are also presented below.



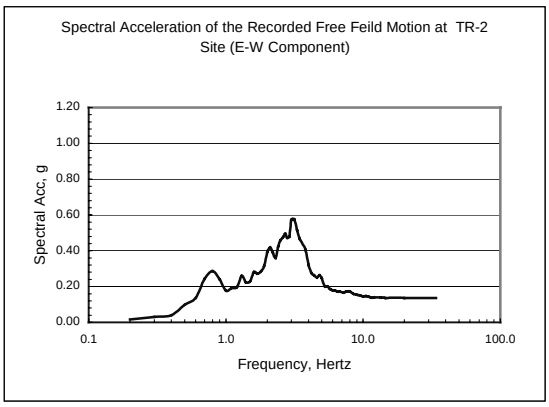
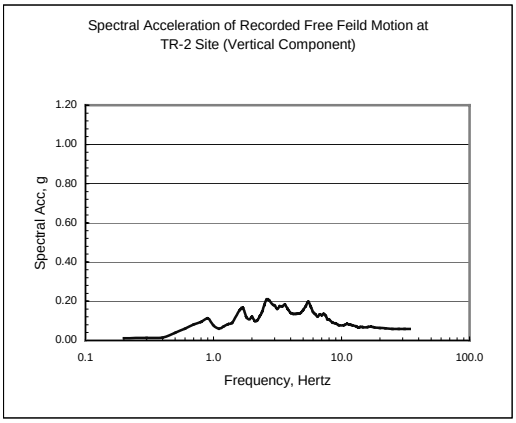


Figure 7.a *Figure 7.b*

Figure 7.a. Spectral Accelerat•on of Recorded Free Field Motion (N-S Comp.)

Figure 7.b. Spectral Accelerat•on of Recorded Free Field Motion (E-W Comp.)



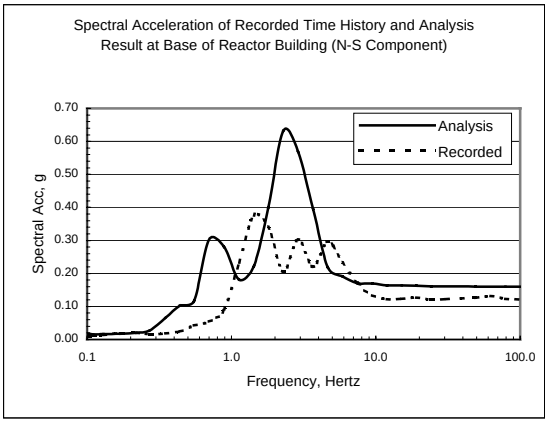
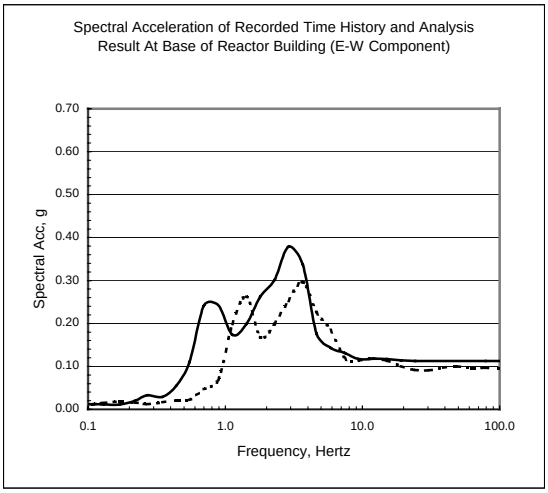


Figure 7.c *Figure 8.a*

Figure 7.c. Spectral Acceleration Diagram for Recorded Free Field Motion (Vertical Comp.)

Figure 8.a. Spectral Acceleration Diagram for Recorded Time History and Analysis Results at the Base of the Reactor Building (N-S Comp.)



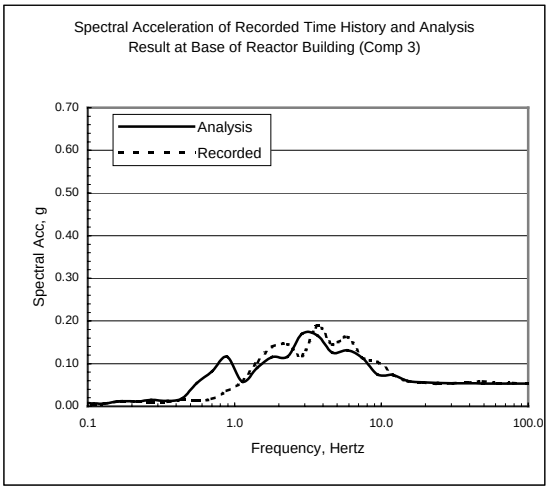


Figure 8.b *Figure 8.c*

Figure 8.b. Spectral Acceleration for Recorded Time History and Analysis Results at the Base of the Reactor Building (E-W Comp.)

Figure 8.c. Spectral Acceleration for Recorded Time History and Analysis Results at the Base of the Reactor Building (Vertical Comp.)

7. Discussion of Results

Acceleration response spectra of the recorded time history and analysis results at the basement of the TR-2 research reactor building are plotted in the same diagram for three components. Analysis results and recorded values are in poor in agreement in the low frequency region for all three components. They also match rather poorly for the North-South components. One of the possible reasons is the topography of the reactor site. The reactor is built on top of a hill. On the other hand, soil layers are assumed to be horizontal in the software we have used. The acceleration time history recorded at the base of surface structure is considered as the free field motion. But it may include response of the neighboring structure also. Another reason is accuracy of the shear wave velocity profile. Shear wave profile is derived from the standard penetration test results using empirical relations due to absence of geophysical studies at the site.

8. Acknowledgment

The authors wish to express their gratefulness to Mr. Alejandro Asfura for his valuable suggestions. We would like to thank to Mr. Feridun Saral for his help in the preparation of the figures..

References

1. NUREG/CR-0098
2. A Seismic Zones Map of Turkey Derived from Recent Data, Polat Gülkan, Ankara, Jan. 1993.
3. Seismic Safety Assessment of CNEAM – 2 Research Reactors, Yener Özkan, Ankara ,1984.
4. ACS SASSI (An Advance Computational Software for 3D Structural Analysis Including Soil-Structure Interaction) Computer Program Manual,
5. Fundamentals of Earthquake Engineering, Nathan M. Newmar and Emilio Rosenblueth.
6. NEHRP Guidelines for the Seismic Rehabilitation of Building, FEMA-273, Oct 1997.

SEISMIC PROVING TEST OF CONCRETE CONTAINMENT VESSEL

Youichi Sasaki and Kiyoshi Hattori

Nuclear Power Engineering Corporation

ABSTRACT

A concrete containment vessel (CCV) in nuclear power plant is important structure of its safety role to be a final barrier to prevent radioactivity release. The CCV is required to maintain leak-tightness even for severe earthquake. In order to prove the structural integrity and functional soundness of CCV, Nuclear Power Engineering Corporation (NUPEC) conducted seismic proving tests of prestressed concrete containment vessel (PCCV) and reinforced concrete containment vessel (RCCV) scaled model using large-scale and high performance shake table of Tadotsu Engineering Laboratory, located in Tadotsu, Japan

The structural integrity and functional soundness up to the extreme design earthquake (S2) were confirmed in the tests. After design earthquake level test, seismic margin tests were conducted using magnified acceleration input of S2. The test model of PCCV and RCCV were damaged finally in large input failure test. From the results of the seismic margin test and the failure test, CCV was evaluated to maintain structural integrity and functional soundness until the seismic loads were five times of S2. The tests were conducted under the sponsorship of Ministry of Economy, Trade and Industry (METI) of Japan.

1.INTRODUCTION

The reactor containment vessel contains the nuclear reactor as well as other important equipment and pipings. It plays an important role in preventing dissipation of radioactive substances to exterior by bearing the load caused by high pressure and temperature in case of a loss of coolant accident. In the conventional schemes, in order to meet the requirements of high pressure resistance and leak-proofness, a steel containment vessel is usually adopted. However, recently, as the size of power plants increases, concrete containment vessels have been used, as it is favorable for construction. Compared with steel containment vessels, concrete containment vessels (CCVs) have many advantages: higher damping characteristics with respect to dynamic loads, a large degrees of freedom with respect to shape and wall thickness, and hence, ability for appropriate layout and design. From the viewpoint of structure, the CCVs can be divided into two types: prestressed concrete containment vessel (PCCV) and reinforced concrete containment vessel (RCCV).

Nuclear Power Engineering Corporation (NUPEC) has been conducting the shaking tests of PCCV and RCCV using large scaled model (1/10 for PCCV and 1/8 for RCCV) with liner plates. The objective of the test is to prove structural integrity as well as the functional soundness of CCVs. The test has been carrying out using the large-scale and high-performance shaking table at Tadotsu Engineering Laboratory of NUPEC. Seismic proving test schedule of CCVs is shown in Figure 1.

2. TEST MODEL

PCCV test model simulates a cylindrical wall of an actual PCCV. The dome is not incorporated in the model since the cylindrical wall, excluding dome, is the section where large shear forces are generated due to seismic excitation. The overall configuration is of 1/10 scale and the wall thickness is of 1/8 scale. Those scales are determined considering the constructability of reinforcing bars and tendons. The test model was lined with liner plates to secure air-tightness. The scale of the liner plate thickness is 1/4 and determined by the welding manufacturability. Figure 1 shows the general arrangement of the test model. The attached mass of 434 tons was placed on the top of the test model.

RCCV test model was a scale model of an RCCV-portion of an ABWR-type nuclear power plant prototype building. The outline of the model is shown in Figure 3. A part of the floor slab is modeled so that the structural behavior with internal pressure is equivalent to the prototype. Masses are attached to the top of the model in order to provide proper acceleration and stress equivalent to those of the prototype. The test model consists of a shell wall, L/D (Lower Diaphragm) access tunnel openings, and a set of steel liners attached to the internal shell surface. The modeling scale is 1/10 for shell wall, 1/4 for liner plate thickness, and 1/8 for over all configuration.

3. INPUT EARTHQUAKE EXCITATION

For PCCV test, the scale of acceleration amplitudes of input earthquake is fixed to 1/1.32 based on the weight ratio of the test model and the actual PCCV. The time increment is determined to 1/2.52 based on the ratio of the natural frequencies between the test model and the actual PCCV. As an example of input seismic excitation, Figure 4 shows the input time history corresponding to the extreme design earthquake (S2) for PCCV test. In the seismic margin test, the magnitude of the excitation force around the to-be-shifted natural frequency was enlarged to enable the test model to receive sufficient excitation force up to collapse.

For RCCV test, the scale of acceleration amplitudes of input earthquake is fixed to 1/1 for RCCV model. The time increment is determined to 1/2.83. As an example of input seismic excitation, Figure 5 shows the input time history corresponding to the extreme design earthquake (S2) for RCCV test. In the seismic margin test of RCCV, S2 acceleration amplitude was multiplied.

4 TEST RESULTS

(1) Response

Table 1 summarizes the responses of the PCCV test model at major seismic excitations. The seismic excitations were carried out sequentially from smaller one to larger one. Structural integrity and air-tightness was maintained in design level test from S1(H) to LOCA+S1(H+V). Before 5.0S2(H) test, structural integrity and air-tightness was also maintained in seismic margin test. During the last test of 5.0S2(H), shear sliding failure occurred at the side of the equipment hatch and led to the total collapse.

Crack pattern is draw in Figure 6. Leak rate at most distorted part at inside liner surface measured after 5.0S2(H) indicated that air-tightness was maintained

The responses of the RCCV test model were summarized in Table 2. Structural integrity and air-tightness was maintained in design level test from 1.3S1(H) to 1.0S2(H)(Test No.R10). Before 9.0S2(H) test, structural integrity and air-tightness was also maintained in seismic margin test. During the last test of 9.0S2(H), structural shear failure was observed near openings and bending failure was observed at the bottom of the shell wall nearly at the same time. Crack pattern is draw in Figure 7. Investigation after 9S2(H) test revealed that there had been tearing of the liner plate near the openings.

(2) Frequency and Damping

The measured dominant frequencies of test models were shown in Figure 8 for PCCV test model and Figure 9 for RCCV test model. PCCV test model shows constant value of about 10 Hz before 2S2(H+V). After 2S2(H), dominant frequency shifted gradually to lower side. Damping ratio was about 1-2%. Dominant frequency of RCCV test model at first condition (experienced no shaking test) was about 13.5 Hz. The early tests significantly reduced the frequency due to stiffness degradation. After 5S2(H) test, dominant frequency shifted to about 5Hz. Damping ratio was about 5%.

(3) Shear Force and Shear Strain

The maximum and minimum values of shear force were extracted from the hysteresis loops obtained in representative excitation tests and were plotted as the relationship between shear force and shear strain in Figure 10 and 11.

Figure 10 demonstrate that the PCCV test model behaved linearly up to S2 test while non-linear behavior is dominant at over S2 test. The proposal skeleton curve modified the JEAG type[1] is also plotted in the figure and shows good agreement with test results. The ultimate shear stress was modified and liner strength and stiffness were considered in the skeleton.

Figure 11 demonstrate that the RCCV test model behaved non-linearly at S2 test. The proposal skeleton curve also modified the JEAG type shows good agreement with test results.

5. Seismic Margin

As the dominant frequency of test model changed and input acceleration frequency was modified for PCCV test, it is difficult to evaluate the seismic margin by the ratio of maximum input acceleration and maximum design input acceleration. In the study, the energy input which was not affected by the above frequency change was used to evaluate the seismic margin.

The energy input can be calculated by equation (1),

$$E_{in} = \int_0^t m\ddot{x}_0(\dot{x} - \dot{x}_0)dt \quad (1)$$

where E_{in} is energy input, m is mass of test model, $\ddot{x}_0$ is input acceleration, $\dot{x}_0$ is input velocity, and $\dot{x}$ is response velocity. When we applied the equation to the test results, we slightly modify the equation to take account of shake table pitching motion. The time histories of the energy input during each test, which is presented as the energy-equivalent velocity $V_E(=(2E_{in}/m)^{1/2})$, are shown in Figure 12 for RCCV tests. It can be seen that the failure of the test model during 9S2(H) test occurred at the same energy levels as V_E of 5S2(H) test.

The energy spectra of the input motions 9S2(H) and S2(H) are shown in Figure 13. As the test model fractured during 9S2(H) test, the energy spectrum of 9S2(H) is smaller than 9 times energy spectrum of S2(H). The energy input V_E=366cm/s during 9S2(H) test is about 6.6 times of the energy input V_E=55cm/s during S2(H) test, if compared regarding the same period. As slipping deformation was measured at the bottom of the shell wall, the net energy should be V_E=322cm/s, which is the gross V_E=366 minus the amount of the slip energy. Therefore, the seismic safety margin obtained from the result of 9S2(H) testing should be regarded as 5.8 times of S2(H).

As the accumulative tests from S2(H) to 9S2(H) had been conducted, seismic margin based on all the accumulative tests was able to be estimated using the accumulative energy input. In the study, the accumulative energy input was calculated by equation (2),

$$E = E_n + 0.5\sum_{i=1}^{n} E_i \qquad (2)$$

where E is accumulative energy input, E_n is energy input, and E_i is energy input of previous test.
The energy inputs of the each test are shown in Table 3, excluding the slip energy at the bottom. V_E is calculated as 481cm/s using the accumulative energy input of these tests, If a representative period of vibration through 1.1S2(H) to 9S2(H) is regarded as 0.19s, which is the mean period, then the seismic margin through all the tests is found to be 7.4 times of V_E=65cm/s of the S2(H) excitation. The same estimation method was applied to PCCV test result, and we have the value of about six times of S2(H). From these estimation results, the seismic margin of concrete containment vessel was evaluated to be about five times of S2.

5. CONCLUDING REMARKS

Seismic proving tests of concrete containment vessels were carried out focusing not only structural integrity but also functional soundness. Two type of test model, prestressed concrete containment vessel and reinforced concrete containment vessel were tested with large-scale and high-performance shake table. For PCCV and RCCV test models, the structural integrity and air-tightness was confirmed to maintain in design level up to the extreme design earthquake S2. From the results of the seismic margin test including failure test, seismic margin of concrete containment vessel was evaluated to be five times of S2.

ACKNOWLEDGEMENT

Since 1980, NUPEC has conducted a series of seismic proving tests of nuclear power plant facility under the sponsorship of Ministry of Economy, Trade and Industry (METI) of Japan. The seismic proving test of PCCV and RCCV are carried out as one of the seismic proving tests. The authors would like to acknowledge the advice of the steering committee of NUPEC.

REFERENCE

[1] Y.J.Park and C.H.Hofmayer, “Technical Guidelines for Aseismic Design of Nuclear Power Plant (Translation of JEAG 4601-1987),” NUREG/CR-6241, 1994.

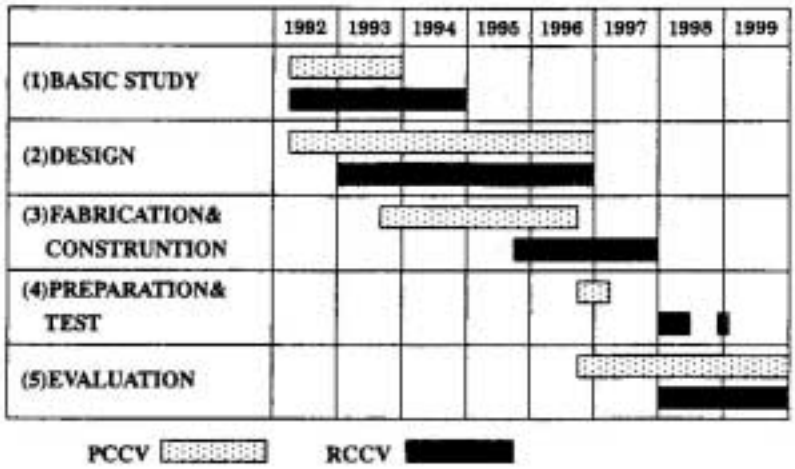


Figure 1 CCV Seismic Proving Test Schedule

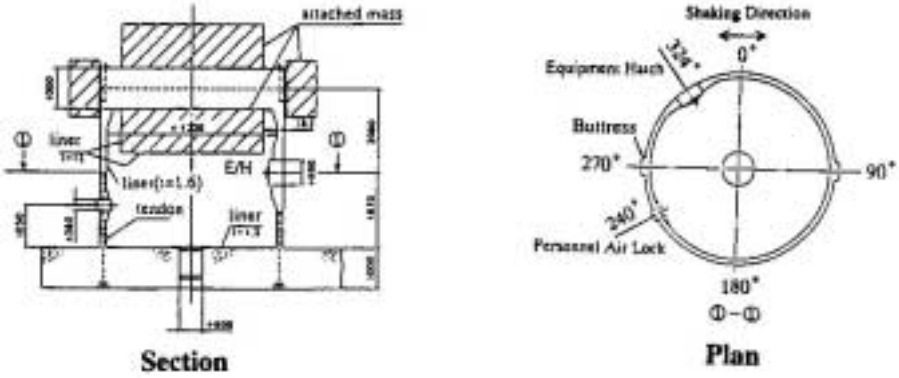


Figure 2 PCCV Test Model Configuration

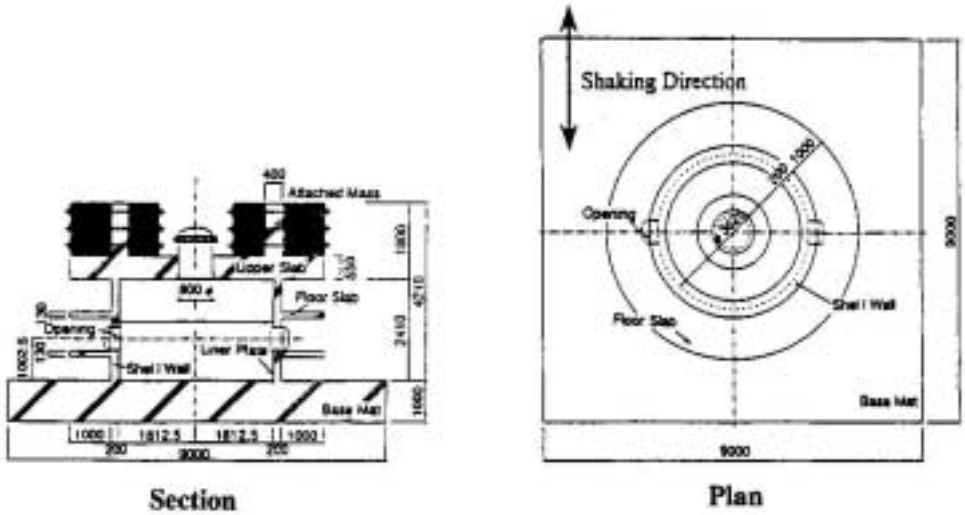


Figure 3 RCCV Test Model Configuration

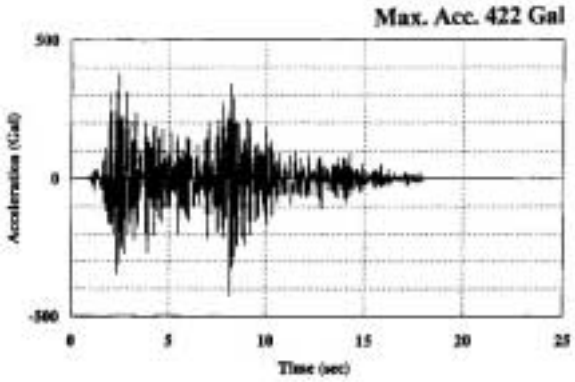


Figure 4 Time Fistory of S2 Input Motion For PCCV Test Model

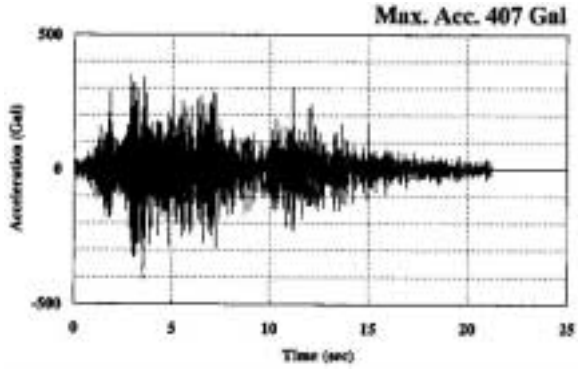


Figure 5 Time History of S2 Input Motion For RCCV Test Model

Table 1 Response of PCCV Test Model

Test No.	Test Name	Direction of Excitation	Maximum Input Acc. (Gal)	Maximum Response at Top Slab Acc. (Gal)	Disp. (mm)
P1	S1(H)	Horizontal	354	981	2.20
P2	S1(H+V)	Horizontal	299	1162	2.78
		Vertical	139	347	0.08
P3	S2(H)	Horizontal	419	1362	3.34
P4	S2(H+V)	Horizontal	411	1529	3.94
		Vertical	226	786	0.17
P5	LOCA+S1(H+V)	Horizontal	289	1012	2.73
		Vertical	157	600	0.16
P6	2.0S2(H)	Horizontal	1414	2650	12.60
P7	3.0S2(H)	Horizontal	2680	3300	19.60
P8	2.7S2(H)	Horizontal	2500	3030	18.43
P9	3.3S2(H)	Horizontal	2510	3010	18.22
P10	4.0S2(H)	Horizontal	2970	3340	21.9
P11	3.9S2(H)	Horizontal	2510	2980	20.2
P12	5.0S2(H)	Horizontal	3280	3120	26.0

Table 2 Response of RCCV Test Model

Test No.	Test Name	Direction of Excitation	Maximum Input Acc. (Gal)	Maximum Response at Top Slab Acc. (Gal)	Disp. (mm)
R1	1.3S1(H)	Horizontal	562	996	2.524
R2	1.1S1(H)	Horizontal	276	793	2.672
R3	1.1S1(H+V)	Horizontal	278	1027	3.455
		Vertical	351	759	0.715
R4	1.1S2(H)	Horizontal	448	1495	5.547
R5	1.1S2(H+V)	Horizontal	508	1610	6.918
		Vertical	632	1496	1.839
R6	L+1.2S1(H+V)	Horizontal	387	872	4.808
		Vertical	260	656	0.849
R7	1.0S2(H)	Horizontal	417	1002	4.444
		Vertical	415	746	0.963
R8	1.0S2(H)	Horizontal	409	1001	4.386
		Vertical	407	731	0.946
R9	1.0S2(H)	Horizontal	412	989	4.361
		Vertical	411	731	0.941
R10	1.0S2(H)	Horizontal	409	1014	4.373
		Vertical	411	729	0.943
R11	2S2(H)	Horizontal	870	2007	10.63
R12	3S2(H)	Horizontal	1568	2654	17.33
R13	4S2(H)	Horizontal	1771	2857	19.73
R14	5S2(H)	Horizontal	2149	3076	26.10
R15	9S2(H)	Horizontal	3931	3626	44.91

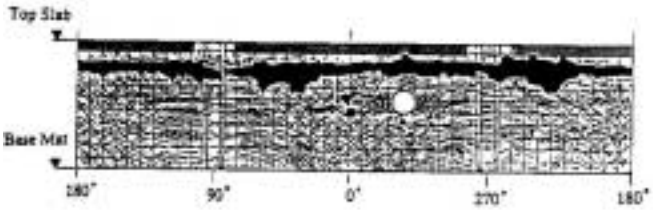


Figure 6 PCCV Crack Pattern after Failure

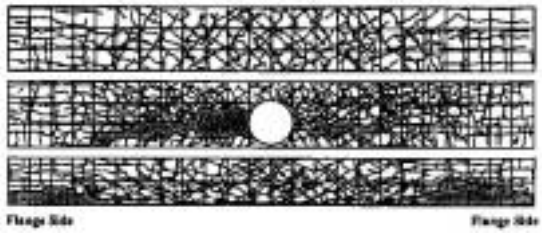


Figure 7 RCCV Crack Pattern after Failure

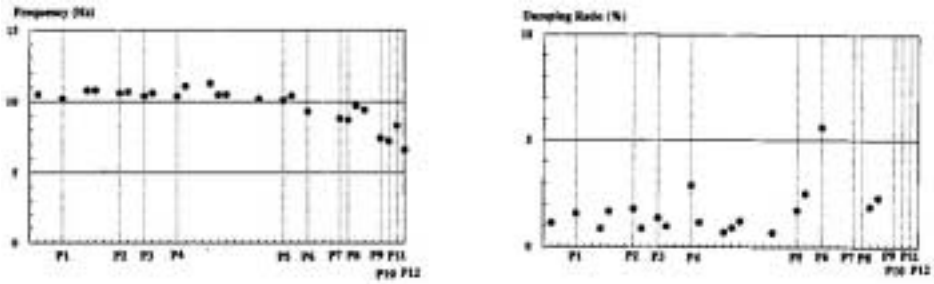


Figure 8 Frequency and Damping Ratio of PCCV Test Model

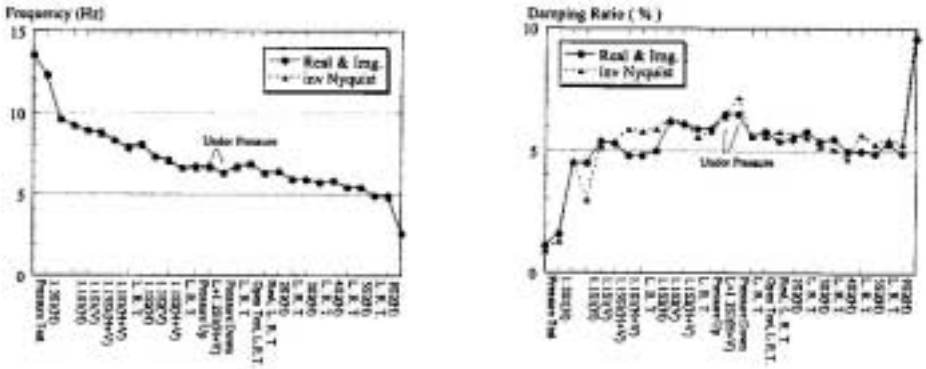
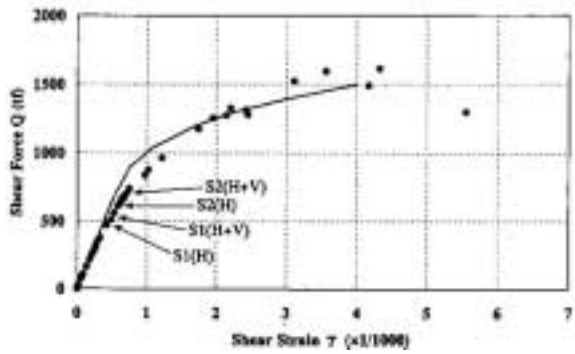


Figure 9 Frequency and Damping Ratio of PCCV Test Model



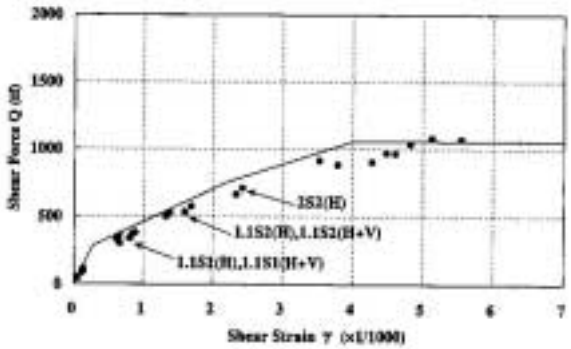


Figure 10 Shear Force and Shear Strain of PCCV **Figure 11 Shear force and Shear Strain of RCCV**

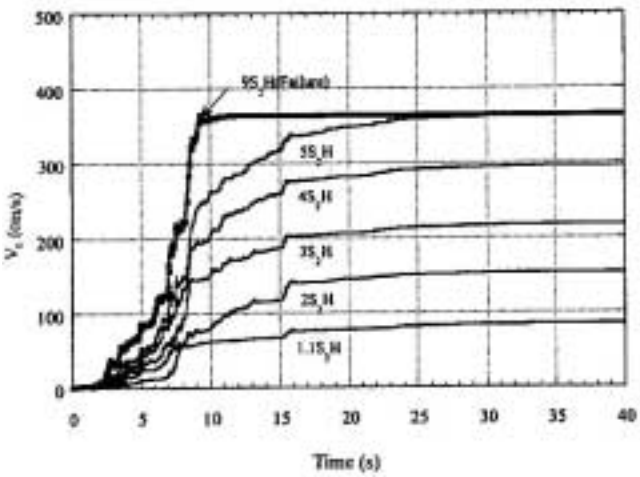


Figure 12 Energy Input Time History

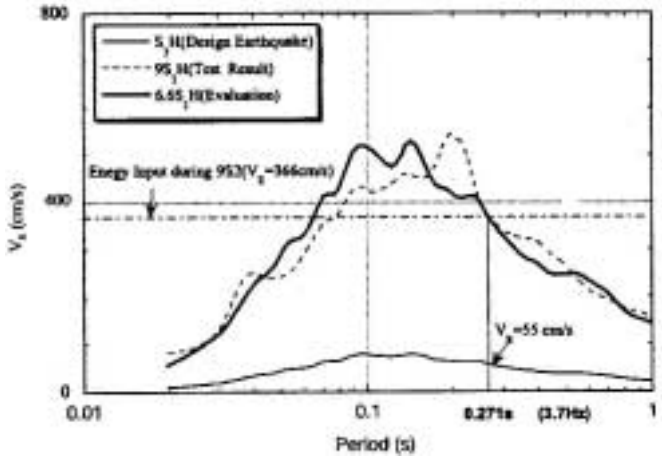


Figure 13 Energy Spectra of Input Motions 9S2(H) and S2(H)

Table 3 Energy Input without Slip of Shell Bottom

	1.1S2(H)	1.1S2(H+V)	2S2(H)	3S2(H)	4S2(H)	5S2(H)	9S2(H)
Total Energy Et [kNm]	121	165	394	779	1452	2218	2344
Slip Energy Es [kNm]	0	0	0	0	206	413	535
Et-Es [kNm]	121	165	394	779	1246	1805	1809
$V_E=(2(Et-Es)/m)^{1/2}$ [cm/s]	83	97	150	211	267	321	322

Seismic reevaluation of PHENIX reactor

P. Sollogoub CEA/SACLAY/EMSI, D. Goux Centrale Phenix, C. Duval
EDF/Septen, E. Gallitre EDF/ CLI

1 INTRODUCTION

PHENIX is a 250MWe fast breeder reactor located in Marcoule center along the Rhone river. The plant was built in the beginning of 70's, and got critical in 1973. It has been designed according to applicable seismic codes at that time.

Due to the need to continue the operation of the plant for research programs, Safety Authorities required the verification with present methods, that the essential safety functions are fulfilled for the level of earthquake defined according to the present methodologies .

The paper presents the different steps of the re-evaluation process for structures which started in 1996: general organization, methodology, general principles taken for the seismic assessment and re-evaluation, definition of the upgrades and works performed.

All the seismic upgrades are under completion and the plant is planned to re-start next year.

2 Brief description of the Plant

2.1 General

PHENIX is an experimental fast breeder reactor of pool type designed during the 60's, installed in the CEA center of MARCOULE along the Rhone river in the south part of France and is operated jointly by CEA (80%) and EDF (20%). The electric power is 250MW. The construction started in 1968 and the reactor got critical in 1973. From seismic point of view, the plant has been initially designed according to the existing codes for an earthquake level defined by the intensity level of 8 for the "Safe Shutdown Earthquake". Later on, following a modification of the operation of the plant, Safety Authority required to verify, with present methods, that the essential safety functions are fulfilled for an earthquake level defined by two response spectra. One, with a ZPA of 0.15g represents the far field sources and the second , with 0.2g, the near field.

2.2 Building description

All the buildings are on a north-south line; the ground level is 0m (figure 1). Main structures are:

The Reactor Building (RB), with main dimensions, 42m length, 26m width and 49m height, with 14m embedment (figures 5 and 6). The infrastructure, from the raft to the 8.5m elevation is a wall and floor

reinforced concrete structure; common with the North Handling Building infrastructure. It supports the reactor vessel and associated systems. The superstructure is made of concrete column and steel beams for the roof. Columns are linked by horizontally prestressed concrete plates. The columns and the plates are prestressed by vertical tendons for confinement.

The Steam Generator (SG) Building (L=42.7m, l=41m, H=43m) is founded on an independent raft with reinforced concrete wall and slab infrastructures and steel superstructures with light boarding (figure 2). It contains steam generators and secondary loops in its south part and a handling space for SG in the north.

The Turbine Building – TB - (L=52.m, l=42m, H=42m) is located in the north. It is a reinforced concrete frame, with walls and steel superstructure (figure 3). It has a turbine on a separate foundation. One row of superstructures of the SG building is founded on TB infrastructure.

The North Handling Building (NHB), which dimensions are comparable to the Reactor Building ones (figure 4), is a steel frame above the infrastructure enclosed with concrete panels with horizontal mortar slice between two panels.

Other buildings are present on the site: along the RB on the west side, auxiliary building, in the south, handling buildings. Parallel, the control room and office building and at the north east, the pump building.

All the structures are founded on a stratified alluvial soil with a slight slope to the North; the mean Young modulus is 1600Mpa.

3 Methodology of seismic reevaluation.

For this task, which is not standardized, the owner asked a Group of Experts to assist it in order to propose methodology and hypotheses of the reevaluation. FRAMATOME/NOVATOME was in charge of the general coordination and Sechaud & Metz was the Civil Engineering consultant and EDF-CLI (Engineering group in Lyon) was in charge of the control.
The different steps of the work where:

3.1 Definition of the safety function of each building and associated behavior requirements.

The earthquake is considered as an “accidental” situation and only safe shutdown, decay heat removal and limitation of radioactive release is required.

The building functions are then Capacity of Supporting safety related equipment and components (RB and NHB infrastructures, SG building, some auxiliary buildings), support of potential missiles (RB superstructure) and Stability (non collapse) for almost all the other structures.

3.2 Assessment of the “as-is” situation

This is required in order to identify the characteristics of the as-built installation, to appreciate its ageing, to check the conformity of drawings with the actual status of the plant, and to be sure of the compatibility between analysis hypotheses and the actual situation of structures. To illustrate this latter point, we may mention the detailing of concrete reinforcement in order to accept some inelastic behavior, the lap splices and the behavior of steel and concrete connections.

This task has been performed by analyzing the drawings, by walkdowns , by non destructive and destructive tests … As examples, we can mention: gaps cleaning and measurement between the different structures, friction coefficient measurement in the mortar between panels, prestressing tendons heads radiography and so on.

3.2 Seismic assessment and reevaluation general principles

Seismic assessment of an existing structure is different from the design of a new structure. In the latter situation, the design principles, based on analysis and detailing practices, induce margins in almost all the steps, due to uncertainties in the design, construction and life of the structure and to absolutely necessary simplifications in the overall process. For assessment, there is no fixed engineering practice and it is necessary to perform a critical evaluation of the analysis methods, detailing practices and criteria, in order to have an as realistic as possible situation. For this, examination of post earthquake and seismic test data was intensively used in order to help the decision making, as well as the use of conventional seismic codes such as PS92 and the Recommendations AFPS90, edited by the French Earthquake Engineering Association (AFPS). This was the main task of the Group of Experts.

Examples of critical evaluation of methods of analysis and criteria are given below.

For definition of loads, it has been suggested to consider differently far field and near field (shallow focus) earthquakes because the elastic spectrum is not a sufficient representative of the damage induced by the earthquake: duration and number of cycles play an important role. For the near field case, it has been proposed to have a reduction coefficient which modulates the elastic response.

In the load combinations, thermal loads have been excluded because they are an applied displacement which does not modify the limit load of the structure. As a complementary margin, they have been considered for the SG building.

One usual way of analysis is to have a simplified beam model for dynamic spectral analysis and, in a second step, to apply accelerations deduced from the spectral analysis where they are combined from each mode, to a 3D finite element model. This approach is not realistic and always conservative; it has been suggested to define loads in the 3D model starting from acceleration calculated by dividing forces by masses.

For structures for which only stability is required, the Group considered that some limited plastic deformation is acceptable. Conditions for this and ways to apply in verification was proposed and achieved in Turbine and Control buildings. In relation with this, some code requirements concerning the minimum longitudinal steel ratio and the lap splices was extended.

The seismic analysis is based mainly on linear models with spectral analysis on beam type models and code verification on 3D finite element models. The configuration of structures is such that “local” panels (walls or slabs) vibration modes may be excited by earthquake input signal. The Group proposed a procedure to introduce the first few local modes in the beam model in order to have a comprehensive modal behavior of structures. Limited non linear analyses with friction between panels, was used to enhance the assessment. Soil structure interaction was considered; the stratigraphy of the site make necessary to adapt the conventional impedance value, mainly for damping which is decreased compared to homogeneous soil.

4 Definition of upgrades

They are the result of the assessment and they can be grouped in the following manner:

Opening of joints between structures in order to avoid impact during earthquake, for RB NHB and TB.

Separation of buildings: the roof of SG building has been separated from the TB one in horizontal directions in order to avoid distortions of the steel frame.

Reinforcement or repair in order to achieve the needed resistance or to avoid fragile parts (figures 5 and 6): tying of the upper part of the RB, strengthening of lap splices with steel jackets in RB and TB, longitudinal reinforcement in RB columns, fixing of the panels in RB and NHB, upgrading of steel structures (in NHB, SG, TB) by adding braces and improving connections, and reinforcement of anchors; in the TB, reinforcement steel has been added in order to improve the local resistance.

5 Conclusion

The seismic re-evaluation of PHENIX plant is a very important task which needed a dedicated organization, which includes an Expert Group to define the overall methodology, which has been successfully applied. It is based on the fact that re–evaluation of an existing structure is different from the design of a new one; all the methods and criteria must be critically examined.

The methodology was approved by Safety Authorities. Works took place essentially during 1999 and 2000 as part of an overall upgrade program. Today, they are almost completed. The overall cost of the seismic project is about 30 millions Euros.

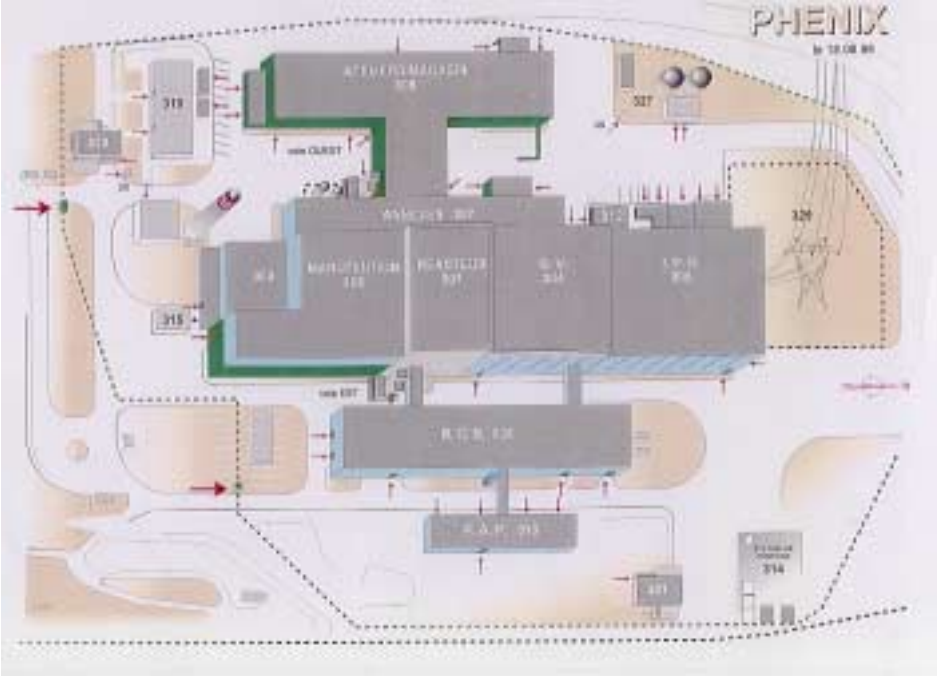


Figure 1 : General view

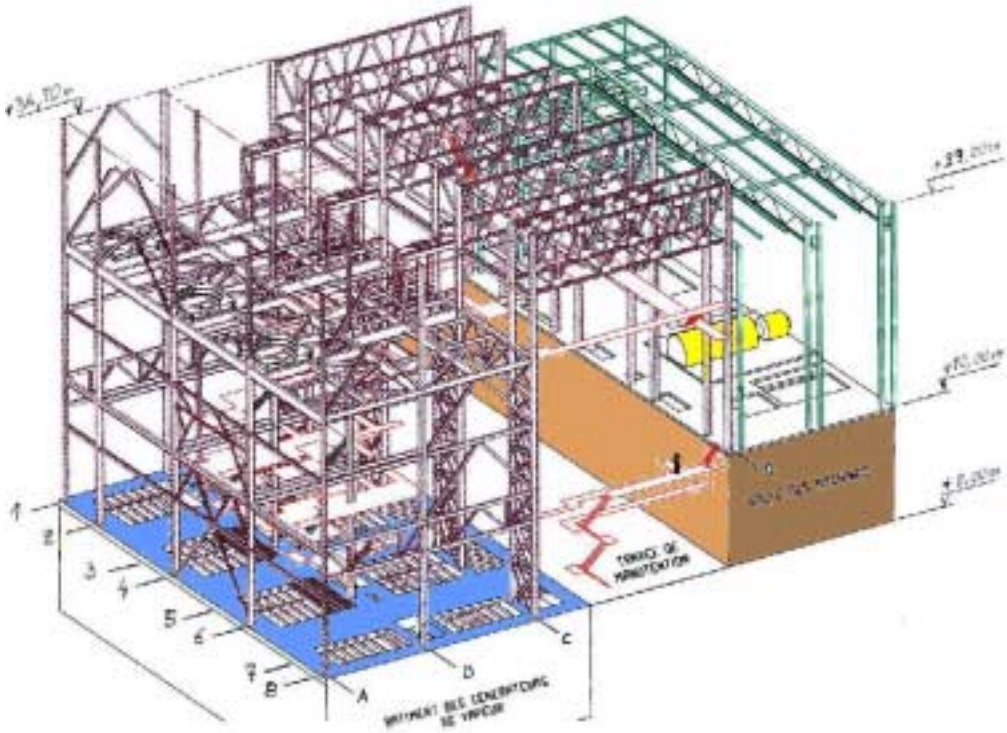


Figure 2 : Steam generator superstructure

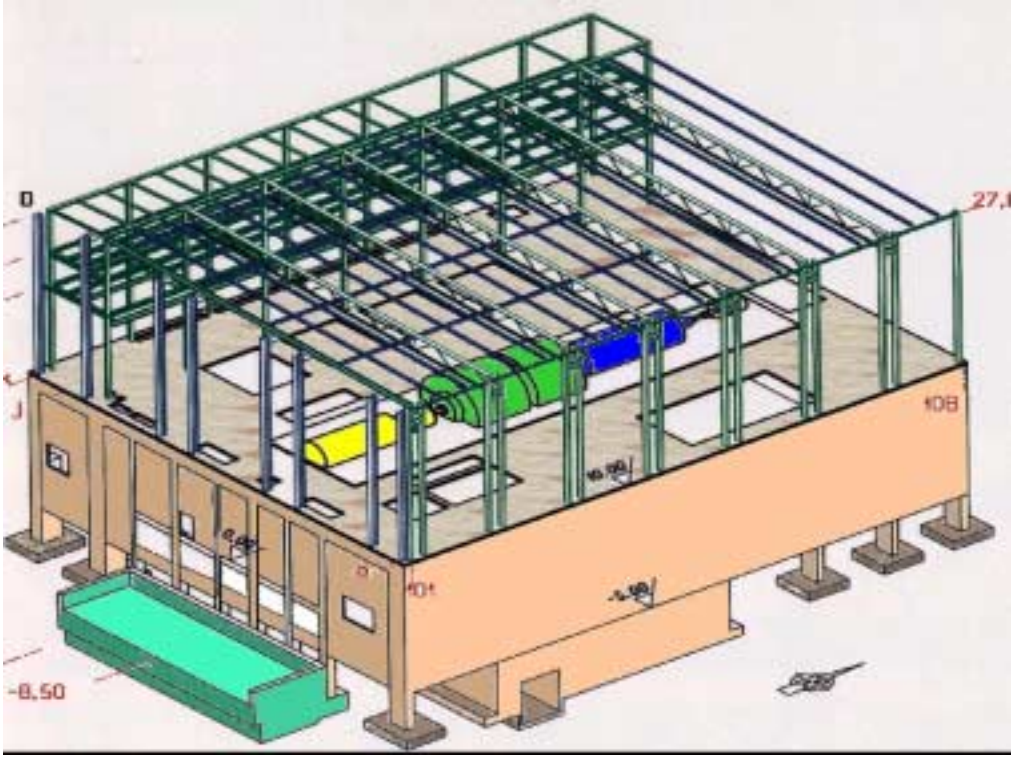


Figure 3 : Turbine building

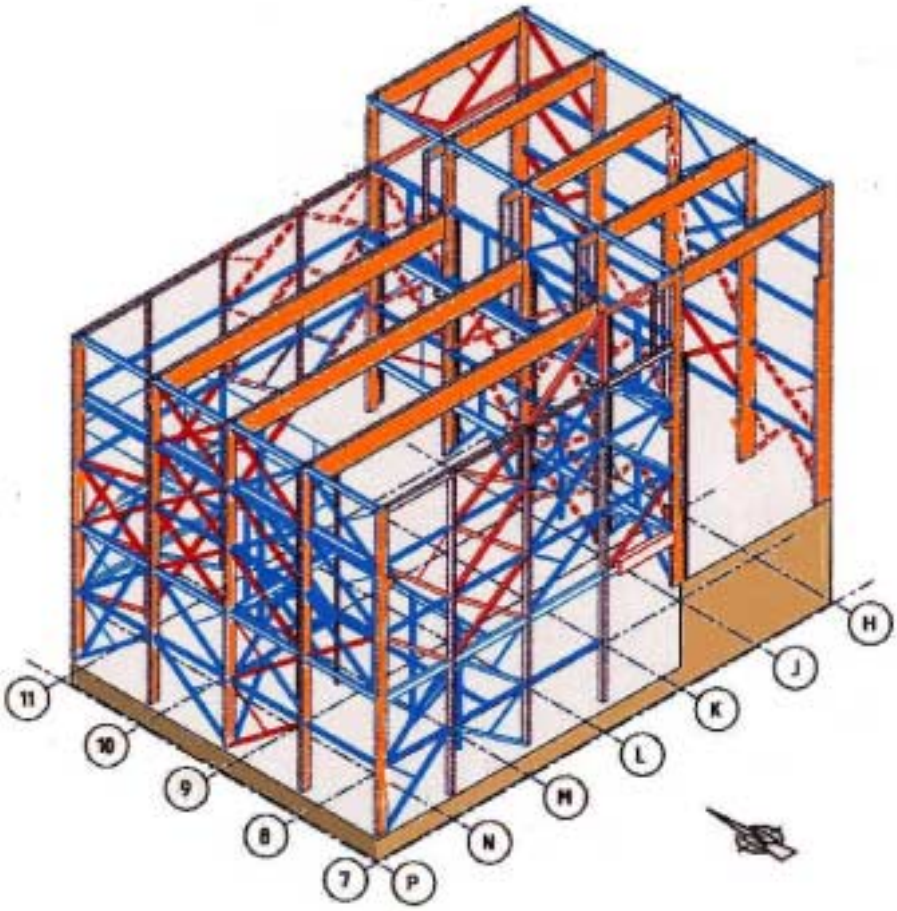


Figure 4 : North handling building superstructure

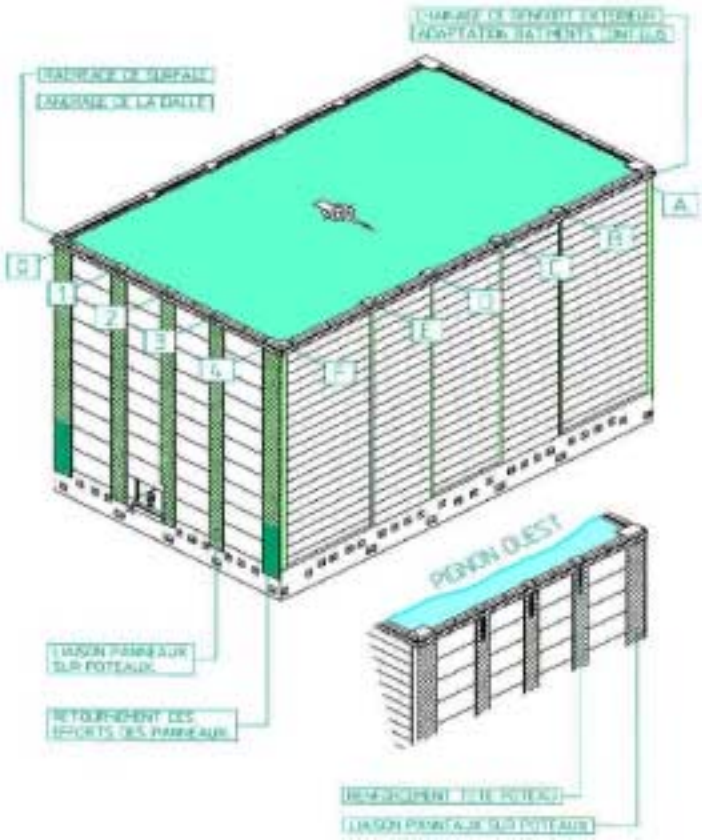


Figure 5 : Reactor building external upgrade

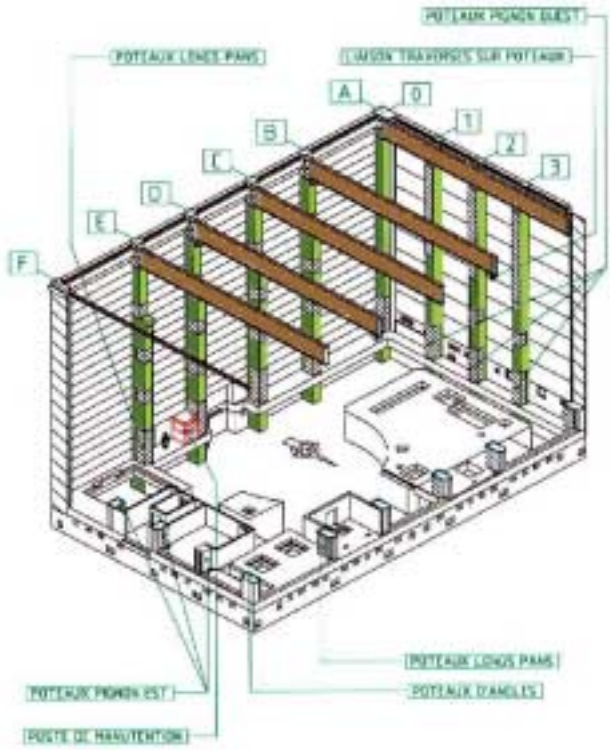


Figure 6 : Reactor building internal upgrade

SEISMIC RE-EVALUATION OF THE PHENIX REACTOR BUILDINGS POSITION OF THE NUCLEAR SAFETY AND PROTECTION INSTITUTE (IPSN)

MARC BOUCHON (IPSN)

1. CONTEXT

The layout and design of the buildings at the PHENIX power plant at Marcoule were based on the rules and methods that were current at the time of its construction in 1970.
Since then, changes have occurred - some of them major - in relation to current methodology, notably due to:

- a change in seismic activity, and to the rules and layout methods employed;
- unsuitable design and construction practices leading to a low ductility.

Consequently, the aim of this seismic reappraisal is to evaluate the behaviour of the buildings at the PHENIX power station when subjected to the prescribed level of seismic activity and to define the improvements that must be implemented in order to guarantee the safety of the facility.
After identifying the discrepancies between the initially-used layout procedures and those employed today, and reviewing the safety requirements for each building, we will present the position of the safety authority with regard to the principles of the seismic reappraisal of the buildings at the PHENIX power station.
The findings for the reactor building will then be presented.

2. IDENTIFYING THE DISCREPANCIES BETWEEN THE INITIALLY USED LAYOUT PROCEDURES AND THOSE EMPLOYED TODAY

The PHENIX reactor buildings were laid out in accordance with earthquake-resistant rules (PS69) that were applicable to buildings at that time and which were designed to withstand seismic activity measuring 8 on the MERCALI scale.
These rules lead to statisticic protection and implicitly assume an intrusion into the plastic area, through the definition of a fixed inelastic seismic movement. Furthermore, these rules do not take ground-structure interaction into account.
Since this time, design rules and methods applicable to nuclear facilities have been laid down, in particular the “Fundamental Safety Rules” (RFS) which “aim to clarify the conditions whose observance is, for the type of installation under consideration and for the purpose they are intended to serve, judged to be in compliance with French regulatory practice”. These

fundamental safety rules are complemented by further rules on design and construction.
The discrepancies between the initially used procedures and those currently in force are quite significant. Thus, the level of seismic stress that can be withstood may be higher by a factor of 3 or 4 than that taken into account at the time the layout was implemented.

Furthermore, the design of the buildings includes a number of distinctive features that do not favour good seismic performance or make use of the ductility of structures. The most important of these distinctive features are as follows:

- The opening of the separation joints between the various buildings is not sufficient to avoid an interaction in the case of seismic activity;
- there is a design discontinuity between the reinforced concrete infrastructure, braced by plates, and the frame-type superstructure in either concrete or steel;
- the foundations for the handling gantry (between the steam generator building and the machine-room building) and those for the machine-room building are footing or raft-type foundations, at different base levels;
- the steel framed northern storage hall comprises prefabricated panels in 15cm-thick reinforced concrete on the walls, connected to the vertical sections of the frame. The roof is reinforced concrete cast onto steel containers and connected locally to the frame sections.
  The behaviour of this structure is very difficult to predict, as part of the stress travels through the frame and part through the wall panels;
- the reactor hall is a mixed structure consisting of pre-stressed concrete posts connected at the top by metal beams; the roof is reinforced concrete, cast on steel sheets and the walls consist of prefabricated concrete panels pre-stressed with embedded tendons, and connected to the posts.

3. SAFETY REQUIREMENTS ASSIGNED TO BUILDINGS

The safety requirements assigned to buildings depend partly on the safety functions allocated to them and the equipment with safety implications that they contain, and partly on the potential threat they pose to buildings with a safety enforcement role.
The safety requirements are as follows:

- stability;
- protection of important safety equipment;
- protection of equipment that must not itself damage important safety equipment;
- absence of harmful interaction between buildings that perform no safety enforcement function and ones that do.

Below is a reminder of the safety requirements assigned to each building (see figures 1 and 2):

- reactor hall, northern handling hall and steam generators building: stability, protection of important safety equipment and protection of equipment that must not be projected;
- southern handling hall and turbine room buildings : sufficient stability to avoid interaction with the adjacent buildings, which could impair the safety functions of these buildings.

4. SEISMIC RE-EVALUATION PROCEDURE

In the case of the PHENIX power plant, the seismic reevaluation procedure takes account of the operational lifespan when deciding upon seismic activity, the reliability of methods, and - given the design, the construction procedures and the quality of the work – the possibility of an intrusion into the plastic area.

Seismic motions

The seismic motions should generally be determined by application of the method in Fundamental Safety Rule RFS 1.2.c (1981). By taking as its base seismic-type accidents and tectonic areas on one hand, and historical seismic data on the other, one or more "Maximum Credible Historical Earthquake" (SMHV) is defined for the site concerned. For each SMHV, an "Safety Earthquake" (SMS) is defined and the SMHV is then deducted from it, in terms of MSK intensity at the site, using the following simple relationship:

$$I_{SMS} = I_{SMHV} + 1 \text{ (in MSK units of intensity)}$$

SMHV are earthquakes likely to produce the most severe effects on the site.

SMS are considered to be the most severe seismic activity that could affect the facility concerned.

In addition, beyond the SMHV, there should be no cliff effect to avoid unacceptable consequencies.

As the operational life expectancy is limited, the building behaviour reevaluation has been agreed at a level of seismic motion corresponding to the most suitable near and far "Maximum Plausible Historical Earthquake" (SMHV) for the Marcoule site. These are defined in terms of horizontal direction by response spectra with respective cut-offs of 0.15g and 0.2g at infinite frequency for near and far earthquakes (see figure 3).

Verification methods and criteria

Given the absence of directives relating to the seismic reevaluation of existing nuclear facilities and with the aim of using margins in order to obtain a certain guarantee that safety requirements will be observed for seismic motions higher than SMHV, the IPSN has requested that the procedure followed should be close to a design procedure.

Consequently:

1.analysis methods will continue to be linear. Seismic activity is defined by the response spectra that distinguish SMHV.

Soil-structure interaction is modelled using impedance functions that represent the stratification of the ground around the foundations. Dynamic and pseudo-static calculations are based respectively on a simplified model of the building and on a three-dimensional finite elements model.

For buildings with no safety role that must not damage safety-classed buildings within the nuclear island, an intrusion into the plastic area is considered a possibility through a coefficient of reduction of the elastic stress due to the earthquake. The value of this coefficient is a function of the type of construction and procedures used (capacity for ductile behaviour), and it varies between 1.5 and 2. Thus in the case of the steel-framed halls of the turbine and southern handling buildings, the value of the coefficient of reduction is taken to be equal to 2.

3.Verification criteria for the various structural elements in concrete or steel rely on the technical rules currently in force, relating to the design and calculations for reinforced concrete and steel construction.

Interaction

The displacements of each building at each level are deduced from linear calculations that take account of the reduction in stiffness of reinforced concrete structures, due to cracking when subject to seismic activity.

Collisions between elements of superstructure are unacceptable.

Consequently, it is essential to increase space between adjacent structures where this is insufficient.

5. EXAMPLE OF REACTOR BUILDING HALL

Description

The reactor-building hall is anchored to the reinforced concrete infrastructure shared by the reactor building and the northern handling building. The dimensions of the hall are 43.20 m in width (east-west), 26.20 m in length (north-south) and 26.60 m in height (see figures 4 and 5).

The walls of the hall are made up of 18 columns of partially pre-stressed concrete and 280 prefabricated panels attached to these posts.

The roof is made up of a steel framework, covered by sealing sheeting, which supports a slab of reinforced concrete 15cm thick. The main cross-members of the framework are steel girders with a span of 24.36 m in a north-south direction, and joined to the wall posts by plate welds. The reinforced concrete slab is secured to connectors welded to the upper part of the girders (see figure 6).

Studies carried out

The operators have carried out several dynamic surveys to evaluate the behaviour of structural elements of the reactor building hall, using spectra that represent near and far earthquakes.

Several hypotheses have been considered, relating to the following parameters:

- the characteristics of the soil and the modelling of soil-structure interaction;
- the value of the pre-stressing force in the vertical cables (72t to 87t) that ensure vertical compression of the wall panels;
- the value of the friction coefficient between the prefabricated panels varies between 0.6 and 0.9.

The IPSN has produced a series of calculations, in order to study the sensitivity of results, given variations in the above-mentioned parameters, to identify the influence of local factors in the walls and roof, and to analyse the resistance of those components under the most strain.

In addition, several inspections have been carried out on site to verify the quality of the work, in particular the connection between the columns and prefabricated panels, and the state of the joints between the panels.

For a near earthquake, the operators initially used a stress reduction coefficient of 1.5. Given the design of the hall and the presence of low-ductility connections between structural components, the IPSN requested that the calculations be performed again without using this coefficient.

Furthermore, in the light of the results of the inspections and feedback received, the IPSN accepted a friction coefficient between panels of 0.8, combined with a pre-stressing force of 80t per cable.

Proposed reinforcement solutions

Following these studies, the operators proposed the following reinforcement solutions, in order to ensure the stability of the reactor building hall in case of an earthquake:

- reinforcement of the connection between the roof slab and the wall sections, including the addition of a peripheral beam (see figure 7) ;
- addition of vertical re-bars on the intermediate columns of the connected to the interior facing, between levels +21.30 and +29.00;
- overlap zone confinement for the vertical re-bars of the columns, through installation of a sheet-metal jacketing;

The IPSN recognised the validity of these reinforcements and requested that the transverse reinforcement of the vertical bars overlap zones be extended to cover the hall corner columns.

6. CONCLUSION

Below is a reminder of the seismic reevaluation procedure for the PHENIX reactor buildings:

- as the operators have undertaken to cease operations in some years, seismic motions considered are that for “Maximum Credible Historical Earthquakes” (SMHV);
- analysis methods are linear; verification of structural elements is carried out according to criteria in compliance with technical rules on design and calculations for buildings using reinforced concrete and metal;
- the reinforcements to be implemented are laid out by applying the same rules;

The procedure followed is similar to a layout procedure, in order to avoid a sill effect, when exceeding the SMHV.

Furthermore, the IPSN has begun to give thought to drawing up directives relating to the seismic reevaluation of existing nuclear installations. These would aim to define a methodology that would enable the behaviour of buildings subject to earthquake to be better understood, including the possibility of intrusion into the plastic area. These directives could benefit from the contribution of methods developed in this field in the United States.

For this reason, the IPSN has undertaken a research programme with the University of Berkeley in order to find out more about the validity of these methods and the extent of their field of application.

ILLUSTRATIONS

Figure 1: View of all buildings of the PHENIX reactor

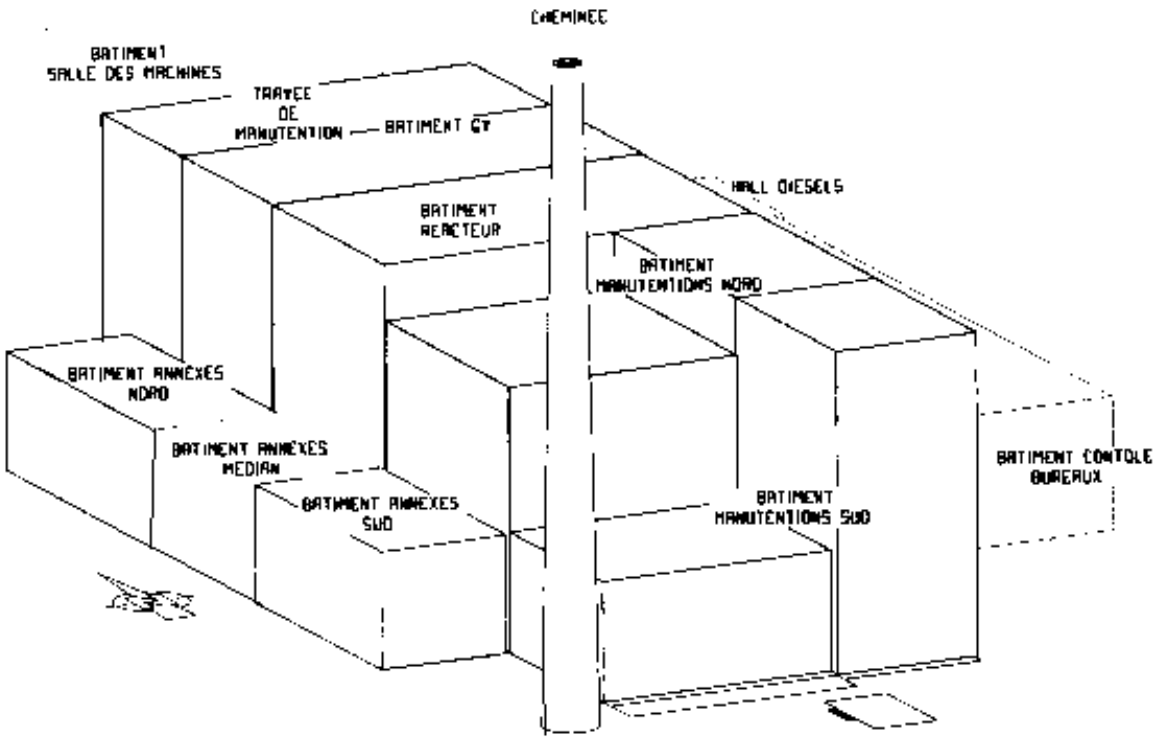


Figure 2: Longitudinal elevation

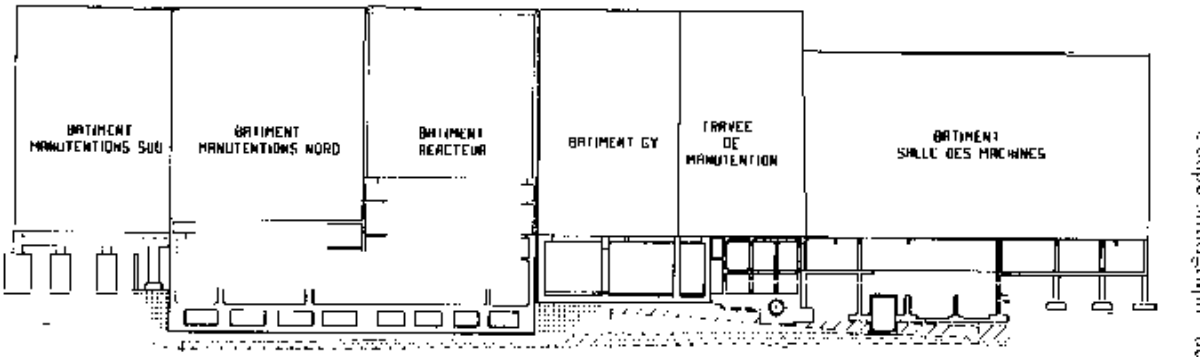


FIGURE 3: Ground response spectra

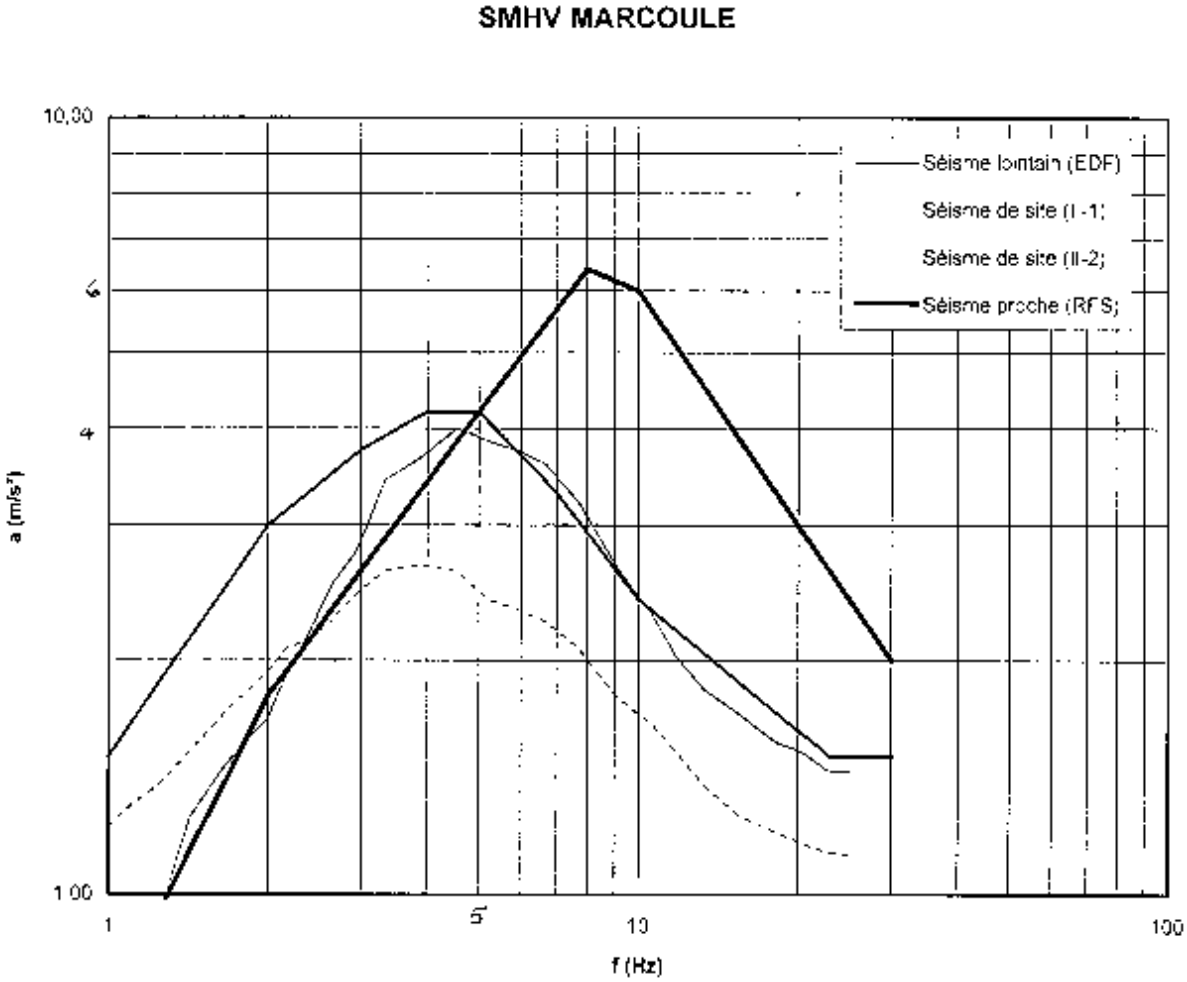


Figure 4 : Reactor building hall

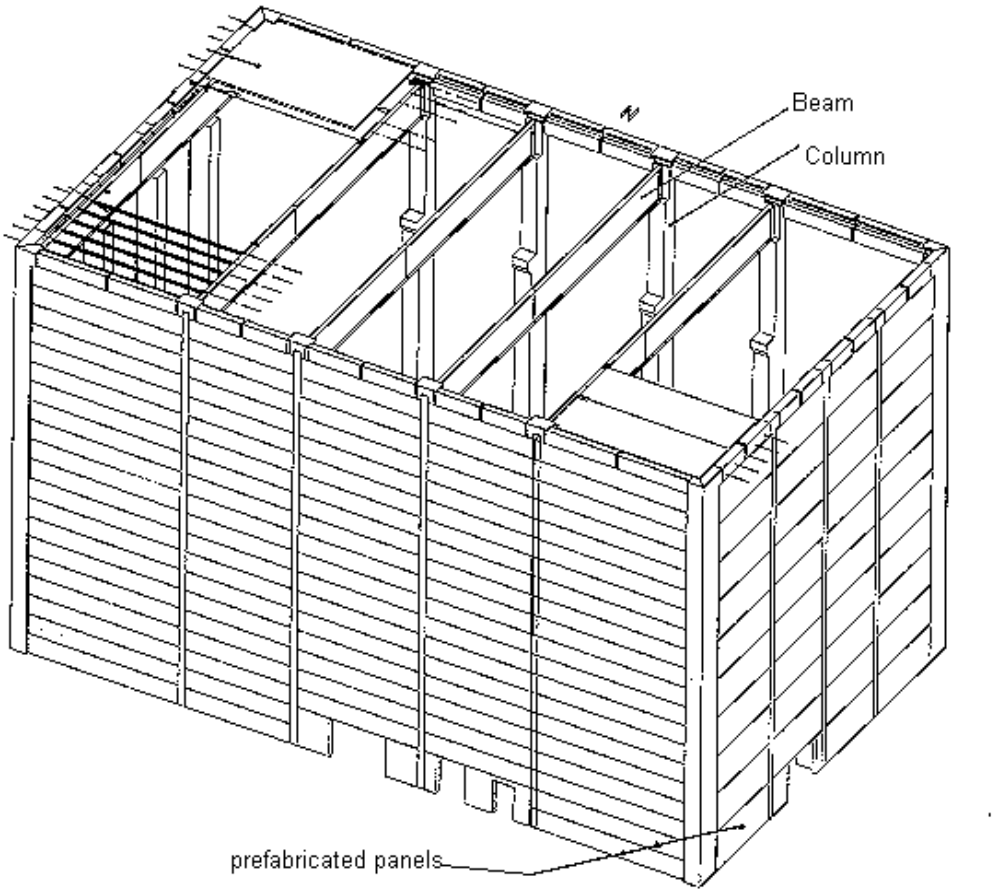


Figure 5: Close-up on panel connections

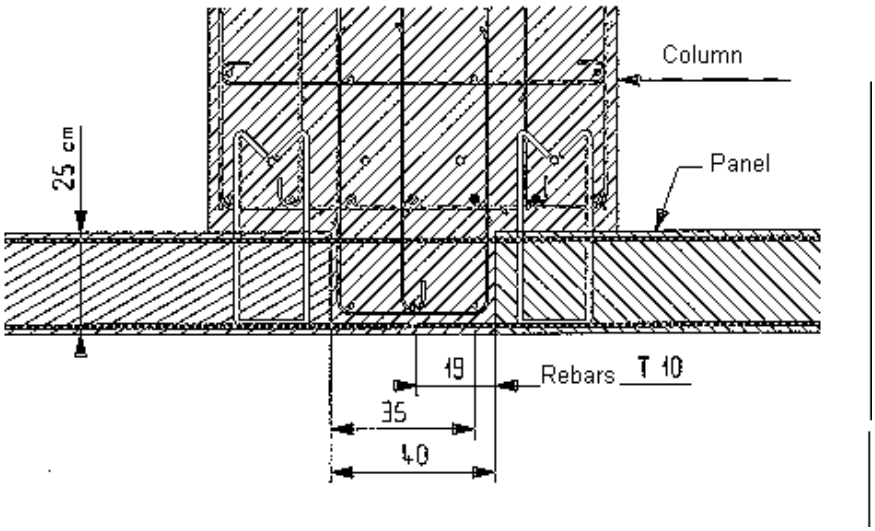


Figure 6: Reactor building hall
Close-up on roof framework

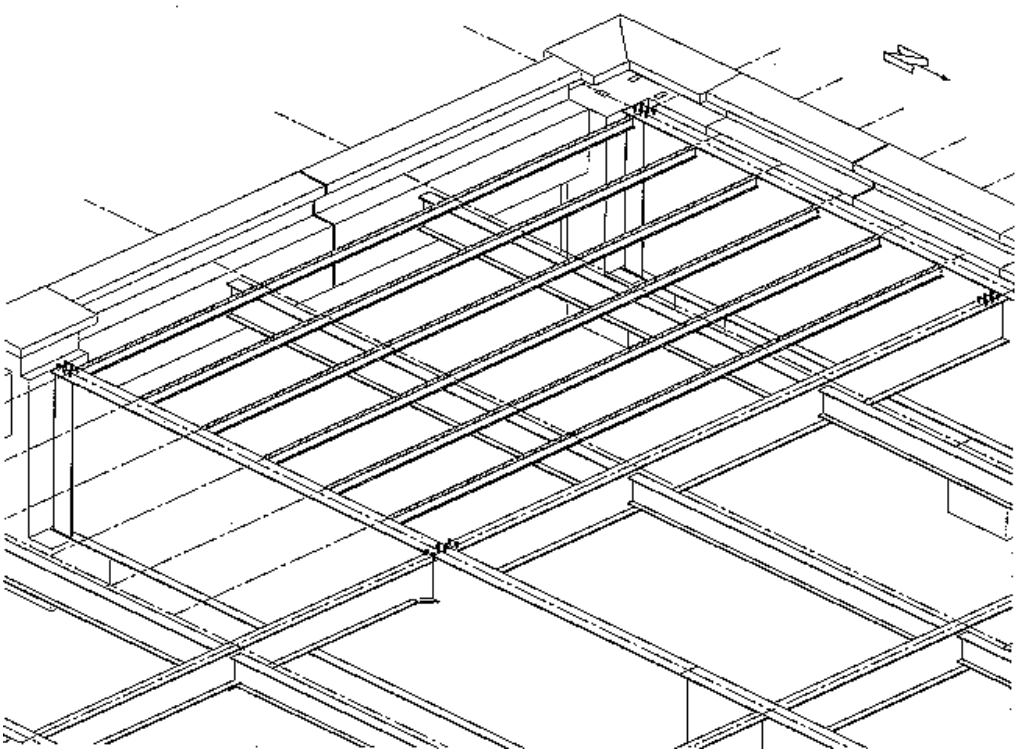


Figure 7 : Principle of roof reinforcement