IAEA-TECDOC-1655

Non-linear Response to a Type of Seismic Input Motion

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NON-LINEAR RESPONSE TO A TYPE OF SEISMIC INPUT MOTION

INTERNATIONAL ATOMIC ENERGY AGENCY
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FOREWORD

Over the past three decades, a few nuclear power plants have experienced earthquake ground motions. In more recent years a number of nuclear power plants, mainly in Japan, have been affected by strong earthquakes. In some cases, the measured ground motions have exceeded the design or re-evaluation bases.

The experience from these events shows that operating plants were shut down immediately following the event and remained shut down for extended periods while comprehensive studies, investigations, and evaluations were conducted assessing their safety. In most cases, no significant damage was identified in these nuclear power plant units. In limited cases, upgrades were implemented to meet new definitions of the design basis or requirements for beyond design basis earthquakes.

In this context, the consideration of near-field input ground motions generated by low-medium magnitude earthquakes has received special attention by the nuclear structural engineering community. Experts have identified that this type of input ground motion have minimal damage potential to engineered structures, observation confirmed by the performance of buildings that experienced such events. The fact that usual practices of earthquake engineering result in a poor estimate of their damaging effects was indicated in 1997 by the OECD/NEA/CSNI as 'the most significant issue' in the field of engineering characterization of seismic input motion. It was concluded that both the conventional description of seismic input motions in the form of response spectra and the associated conventional engineering practices were not appropriate to address and resolve the identified issue. Thus, there was a need for formulating specific and detailed criteria and procedures for addressing these situations.

As response to that need, the IAEA organized a Coordinated Research Project (CRP) on the "Safety Significance of Near-field Earthquakes' consisting of two main steps: (a) to perform a benchmark study using the testing by French organizations of a structural six storey conventional shear wall model, which includes the analytical modelling, the assessment of the predicted behaviour when subjected to two recorded motions from Japan and the sensitivity studies on the impact on floor response spectra of nonlinear structure behaviour; and (b) to propose alternative seismic design procedures to better represent the effects of these non-damaging events on engineered structures and their design on the basis of the results from the benchmark studies obtained in the first step and the earthquake engineering expertise of the CRP participants.

Twenty-two institutions from eighteen Member States were involved in the IAEA CRP, which was jointly funded by the IAEA and the European Union (The Joint Research Centre - Ispra) and was implemented in the period 2002-2006.

This report documents the entire CRP process and presents the results obtained by the participants which were evaluated, treated statistically and interpreted. All data and their evaluation are documented herein, including the outline on the need for additional research and development.

At the time this report was being completed, in July 2007, the Niigataken Chuetsu-oki earthquake occurred in Japan affecting the biggest nuclear power plant in the world, the Kashiwazaki-Kariwa NPP, located about 16 km from its epicentre, i.e. in the near-field proximity. The IAEA had a strong involvement in the seismic safety evaluation of the plant conducted since then through a number of seismic safety review missions, experts meetings and international workshops and conferences. The establishment of the International Seismic Safety Centre (ISSC) at the IAEA Nuclear Safety and Security Department was an institutional and effective response from the IAEA to the interest and needs arising from this event. The lessons learned and the feedback from the reviews and meetings were incorporated into this report and a subsequent benchmark project (entitled KARISMA) was launched based on the experience from this CRP benchmark

Also, precisely because of the near-field characteristics of the July 2007 earthquake at the Kashiwazaki-Kariwa NPP site and the high values of peak ground accelerations recorded, additional efforts were spent to verify that the results and conclusions from the CRP were still valid in light of such extreme experience. This is the main reason for the delay in issuing this report immediately after completion of the CRP.

This report complements the IAEA Safety Standards as a technical supporting document relative to seismic safety of new and existing nuclear installations and it was developed within the framework of the ISSC activities. Thus, it contributes to the implementation of IAEA Safety Standards providing detailed guidance in relation to seismic analysis, seismic design and seismic safety re-evaluation of Nuclear Installations and, particularly, for the revision of the current Safety Guide NS-G-1.6, Seismic Design and Qualification for Nuclear Power Plants.

The work results reported are of great value to researchers and practicing engineers in these areas. It is also of value to the Member States' governmental organizations, e.g. regulatory authorities, who are responsible for the review and approval of design and evaluation of engineered structures subjected to earthquakes.

The contributions of all those who were involved in the drafting and review of this report are greatly appreciated. P. Labbe of France should be acknowledged for his strong leadership of proposing and managing the CRP. The IAEA officers responsible for this publication were A. Godoy and P. Sollogoub of the Division of Nuclear Installation Safety.

EDITORIAL NOTE

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SUMMARY

INTRODUCTION

The fact that usual practices of earthquake engineering result in a poor estimate of the damaging effects of near-field earthquake input motions generated by low–medium magnitude earthquakes was identified in 1997 by the OECD Nuclear Energy Agency (OECD/NEA) Committee on the Safety of Nuclear Installations (CSNI) as ‘the most significant issue’ in the field of engineering characterization of seismic input motion.

To address this issue, the IAEA organized a Coordinated Research Project (CRP) on the ‘Safety Significance of Near-field Earthquakes’ consisting of two main steps:

(a) Carrying out a benchmark on near-field earthquake (NFE) effects:

- In a first step, the benchmark consisted of interpreting existing experimental data, provided by France, relating to a concrete wall, the CAMUS specimen, subjected to different seismic input motions on a shaking table. Participants modelled the experiments with static and dynamic methods;
- In a second step, the participants were invited to carry out numerical simulation of the response of their models of the CAMUS specimen to a set of seismic input motions provided by Japan;
- A third step consisted of carrying out sensitivity studies about the impact of nonlinearity on floor response spectra, with two types of input motions.

(b) Making proposals for evolution of engineering practice:

- On the basis of the benchmark results, the purpose was to make proposals for possible evolutions of engineering practices so as to realistically account for the effects of the type of near-field input motions and their safety significance.

Twenty-two institutions from 18 Member States were involved in the IAEA CRP, which was jointly funded by the IAEA and the European Union (The Joint Research Centre (JRC), Ispra). Processing and synthesizing the benchmark outputs delivered by the participating institutes were carried out by the JRC Ispra.

CONTEXT AND SCIENTIFIC BACKGROUND

The low damaging capacity of this type of input motion was identified by experts early on and confirmed by feedback from experience. It was extensively discussed on the occasion of experts meeting either within an OECD/NEA or IAEA framework. It was concluded that both the conventional description of seismic input motions in the form of response spectra and the associated conventional engineering practices were not appropriate to resolve the identified issue.

Significant developments have occurred in the last decade in the field of earthquake engineering for conventional buildings, principally with the development and refinement of displacement based approaches (DBAs). However, it was recognized that the nuclear industry has to resolve specific issues that are not addressed by the conventional building industry, namely:

- The nuclear industry is not only interested in the capacity of buildings but also in the transfer of the seismic input motion to equipment; this is known as the floor response spectra generation issue.
- The nuclear industry is interested in refining the analysis of the structural response, in the range of immediate post-elastic behaviour, limited by the conventional limit states (there

is no need to develop tools that would enable a description of the ultimate behaviour of structures in the field of large strains that control the collapse modes). In this regard, a conclusion of the August 2003 IAEA Symposium on Seismic Evaluation of Existing Nuclear Facilities was that “It should therefore be possible to set-up simple methodologies qualified in the range of small nonlinearity.” Although Japanese practice is based on systematic use of time history analysis, the current Japanese practice, described in this IAEA-TECDOC, provides elements of such a rather simple methodology.

On 16 July 2007, the Niigataken-Chuetsu-Oki (NCO) earthquake (moment magnitude of 6.6), affected the TEPCO Kashiwazaki-Kariwa NPP, the biggest nuclear power plant in the world, located at about 16 km of its epicentre; The design basis ground motion were significantly exceeded during the event [99]. Reinforced concrete nuclear buildings (R/B and T/B) had a very good behaviour (only very limited cracks in shear walls) during this event. Review level seismic ground motion according to the new Japanese Seismic Code, lead to higher acceleration (2.3 g compared to original design value, 0.45 g at the rock outcrop) to be considered as basis for seismic safety re-evaluation. Under such a high acceleration value, it is expected that the structure will behave non-linearly. This highlights the importance of good analysis methodology for non-linear behaviour of reinforced concrete shear walls as it is assessed in this report. Moreover, it was important to have access to the first results of analyses performed after NCO earthquake in order to check their consistency with the results and proposals of the CRP reported in this IAEA-TECDOC.

It is worth noting that the different models used to evaluate the seimic response of structures during the NCO earthquake as well as for evaluation of the new seismic ground motion, are very much comparable to those used by different teams in the CRP. This confirms the operational feasibility of performing non-linear analyses in the range of immediate post-elastic behaviour, as illustrated in the CRP programme.

INPUTS FOR THE BENCHMARK

CAMUS experiment

The CAMUS specimen consists of two similar parallel shear walls, strongly clamped on a shaking table and subjected in their plane to 1-D horizontal seismic input motions. The specimen is a mock-up at 1/3 scale of typical shear walls of a six level conventional structure. Its total mass is 36 t. The R-bar system was designed in compliance with the French regulation for conventional buildings against a conventional (referred to below as ‘Nice type’) 0.2 g input motion.

The shaking table was activated by input motions representative of far-field (Nice type) and near-field (San Francisco type) cases scaled to different peak ground acceleration (PGA) values according to the series presented in the table below. Recorded top displacements substantiated the fact that a near-field type motion is less damaging than a far-field type at the same PGA value. A key point for the CRP is that design criteria were not exceeded during these tests and that consequently only relatively small nonlinearity occurred.

Input motions (g)	Nice 0.24	San Francisco 0.13	San Francisco 1.11	Nice 0.41
Top displacements (mm)	7.0	1.5	13.2	13.4

Japanese input motions

Japan is now equipped with a dense network of about 2600 seismometers, which has provided many records in the recent past. As proposed by the Japan Nuclear Energy Safety Organization (JNES), the following input motions were selected from the available near-field record set and the corresponding input motions used by the participants for calculating the response of the CAMUS specimen.

	PGA (g)	PGV (m/s)
N-S component, Ito-Oki	0.19	0.25
E-W component, Kashyo dam	0.53	0.51

OUTPUTS OF THE BENCHMARK EXERCISE

As mentioned in the introduction, the benchmark was organized in the form of a three step exercise. It resulted in a series of 34 analyses of the CAMUS specimen that participants were requested to carry out:

- In step 1, participants were requested to carry out analyses of the response of the CAMUS specimen according to the spectral method, the DBA method and the time history method. Comparative performance, from processing participants' outputs, is presented in the IAEA-TECDOC for top displacement and acceleration of the specimen as well as for bending moment, shear forces and tensile strains in R-bars at the base of the specimen.
- A major interest of step 2 was that (as opposed to step 1) participants could not calibrate their respective outputs against experimental results. step 2 could be regarded as a type of 'blind prediction exercise'. Examining the coefficient of variation (COV) of participants' outputs and comparing it to the COV for step 1 led to the interesting conclusion that COV did not increase and was not larger for high level input motions than for low level inputs.
- A major output of step 3 was to reveal the extreme sensitivity of floor response spectra to small nonlinearity. To a large extent, issues posed by floor response spectra generation are not comparable to issues posed by displacement and/or forces assessment, and are certainly more complicated. For displacement and/or forces evaluation, assumption of linear or quasi linear behaviour may lead to acceptable outputs, while the nonlinear effect can hardly be neglected when dealing with floor response spectra generation.

STRUCTURAL ENGINEERING PRACTICES AND THEIR POSSIBLE EVOLUTIONS

The available engineering methods are presented in this document in the form of a selection of six typical methods (referred to as M1 to M6), from the simplest (linear spectral approach) to the more sophisticated (time history analyses). An outline and the major features of each method are presented with comments. Comments focus on the philosophy of the method: Does it imply static or dynamic equilibrium? Is the input motion implicitly regarded as force or displacement controlled?

The introduction and summary of available methods is followed by a discussion of DBAs, which are becoming more and more popular for the design and evaluation (i.e. the verification of the design) of conventional buildings. Codified methods developed in Europe, New Zealand and the USA are presented and compared. The possible application of these methods to nuclear buildings is discussed, addressing, in particular, the complexity of nuclear structures, the soil–structure interaction issue and the acceptance criteria.

Finally, other options for the evolution of engineering practice are explored, including full scope time history analysis and modelling simplification techniques such as the macro-element approach. Based on the feedback of experience of well established geotechnical engineering methods, an equivalent linear analysis method is proposed and its outlines presented. Member States are invited to test and calibrate it.

CONCLUSIONS AND PROPOSALS FOR THE EVOLUTION OF ENGINEERING PRACTICE

The conclusion of the CRP and the recommendation for the evolution of engineering practice are organized under the following specific topics:

Conclusions of the benchmark exercise

On the safety significance of near-field input motions

- The root cause of the 'significant issue' raised by the low–medium magnitude near-field input motions is not their damaging capacity (there is a consensus that it is very low in spite of their possible high PGAs), but the fact that the engineering community used the response spectrum as an indicator of the damaging capacity of these type of input motions. This indicator significantly overestimates the actual damaging capacity of these types of input motions due to the fact that seismic input motions are conventionally regarded as force controlled loads (or primary loads in mechanical engineering terminology), while high frequency input motions (with respect to the structure frequency) act principally as displacement-controlled loads (or secondary loads in mechanical engineering terminology), thus ignoring the favourable combination of the high frequency content of this type of input motion and the ductile capacity of structures;

On engineering approaches alternate to the response spectrum method:

- DBAs: A drawback of DBAs is that as well as the conventional response spectrum method, they are inherently not appropriate for floor response spectra generation. These methods have been developed for (low frequency) conventional buildings. So far, regarding stiff structures such as nuclear structures, outputs provided by these approaches have not been benchmarked against time history analyses. Nevertheless, the evolution of DBAs should be monitored for possible application to structures typical for nuclear power plants;
- Time history analysis: A major conclusion is that, at least in the simple case of the CAMUS experiment, dispersion of the time history outputs was not greater than dispersion of the response spectrum method outputs. Time history analysis appears to be the most robust method regarding the estimate of displacements, accelerations, forces and moments. This method is also the most robust for estimating the acceptable PGA (the PGA value that leads the structure to the conventional limit state) associated with a given spectral shape and, if properly implemented, is the only method for computing reasonably realistic floor response spectra.

On challenges to nuclear power plant engineering practice

- There is a lack of consistency in the classical nuclear power plant engineering approach due to the concurrent following of practices and/or requirements: structural responses are calculated on an (equivalent) linear behaviour assumption, and acceptance criteria stipulate that forces and moments should not exceed those corresponding to the conventional limit state. On the contrary, significant nonlinear effects appear for low PGAs (significantly lower than those corresponding to the conventional limit state or leading to plastic yield in R-bars). Therefore, any concrete structure, even if designed according to nuclear standards, should be recognized as exhibiting nonlinear behaviour under seismic input motion. Moreover, reasonably realistic floor response spectra cannot be computed without accounting for small nonlinearity effects. Depending on the circumstances, neglecting these effects may lead either to undue margins or on the contrary to a lack of margins in the generated floor response spectra. Thus, an evolution in nuclear power plant engineering practice is highly desirable in this regard.

Proposal for the evolution of engineering practice

Generic recommendations

- In order to adequately calculate the dynamic response of a structure, all aspects of input and models need to be capable of representing phenomena observed or expected, including nonlinear behaviour and complex boundary conditions. Acceptance criteria of structures and components should allow inelastic deformations compatible with the required performance and corresponding performance criteria. It is recommended that the nuclear industry pursues the evolution of the dynamic modelling techniques taking into account at least small nonlinearities in the models;

Accompanying R&D effort

- Further R&D effort should focus mainly on theoretical evolution and experimental tests to improve and validate the DBAs and the time history approaches. For the latter approach, standard procedures for design and verification should be implemented and proven to be realistically conservative;

Specific recommendation on strong motion scaling factors

- It is expected that in the future more and more high frequency input motions will be recorded, thus resulting in higher and higher PGA values, which are meaningless in terms of input motion damaging capacity to structures. It is therefore strongly recommended that a more relevant and simple indicator be selected and adopted by the structural engineering community as a scaling factor of recorded strong motions, such as peak ground velocity (PGV) or cumulative absolute velocity (CAV), and that a significant R&D effort be carried out to concur on engineering practices incorporating this new scaling factor.

1. INTRODUCTION

1.1 Background

It is a well known technical finding that the usual practices of earthquake engineering result in a poor estimate of the damaging effects of near-field seismic input motions. This observation is valid for both large magnitude and small magnitude earthquakes. However, according to international and national safety standards, locating nuclear power plants in the vicinity of seismogenic sources capable of generating large magnitude earthquakes is to be avoided[1]. Therefore, in the frame of the engineering of nuclear power plants, the case of small magnitude near-field input motions is most relevant, and was identified in a 1997 OECD/NEA report [1] as ‘the most significant issue’ in the field of engineering characterization of seismic input motion. It is

[1] e.g. according to the IAEA Safety Standards, it is precluded to locate a nuclear power plant on a site where there is evidence of a capable fault that can generate surface faulting.

also recognized that this issue is more critical for the evaluation of existing nuclear facilities than for the design of new ones.

According to the state of the art, the aforementioned poor predictive performance is very likely linked to the fact that most structures exhibit nonlinear behaviour under seismic input motions. Consequently, in the case of near-field input motions, when computing structural response on a linear behaviour assumption, displacement estimate is generally more reliable than stress estimate, while unfortunately structure assessment is based on stress analysis. Therefore, extensive academic work has been carried out in the past decade in order to better take into account the role of displacements. It resulted in the development of DBAs. These approaches are codified and applied by the conventional building industry and are under consideration, but not yet applied in practice in the nuclear industry[2]. However, it has to be recognized that the features of nuclear buildings and the required behaviour in the case of an earthquake are different from those of conventional buildings.

1.2. Objectives of the publication

A significant amount of work was performed during the CRP with many achievements in different domains such as seismic analysis of structures in linear and non-linear range, comparison between different computational models, variability of results among different seismic excitations and different teams, characterisation of input signals from different sources and their damaging capacity, proposition of criteria to be used in conjunction with linear and non-linear analyses, exemples of codification of non-liner analyses, derivation of floor response spectra and proposals for evolution of nuclear power plant engineering practice. All these subjects are very important for the assessment and evolution of methods used for seismic design.

The objective of the present publication is to present the research results of the IAEA CRP in the light of the observed behaviour of the Kashiwazaki-Kariwa Nuclear Power PLant subjected to NCO earthquake and to contribute to the development, revision and implementation of IAEA Safety Standards related to seismic analysis, seismic design and seismic safety re-evaluation of nuclear installations such as the Safety Guide NS-G-3.3 on Evaluation of Seismic Hazard for Nuclear Power Plants [4], under revision as DS422, Seismic Hazards in Site Evaluation for Nuclear Installations, the Safety Guide NS-G-2.13 on Evaluation of Seismic Safety of Existing Nuclear Installations, or the Safety Guide NS-G-1.6, Seismic Design and Qualification for Nuclear Power Plants which revision is starting and will take full advantage of information included in this publication.

In this context, the following aspects are addressed:

- Safety significance of the small magnitude near-field input motions, which was the aim of the aforementioned OECD report, and to draw conclusions on it. In the frame of this CRP, the term ‘near-field earthquake’ (NFE) corresponds to this type of input motions;
- To determine to what extent the DBAs are also recommendable for the assessment of nuclear facilities, in particular when subjected to near-field inputs; and more generally to propose an appropriate evolution of engineering design practices for nuclear facilities in this regard.
- Compare the IAEA CRP results with the situation observed at the Kashiwazaki-Kariwa Nuclear Power Plant which experienced NCO earthquake and to draw conclusions for further research and improvement of seismic design methodologies.

[2] ASCE 4 revision, as well as Eurocode 8, have a section on nonlinear pushover analyses.

1.3 Scope and process of the CRP

The focus of the IAEA CRP is on the behaviour of structures when subjected to near-field earthquake ground motions. The observations and conclusions herein may also apply to ductile systems and components whose failure modes are due to multiple cycles of vibratory motion. The observations and conclusions herein do not pertain to acceleration-sensitive devices and components whose failure mode may be due to brittle stress or strain related failure or due to operability issues such as relay chatter. The terms ‘safety’ or ‘safety significance’ as used herein refer to the ability of structures, systems and components (SSCs) to perform their required function during and/or after the occurrence of an earthquake.

The substance and the outlines of this IAEA CRP were discussed and approved by the Member States during a Technical Committee Meeting (TCM) on Seismic Evaluation of Existing Nuclear Facilities that was held on 3–7 December 2001 in Vienna. Prior to the TCM, the Member States were invited to inform the IAEA about their available input to a possible IAEA CRP. From the positive answers received, the following inputs were selected for the IAEA CRP:

- Results of experiments on a shaking table, with inputs representative of NFEs, provided by France;
- A database of NFE records provided by Japan;
- An assessment of the relevance of DBAs provided by the USA.

It was decided to organize the IAEA CRP around a benchmark exercise on the basis of the French and Japanese inputs, and to include discussions on the evolution of engineering practices starting with the input from the USA.

The IAEA CRP was planned for a two year period. The kick-off meeting, also the first Research Coordination Meeting (RCM), was held in Istanbul in October 2002. During the meeting, the documentation necessary to conduct the benchmark was distributed to the participants and the detailed schedule of the IAEA CRP was finalized. This meeting was held concurrently with an OECD/NEA workshop on exchanges between seismologists and engineers.

The second RCM was held in Trieste in March 2004. On the basis of the encouraging results already achieved, the participants expressed the wish that the IAEA CRP be extended for one year and its scope broadened. The IAEA Research Programme Committee approved this request in April 2004: the research activity was extended until the end of 2005 and step 3 of the benchmark was added to the work plan.

In view of the objective stated above, and building on the contributions provided by the Member States, the IAEA CRP consists of:

(a) Carrying out a benchmark on NFE effects

The benchmark was organized as a three step exercise, as follows:

Step 1: Interpretation of the CAMUS experiment

In a first step, the purpose is to carry out an interpretation of existing experimental data so that the participants share:

(i) The safety significance of the experimental results;

(ii) A common view of the necessary evolution of engineering practice.

The experimental background consists of outputs of the response of a concrete wall, the CAMUS specimen, subjected to different seismic input motions on shaking tables. This background, provided by France, is presented for information in Annex III on the attached CD. The input motions are representative of near-field as well as of far-field ground motions for the purpose of comparison of the effects.

Step 2: Numerical simulations with Japanese input motions

In a second step, the participants are invited to carry out numerical simulation of the response of their models of the CAMUS specimen to a set of seismic input motions representative of NFEs (provided by Japan) and to examine the outputs of engineering methods on these examples.

Step 3: Effects on floor response spectra generation

An input motion representative of a far-field and another one representative of a near-field are selected. For each type of input, a series of time history analyses of the CAMUS specimen are carried out, with a PGA scaled from 0.1 to 0.6 g. The purpose is to compare the damaging effects of the two types of input motions and the corresponding impacts on the floor response spectra.

(b) Concurring on an engineering practice

On the basis of the benchmark results, the purpose is to concur on the main features of an appropriate methodology to realistically account for the effects of near-field input motions and their safety significance. A basis for this is the 'Assessment of the relevance of displacement-based methods' (NUREG/CR-6719) [2]. The IAEA CRP covers the scope of the NFE that is not specifically addressed in this document.

The list of the participating institutions is shown in Table 1; more detailed information about participating institutions is provided in Annex I. Twenty-two[3] institutions from 18 Member States (Armenia, Bulgaria, Canada, China, Finland, France, India, Italy, Japan, Republic of Korea, Pakistan, Romania, the Russian Federation, Slovakia, Spain, Turkey, the United Kingdom and the USA) were involved in the IAEA CRP. Each research team was required to provide a yearly research report (a draft report was also requested) and to provide its benchmark outputs in a prescribed format. The institutions from Member States that were not entitled to receive financial support through the IAEA Technical Cooperation programmes funded their respective contributions.

The IAEA CRP was funded by the IAEA and by the EU, which supported institutions from Member State candidates to EU accession. A memorandum of understanding was signed in this regard between the IAEA and the JRC of the European Commission (EC). The JRC in Ispra also contributed to the development of the IAEA CRP by two other means. First, an exchange platform was developed and managed at Ispra; every document relevant to the IAEA CRP, either prepared by the organizing committee (OC) or by a participant, was uploaded on this platform, which was accessible through the internet. In particular, all the benchmark outputs were uploaded on the platform. Second, the JRC recruited a visiting scientist for two years in charge of synthesizing the benchmark outputs delivered by participants.

As well as the IAEA and the EU, France, Japan and the USA, which have provided inputs to the IAEA CRP, were represented in the OC. There was also a participants' representative and a representative of the OECD/NEA[4] (Table 1). The role of the OC was to review the research reports and to make decisions for the conduct of the IAEA CRP. For instance, the OC finalized the benchmark output format (BOF) that was prescribed for participants, selected the Japanese input motions for step 2 of the benchmark and decided on the content of step 3 that was a consequence of the extension of the IAEA CRP.

[3] Twenty-one research teams participated in the benchmark exercise. One institution, the French Commissariat à l'Energie Atomique (CEA), provided data for the benchmark.

[4] The IAEA and the OECD-NEA closely coordinate their activities in the field of earthquake engineering.

TABLE 1. PARTICIPATING RESEARCH TEAMS AND ORGANIZING COMMITTEE

Participating research teams		
Member	Institution	Team leader
Armenia	ANRA	Zadoyan, P.
Bulgaria	BAS	Kostov, M.
Canada	AECL	Elgohary, M.
China	BINE	Wei, L., Chen, M.
Finland	Fortum	Varpasuo, P.
France	CEA	Sollogoub, P.
France	INSA Lyon	Nazé, P.-A.
France	IRSN	Orbovic, N.
India	AERB	Basu, P.
Italy	Polit. Di Milano	Mulas, G.
Japan	JNES	Kitada, Y.
Republic of Korea	KOPEC	Park, C.S.
Republic of Korea	KINS	Hyun, C.-H.
Pakistan	PAEC	Mahmoud, H.
Romania	UCTB	Lungu, D.
Russian Fed.	CTKI_Vibroseism	Kostarev, V.
Slovakia	SAS	Juhasova, E.
Spain	IDOM	Beltran, F.
Turkey	TAEK	Altinyollar, A., Saral, F.
Turkey	METU	Yakut, A.
UK	HSE	Donald, J.
USA	BNL	Simos, N.
Organizing committee		
IAEA		Labbé, P., Godoy, A.
EU, JRC Ispra		Renda, V.
EU, JRC Ispra		Altinyollar, A.
France	CEA	Sollogoub. P.
Japan	JNES	Kitada, Y.
Turkey	METU	Gülkan, P.
USA	NRC	Murphy, A.
OECD/NEA		Mathet, E.

1.4 Summary of the KK-NPP behaviour during NCO earthquake

An earthquake with moment magnitude of 6.6, occurred at 10:13 h local time with its hypocentre below the seabed of the Jo-chuetsu area in Niigata prefecture (37º 33’ N, 138º 37’E) in Japan, affecting the Kashiwazaki-Kariwa Nuclear Power Plant (NPP) located approximately 16 km south of its epicentre. Kashiwazaki-Kariwa NPP is the biggest nuclear power plant site in the world. It is located in the Niigata prefecture, in the northwest coast of Japan, and it is operated by Tokyo Electric Power Company (TEPCO). The site has seven units with a total of 7965 MW net installed capacity. Five reactors are of BWR type with a net installed capacity of 1067 MW each. Two reactors are of ABWR type with 1315 MW net installed capacity each. The five BWR units entered commercial operation between 1985 and 1994 and the two ABWRs in 1996 and 1997 respectively.

At the time of the earthquake, four reactors were in operation: Units 2, 3 and 4 (BWRs) and Unit 7 (ABWR). Unit 2 was in start-up condition but was not connected to the grid. The other three reactors were in shutdown conditions for planned outages: Units 1 and 5 (BWRs) and Unit 6 (ABWR). The earthquake caused automatic shutdown of the operating reactors, a fire in the in-house electrical transformer of Unit 3, release of a very limited amount of radioactive material to the sea and the air and damage to non-nuclear structures, systems and components of the plant as well as to outdoor facilities, as reported by TEPCO on their web page.

Records obtained from this earthquake in all locations where instruments were installed show that the range of frequencies of signals is wider than the specific type discussed in this document. Observations indicated that the safety related concrete structures most probably preserved their elastic behaviour expected in original design during the earthquake. For reevaluation to higher design basis earthquake, it was necessary to have robust analyses approaches able to cope with non-linear behaviour of structures. This CRP investigated the effectiveness of such approaches.

1.5. Structure of the publication

Following this introduction, Section 2 provides information on the context of the CRP and the history of the NFE issue, including developments of engineering practices in the conventional building industry over the past decade and the views of the nuclear industry on the subject. Input data for the CRP (French and Japanese inputs) are also presented in Section 2.

Section 3 is dedicated to outputs of the benchmark as they were provided by the participants and processed in order to present average values and standard deviations of these outputs. Lessons learnt from the benchmark are also presented for each step. This section also includes an analysis of the consistency of the outputs from the 34 different analyses that were carried out by the participants, as well as an analysis of the specimen on the basis of the conventional reinforced concrete (RC) approach.

In Section 4, the range of available engineering methods is presented in the form of a selection of six typical methods, from the simplest (linear spectral approach) to the more sophisticated (time history analyses). An outline and the major features of each method are presented with comments. The second part of the section is dedicated to DBAs, which are becoming more and more popular in the conventional building industry. Codified methods developed in Europe, New Zealand and the USA are presented and compared. The possible application of these methods to nuclear buildings is discussed. Finally, in the third part of the section, other options for the evolution of engineering practice are explored, and, based on the feedback of experience of well established geotechnical engineering methods, an equivalent linear analysis method is proposed and its outlines presented.

Section 5 summarizes the main conclusions of the CRP and recommendations for the evolution of engineering practice. The conclusions are presented in 15 items, organized into the following topics:

- On the safety significance of near-field input motions;
- On alternative engineering approaches to the response spectrum method;
- On challenges to nuclear power plant engineering practices.

Proposals for the evolution of engineering practice consist of 14 items, organized into the following topics:

- Generic proposals;
- Accompanying R&D efforts;
- Specific recommendation on a strong motion scaling factor.

On an attached CD-Rom detailed results and developments are organized in 11 Annexes:

ANNEX I	List of Participants
ANNEX II	Summary of the Research Coordination Meetings (RCM)
ANNNEX III	Description of the CAMUS Data
ANNEX IV	Description of the Japanese Input Motions: Near-field Earthquakes Observed Recently in Japan
ANNEX V	Description of the Outputs Requested of the IAEA CRP Participants
ANNEX VII	Results of Benchmark step 1
ANNEX VIII	Results of Benchmark step 2
ANNEX VIII	Results of Benchmark step 3
ANNEX X	Scientific background on classification of seismic loads as primary or secondary
ANNEX XI	Japanese Practice on Nonlinear Seismic Response Analysis of Safety Related Important Structures

2. CONTEXT AND SCIENTIFIC BACKGROUND

2.1. Challenges posed by near-field input motions

2.1.1. Successive steps in the recognition of the issue

2.1.1.1. The classical postulate of earthquake engineering in nuclear industry

As described in detail in Section 3 under Method M1, the classical practice of earthquake engineering in the nuclear industry, was established some decades ago on the following bases:

- The input motion was described by a response spectrum;
- The analysis of the response of the structure was based on the assumption of elastic behaviour and evaluated through classical response spectrum analysis (RSA);

- The acceptance criteria were derived from criteria of the conventional building industry, such as criteria linked to the concept of ultimate limit state (ULS).

Implicitly, this approach relies on a postulate that can be formulated as follows:

Postulate A: The classical description of the input motion associated to the classical engineering practice leads to a suitable[5] estimate of the damaging capacity of the input motion.

2.1.1.2. Challenging observations relating to seismic input motions

As mentioned in the introduction, deficiencies of Postulate A were identified as soon as 1997 in an OECD/NEA report [1]. In November 1999, an OECD/NEA workshop was organized at the Brookhaven National Laboratory (BNL) in order to thoroughly investigate the issue [3]. During this workshop, the new challenging observation was described as follows: "In Central and Eastern US as well as in lower seismic regions of Europe, site-specific response spectra developed in recent research studies for rock sites have very different shapes than those we have traditionally used in design of NPPs."

Those spectra were qualified as 'modern' spectra, characterized by significantly more high frequency content and significantly less low frequency content. The conclusion of the workshop was that "The new ideas challenge the established concept of design response spectra".

Consistently, recommendations were made in order to improve the description of the seismic input motion (methods for ground motion estimation, data collection of motions, site characterization issues, data from high seismic area vs. data from low seismic area, etc.). The new implicit postulate was thus:

Postulate B: A sophisticated description of the input motion associated to classical engineering practice will lead to a suitable estimate of the damaging capacity of the input motion.

2.1.1.3. Challenging observations relating to the engineering approach

The deficiencies of the classical approach do not, however, only relate to the description of the seismic input motion. The poor predictive capacity of the classical engineering approach is a well known technical finding, substantiated by the feedback of experience (fact finding from the feedback of experience is developed in Section 2.1.3), and by experimental observation, particularly by the CAMUS experiment, which is addressed in the frame of the IAEA CRP.

This poor predictive performance was the subject of extensive academic work over the past decade. As opposed to the conventional nuclear approach that relies on seismically induced inertial forces, it is now recognized that a safe anti-seismic design consists of accommodating large strains much more than balancing large forces. In other words, instead of only putting the emphasis on acceleration effects, greater attention should be paid to displacement effects. Now, regarding small magnitude near-field input motions, they are characterized by rather high accelerations associated with small displacements. It was, therefore, expected that an adequate development of the engineering approach should concur to eliciting the issue raised by the near-field input motions.

In April 2000, the OECD/NEA working group on the seismic behaviour of structures recognized that some progress is necessary in the description of the input motions. However, it recommended

[5] Suitable for the purpose of design of facilities, in the sense that in case a criterion is not met the structure is regarded as not properly designed.

not to put the emphasis only on this facet, but to work in parallel on the development of engineering methods.

Postulate C: An improved description of the input motion associated to improved engineering practice will lead to a suitable estimate of the damaging capacity of the input motion.

The IAEA CRP was organized in the spirit of Postulate C, developing activities both in the field of engineering characterization of seismic input motions and of engineering practices.

2.1.2. Which type of near-field input motion?

The type of challenging input motion was identified at the BNL workshop in 1999 (see Section 2.1.1.2). Input motions are generated by low–medium magnitude earthquakes in their near-field. The concept of near-field was extensively discussed during the second RCM of the IAEA CRP and the discussion resulted in the clarification of the terminology. For any earthquake, it is possible to identify a near-field and a far-field, depending on the distance to the epicentral area. It is clear that the closer the epicentre, the stronger the expected ground motion. However, the amplitude is not the only feature of the near-field ground motions; for instance, the distance to the epicentre also has significant effects on the duration of the input motion and its frequency content.

Not only are the features of near-field input motions different from those of far-field input motions, but in addition it is clear that the damaging capacity and the features of a near-field input motion delivered by a magnitude 5 earthquake are totally different from those delivered by a magnitude 7 event.

According to the rules and regulations adopted for the selection of nuclear power plant sites, and in particular according to the IAEA Safety Standards [4], a nuclear power plant may not be located in the near-field of a possible large magnitude earthquake. Consistent with this siting rule and with the case rose by the OECD/NEA [1, 3], the near-field input motions considered in this IAEA CRP can be defined as follows: “Short duration, relatively high frequency content input motions generated by low–medium magnitude earthquakes, particularly in low-seismicity areas.”[6]

The main features of some input motions of this type are summarized in Table 2. The high frequency content of these signals is indicated by the PGA:PGV ratio. It is usually considered by seismologists that a ratio larger than ten is an indicator of high frequency content. For the input motions shown in Table 2, the lowest value is 20.4.

Although San Francisco area cannot be regarded as a low seismicity area, the 1957 San Francisco earthquake ground motion mentioned in this table was used during the CAMUS experiment because it was regarded as representative of the type of near-field input motions under consideration in the NFE issue.

The results of seismic hazard studies performed in 2006 for rock sites contain very significant high frequency content for near-field, low magnitude events, e.g. significant acceleration spectral ordinates in the frequency range greater than 20 Hz.

[6] For the purposes of this study, ‘near-field’ ground motions refer to ground motions with a frequency content at higher frequencies than the fundamental frequency of the specimen tested and analysed.

TABLE 2. EXAMPLES OF NEAR-FIELD INPUT MOTIONS CONSIDERED IN THE IAEA CRP

Event and station	Magnitude	Focal distance (km)	PGA (cm/s^2)	PGV (cm/s)	PGD (cm)
Ancona 1972 Rocca	4.9	8.5	598	9.4	0.7
Lytle Creek	5.4	14.4	196	9.6	1.0
San Francisco 1957 Golden Gate Park	5.3	18.0	118	4.6	0.8
Stone Canyon 1972 Melendy Ranch	4.6	9.4	696	19.5	0.6
Leroy 1986 Perry NPP	5.0	17.5	180	2.2	0.15

2.1.3. Feedback of experience and expert judgement on these NFE input motions

Overestimating the damaging capacity of input motion such as that derived from indicators based on acceleration and consequently on the high frequency content of the seismic input motion was questioned as early as 1981 [5]. Furthermore, in 1981, a paper dealing with 'The response of a nuclear power plant to near-field moderate magnitude earthquake' was published by Newmark, Kennedy and Short at the SMIRT Conference [6]. The conclusion of this paper is that "…a 0.5 g near-field acceleration time history from a low magnitude earthquake is not damaging to a nuclear power plant structure designed for a broad frequency content response spectrum anchored to a safe shutdown earthquake (SSE) acceleration of 0.2 g." In 1987, the Electric Power Research Institute (EPRI), USA organized a workshop on 'Engineering characterization of small magnitude earthquakes' that summarized all available data on the matter at that moment [7].

The very low damaging capacity of this type of input motion was confirmed in 1986 by the effect of the Leroy earthquake (magnitude M_L= 5.0) that occurred 17.5 km from the Perry nuclear power plant in Ohio. The SSE spectrum of this nuclear power plant was anchored at 0.15 g. Recorded motion on the foundation and in the structure showed that the SSE response spectra were expected above 15 Hz. The recorded peak acceleration on the foundation was 0.18 g. The staff at the plant did not observe any indication of damage, which was confirmed by follow-up inspections.

As early as 1978, Newmark stressed the point that the (first) structural eigenfrequency plays a crucial role in margins generated by design practices. His conclusions were expressed in the NUREG/CR0098 [8] in the form of a reduced spectrum applicable for the seismic review of nuclear structures built early on. It was stated later that a key parameter is the (first) structural eigenfrequency in comparison with the frequency content of the input motion. In other words, it was identified that input motions with high frequency content tend to act as secondary loads, which is the root cause of their low damaging capacity [9] in relation to the ductile capacity of the structures.

Another root cause of the low damaging capacity of this type of input motion was discussed as early as 1986 in Ref. [10], in relation with the incoherence of the high frequency content. Subsequently, considerable additional data on this matter became available, in particular thanks to the Lotung and Hualien dense instrumentation arrays implemented under EPRI leadership [11, 12].

In 1993, EPRI issued a report [13] that summarized the available information on the damaging capacity of this type of input motions. According to this reference: "It is widely accepted among knowledgeable engineers that ground motion must have a substantial content in the 1–4 Hz range to be potentially damaging to nuclear power plant structures, and that higher spectral accelerations at frequencies in excess of about 10 Hz are of little or no interest in the design of such structures, unless a very severely poorly designed and constructed link exists within the structure."

Consequently, the main purpose of this report is not to establish the low damaging capacity of this type of input motion, but to make recommendations for reducing the high frequency content of ground response spectra for design purposes.

Earthquake ruggedness of equipment is not discussed in the frame of the IAEA CRP. However, it must be mentioned that, if not damaging for structures, high frequency input motions generated by low to moderate magnitude earthquakes may be an issue for acceleration-sensitive equipment. It is particularly the case of relays that may chatter, as observed at the Perry nuclear power plant in 1986 [14].

2.2. Recent developments in engineering practices

2.2.1. Comparing developments in the conventional and nuclear building industries

2.2.1.1. Seismic design of conventional buildings

The required performance of a conventional building in the case of a seismic event is not comparable to that required from a nuclear building. Generally, life safety is the performance criterion for the minimum design requirements for conventional structures. Possible damages are accepted, corresponding to post-elastic drifts in structural elements. In order to fit these type of requirements, the conventional building industry has developed appropriate methods. One such typical method is the 'behaviour factor method' (Method M4 in Section 3). Along general lines, such methods consist of:

- Computing the response of the structure assuming elastic behaviour. Soil structure interaction (SSI) effects are neglected. Usually, for frame structures, a reduced Young modulus is selected so as to account for cracking effects.
- Applying reduction factors either to seismic load or to forces.
- Checking compliance of forces with acceptance criteria.

More recently, the technical finding mentioned in Section 2.1.1.3 about the prominent role played by displacements in a safe anti-seismic design resulted in the development of DBAs, which are described in detail in Section 3. An interesting feature of these methods is that they imply developments in both the description of the input motion (in the form of an acceleration–displacement response spectrum) and the engineering approach (in the form of the pushover curve). As well as the 'behaviour factor method', the DBAs are now codified and are currently being applied by the conventional building industry.

2.2.1.2. Seismic design of nuclear facilities

A major difference between classical nuclear industry practice and conventional building industry practices is that the reduction factors mentioned above have not been accepted. In the past decades, principally due to the fact that nuclear power plant building activity has been stopped in most countries, classical nuclear practice as described in Section 2.1.1.1 has practically not evolved.

In the recent past, several pieces of evidence for evolution appeared; for instance, in the USA with the publication of the ASCE code 43-05 [15] or in France with the publication of the updated Safety Guide on seismic design of nuclear facilities [16]. In those standards, a realistic modelling of concrete behaviour is recommended, including cracking effects to the necessary extent. However, so far, these standards have not yet been used for the construction of any facility.

2.2.1.3. Seismic evaluation of existing nuclear facilities

In the past two decades, in relation with the seismic re-evaluation of existing facilities, the nuclear power plant engineering community had to face challenges, either relating to the re-evaluation of the seismic input motion or to a poor anti-seismic original design, resulting in the design criteria not being met. Developments in the nuclear industry's methods were driven by the necessary resolution of this issue.

In order to cope with this situation, pioneering work was carried out in the USA. It resulted in the development of methods used to evaluate nuclear power plants for beyond design basis earthquakes; these methods included seismic margin assessment and seismic probabilistic safety assessment (PSA) techniques. The development of the generic implementation procedure (GIP) was motivated by the need to address the issue of the lack of seismic qualification for equipment and components designed, procured and installed under versions of the IEEE 344 1972. The GIP extensively used earthquake experience data, i.e. the documented observed behaviour of equipment, components and commodities in large earthquakes. It also relied on test data denoted GERS to address operability issues. The GIP was used extensively in the performance of the SMAs and SPSAs [17]. This approach was successfully implemented in USA, for instance by the Department of Energy [18]. It spread in other countries, and in particular it was adapted to the case of the Eastern European nuclear power plants. A consensus on the conduct of seismic evaluation of existing nuclear power plants was achieved and is reflected in Safety Reports Series No. 28 [19]. Basically, the consensus is that procedures for seismic evaluation of existing nuclear facilities should be similar to those for the design of conventional buildings with adapted reduction factors.

2.2.1.4. Challenge to the leadership of the nuclear industry

In the past, approximately 20 years ago, the nuclear industry was regarded as a leader in the field of earthquake engineering. The engineering developments were first carried out for the nuclear industry and afterwards spread to the conventional building industry (e.g. the conventional RSA).

Meanwhile, the situation has changed. The nuclear industry has developed original methodologies for the seismic evaluation of existing nuclear facilities. However, to a large extent, these methods are based on methods, such as or similar to the behaviour factor method, which were first developed for the conventional building industry.

The situation is similar when addressing the issue raised by the near-field input motions. It is expected that the DBAs, which were first developed for and by the conventional building industry, will provide tools for its resolution.

Thus, it appears that the nuclear industry is challenged not only from a technical viewpoint by the issue of near-field input motions, but also in its role of leading industry in the field of earthquake engineering. It is expected that the successful accomplishment of this IAEA CRP will be a step for the nuclear industry on its way towards once again playing a leading role in this field in the future.

In defence of the nuclear industry, it must be stated that applying methods of the conventional building industry is far from easy due to the complexity of nuclear buildings and the specificity of the nuclear safety requirements.

s developed further in Section 2.2.2, the nuclear industry has already considered the effects of this challenging context. In particular, the interest of DBAs and their possible use have been addressed in Safety Reports Series No. 28 [19] and in NUREG CR-6719 [2]. In this regard, an objective of the IAEA CRP on the safety significance of near-field earthquakes is to investigate to what extent the new methods of the conventional building industry can be adopted and/or adapted to the nuclear industry context.

2.2.2. The nuclear industry's views on DBAs

In 2001, the US Nuclear Regulatory Commission (NRC) issued NUREG/CR-6719 [2] on the Assessment of the Relevance of Displacement-based Methods. The assessment was based on two case studies: (1) a shear wall and (2) a turbine building with strong geometrical nonlinearity. The conclusions were that (1) for the shear wall the DBAs do not provide added value as compared to the classical force based approach with reduction factors, and (2) for the turbine hall, the DBA does not work[7].

The NRC concluded that the method is not applicable to the design of nuclear structures but is suitable for seismic margin studies in the case of material nonlinearity only. In such a case, there is a significant advantage of the pushover technique as compared to time history analyses. The NRC further concluded that if the method is to be applied on a wide scale, it is necessary to develop an appropriate version for nuclear facilities (e.g. drift limits consistent with the importance of the structure).

In February 2003, the OECD/NEA working group on seismic behaviour of structures edited a topical paper on the 'Apparent discrepancies between nuclear and conventional seismic standards' [20]. Its conclusions were that the use of performance-based engineering approaches, and in particular displacement-based analysis should be encouraged where appropriate in the frame of the engineering of nuclear power plants.

In the Safety Reports Series on Seismic Evaluation of Existing Nuclear Power Plants [19], views on DBAs are expressed as follows: "…according to the state of the art … approaches orientated towards strain evaluation (displacements approach) are more relevant than those based on stresses evaluation (forces approach). Consistent strain analysis is generally difficult to achieve because engineering practices and engineering tools (education, standards, criteria and computer codes) are orientated towards stress analysis. For this reason, in order to provide convenient guidance, the rules that follow are expressed in the general framework of stress analysis; in this framework the inelastic energy absorption factor $F\mu$ is introduced … Nevertheless should a strain based approach be proposed, it should be regarded with interest and carefully examined."

In August 2003, the nuclear community discussed in detail the interest of the DBAs on the occasion of the IAEA Symposium on Seismic Evaluation of Existing Nuclear Facilities, held in Vienna [21]. The conclusions were as follows:

[7] During the August 2003 IAEA Symposium [21], the interest of the turbine hall case was regarded as questionable and the result not significant because in any case the classical force based approach does not work either in case of strong geometrical nonlinearity.

“The Displacement Based Method seems a very attractive, simple and promising methodology but it cannot yet be considered as a standard assessment procedure for nuclear power plants because of its current limitations:

- The definition of criteria in terms of allowable displacements;
- The global criteria might not be sufficient;
- The procedure is applicable for framed and regular structures;
- The procedure applies only to one direction (horizontal).

Developments of this method are also necessary in order to:

- Take into account soil–structure interaction;
- Derive the floor response spectra and relative displacements;
- Examine the impact on component assessment.

Further R&D in view of the application of this method should be carried out, and when applicable the use of the method should be encouraged.”

2.2.3. Limited need of the nuclear industry for nonlinear analyses

2.2.3.1. Needs for addressing nonlinear behaviour

A feature of any reinforced concrete nuclear structure is that it is basically designed to stay in the elastic range of both R-bars in tension and concrete in compression. However, far before encountering these limits, nonlinearity may appear in the form of concrete cracking, starting with micro-cracking phenomena. So far, in several Member States this nonlinear effect has generally not been considered when addressing their seismic response in view of the design of nuclear installations. As mentioned in Section 2.2.1.2, this situation is now evolving [15, 16].

It is well known by engineers that neglecting this effect may lead to unrealistic and unreliable conclusions, as exemplified by the analyses of the CAMUS experiment developed in the frame of the IAEA CRP and reported further in this document. There is now evidence that this phenomenon should be taken into account when addressing the response of nuclear buildings under seismic input motions.

2.2.3.2. Limits induced by nuclear safety considerations

It is clear for the public, as well as for the regulators that, in the face of seismic threat, the safety of nuclear installations should be significantly better than the safety of conventional buildings. This objective could be achieved either by a higher level of seismic input or by a more demanding engineering approach. In the current context, the objective of better safety is achieved not only through higher levels of seismic input motion but also through a more demanding engineering approach. For instance, the requirement for a conventional building is that it does not collapse (heavy damage to the equipment is possible, the objective is to preserve the lives of the occupants), while, in addition to life safety, the performance requirements of nuclear SSCs are required for a nuclear power plant, so that the installation can be safely shut down during and/or after the earthquake shaking.

What are the expectations of the nuclear industry regarding the modelling of the nonlinear behaviour of concrete structures? This item was discussed in August 2003, during the IAEA Symposium on Seismic Evaluation of Existing Nuclear Facilities. It was concluded that, at least in a first step, there is no need to develop tools that would enable describing the ultimate behaviour of structures in the field of large strains that control the collapse modes. The interest of the nuclear industry is in refining the analysis of the structural response, in the range of immediate post-elastic behaviour, limited by the conventional limit states. A conclusion of the symposium was that "It should therefore be possible to set-up simple methodologies qualified in the range of small nonlinearity."

2.2.4. Japanese practice on nonlinear time history analysis

Despite the description in the previous section, Japan has already introduced nonlinear analysis in the seismic design of nuclear power plants because Japan is an earthquake prone country and design earthquake ground motions of some nuclear power plants have a high PGA.

In the design analyses, two types of earthquake ground motions, S_1 and S_2, have been applied [22]. The S_1 earthquake ground motion is defined as the maximum design earthquake and the response to this motion is limited within the linear range. The S_2 earthquake ground motion is defined as the extreme design earthquake and the response to this motion is allowed to be in the range of small nonlinearities with considerable margins by taking into account the realistic earthquake response behaviour of the structures.

In the nonlinear analyses, two types of geometrical and material nonlinearities are taken into account as shown in Fig. 1. One type is the base-mat uplift phenomenon, which is considered necessary when the input motion is large and the overturning moment under the base-mat is large. In general, this phenomenon is taken into account by introducing nonlinear rocking springs in the analytical model.

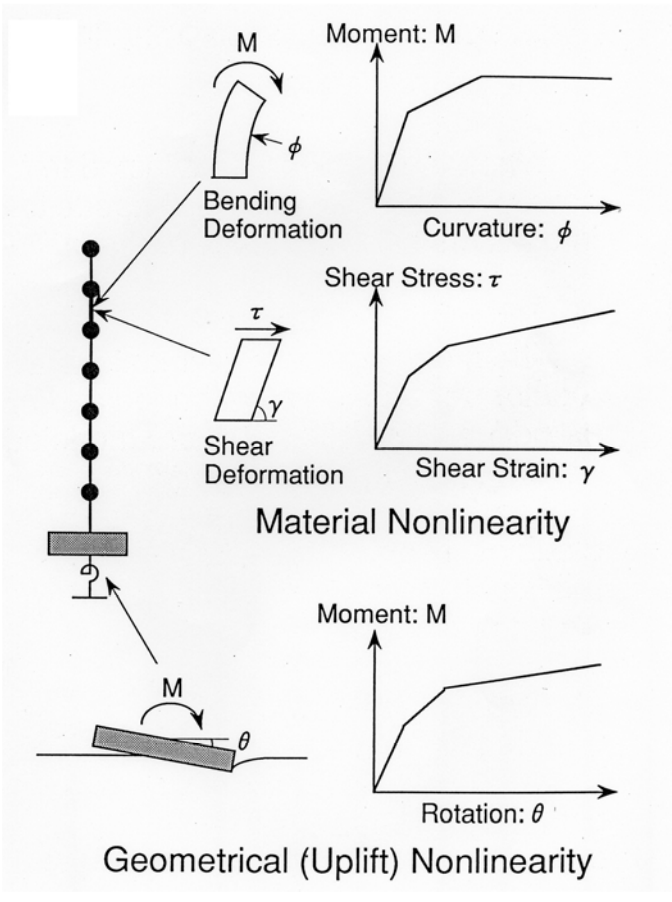


FIG. 1. A conceptual model of nonlinear earthquake response analysis of nuclear buildings.

The second type is bending and shear deformation of RC shear walls. The bending moment–curvature relationship and shear stress–strain relationship are idealized as bi- or tri-linear skeleton curves. The methodology for determining the skeleton curves and hysteresis loops for bending and shear deformation characteristics currently recommended in the technical guidelines for anti-seismic design of nuclear power plants in Japan are shown in Annex XI.

The building models usually used in the design analyses are so called bending-shear type lumped-mass. Several box-shaped and cylindrical RC shear walls modelled as bending/shear type elements are assembled in the model. In evaluating the bending and shear stiffness, the 3-D effect is taken into account using FEM, etc.

Typical analytical models for BWR type and PWR type reactor buildings are shown in Figs 2 and 3, respectively.

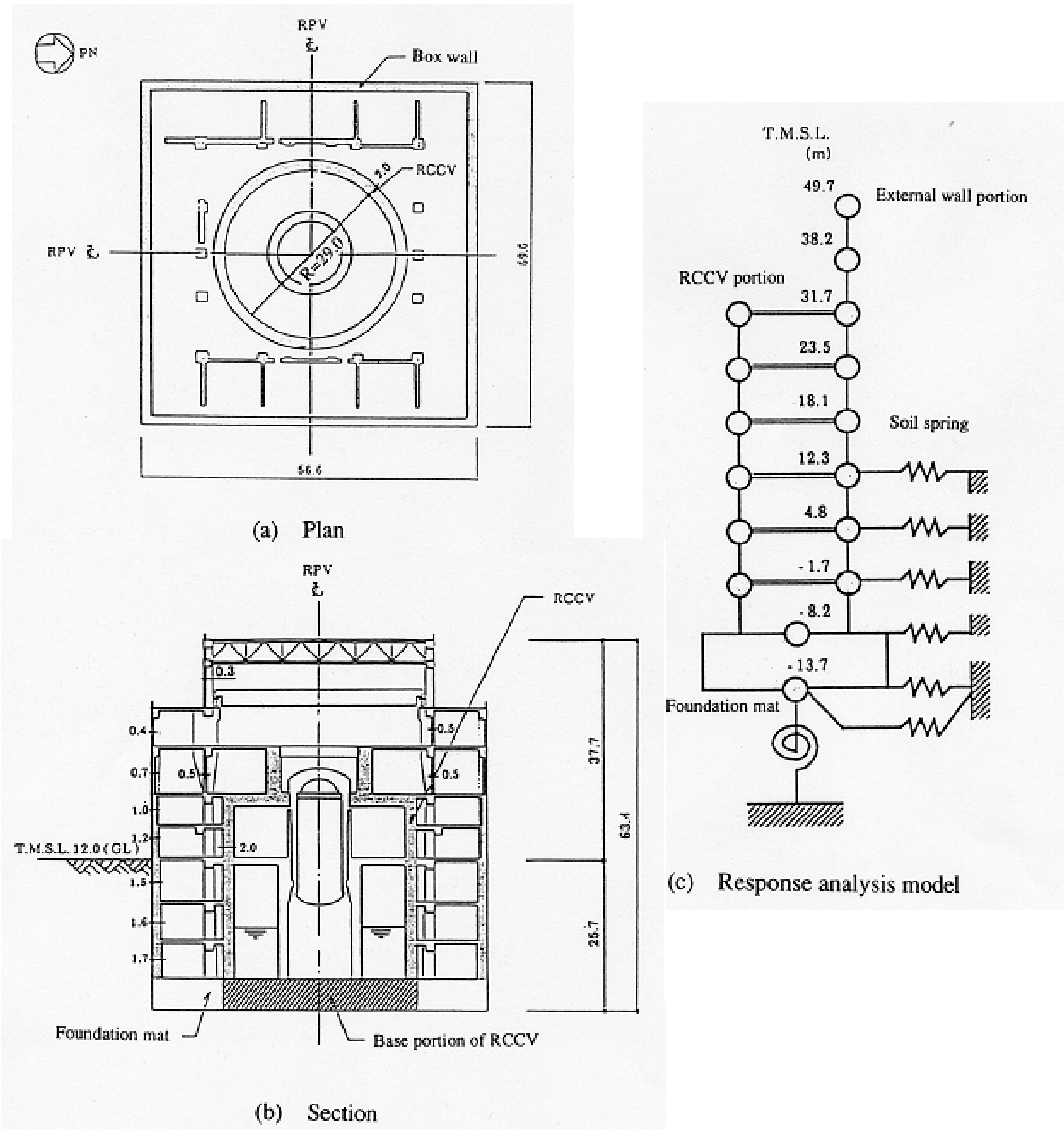


FIG. 2. A typical earthquake response analysis model of a BWR type reactor building [23].

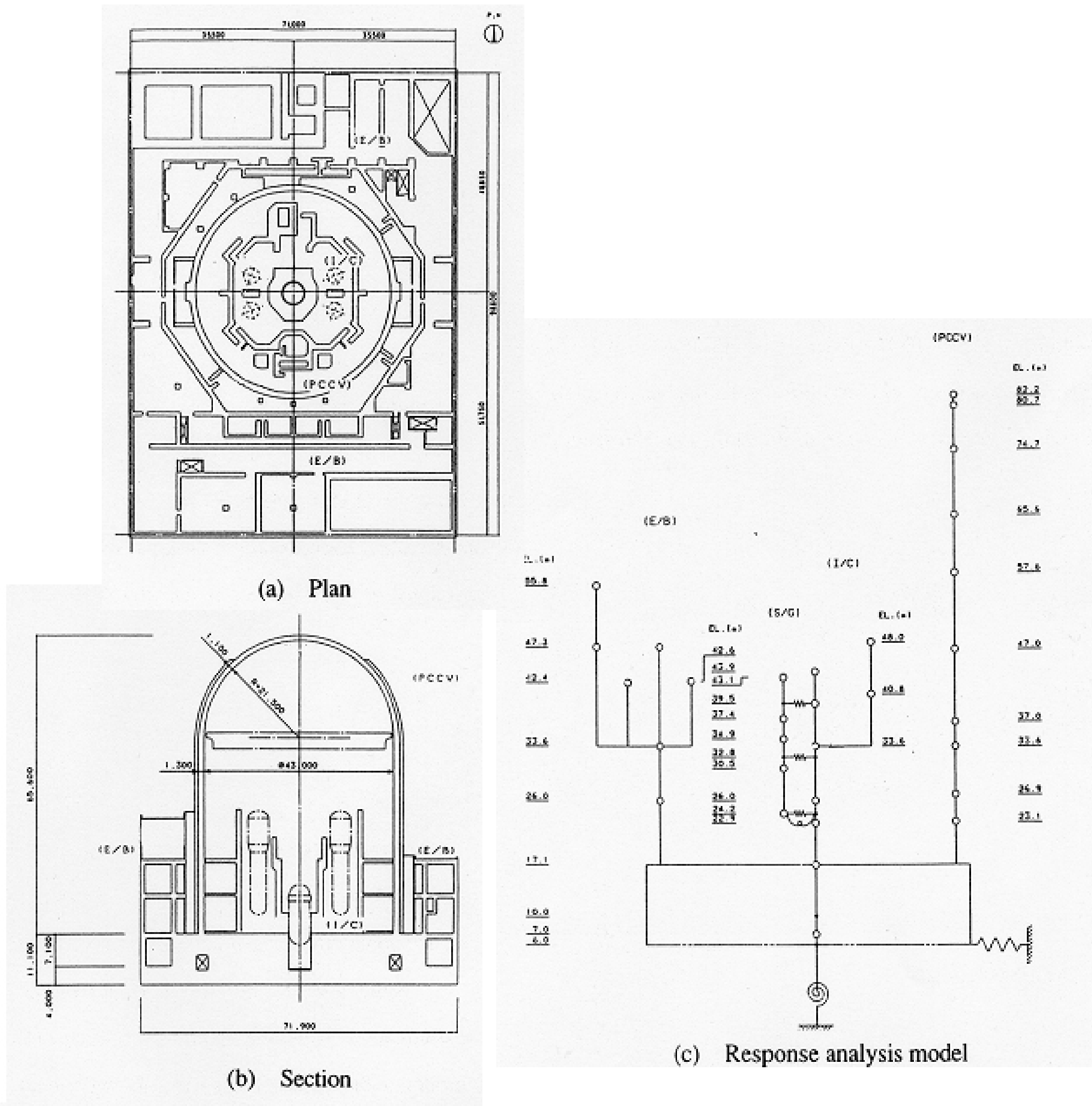


FIG. 3. A typical earthquake response analysis model of a PWR type reactor building [23].

In order to keep rational safety margins in the earthquake response of the safety related important structures, the base-mat uplift phenomenon must be incorporated in earthquake response analysis of structures. A linear response analysis is applicable when the base-mat contact ratio is larger than 75%. A nonlinear analysis can be applicable when the contact ratio is between 65 and 75%. For a contact ratio of less than 65%, a sophisticated detailed analysis or some design change is required to improve the contact ratio so that it becomes larger than 65%.

Likewise, for the shear deformation of RC shear walls, it is proposed that the strain level of each shear wall element be smaller than 2000 μm. The value is determined as a half value of the ultimate deformation value of 4000 μm, which is evaluated from many experimental data of RC shear walls by taking into account the variation and rational margins (see Annex XI). Figure 4 shows an example of maximum response shear strain of a five storey actual reactor building. The number that is circled in this figure corresponds to the story of the building model.

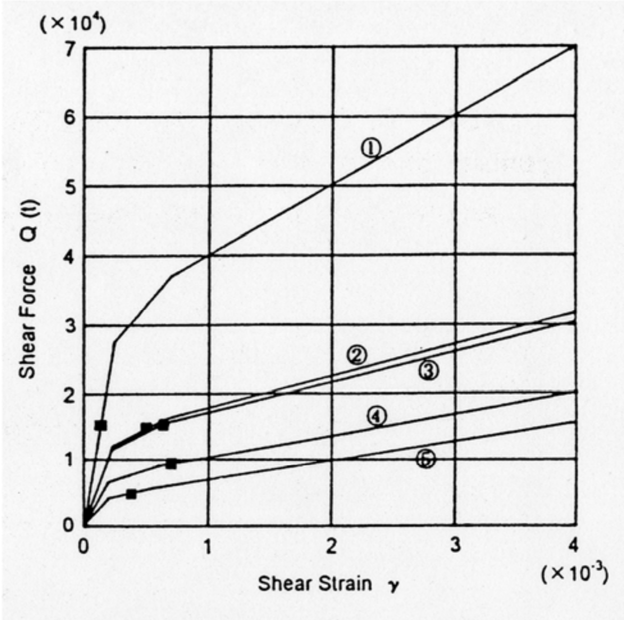


FIG. 4. An example of maximum response shear strain of an earthquake response analysis of a reactor building [24].

The current Japanese 'Examination Guideline for Aseismic Design of Nuclear Power Reactor Facilities' was upgraded in 2006 to enhance the reliability of seismic safety of nuclear power plants based on the latest knowledge of earthquake ground motions and their damaging effects. In this document upgrading, one of the major points relates to design ground motions. The new design ground motions are the 'Earthquake ground motion for use to confirm the designed safety function (Ss)' and the 'Basic design earthquake ground motion (Sd)':

- Safety functions of all Class S SSCs are required to be checked against Ss ground motion. The Class S SSC is also newly introduced to unify and replace the conventional seismic classes A and As. In other words, conventional SSC Class A was upgraded to meet Class S;
- The Sd earthquake ground motion is introduced for the purpose of checking the elastic design of SSCs. It almost follows the standard ground motion S1 as it is in the previous version of the 'Examination Guideline'.

2.3. Experimental results on walls

2.3.1. Japanese experience

The JNES conducted a project entitled Model Tests of Multi-axes Loading on RC Shear Walls to clarify the effects of multi-directional seismic load on the ultimate strength of RC shear walls to confirm the validity of the current aseismic design procedure of the RC buildings in nuclear power plants under simultaneous 3-D earthquake excitation. The project was performed for ten years (fiscal years 1994 to 2003 in Japan) with a subsidy from the Ministry of Economy, Trade and Industry of Japan [25]. Two kinds of static and dynamic loading tests were performed. The static loading test was performed to study basic characteristics of RC shear walls under the multi-axes loading condition by applying the static loads using oil-jacks. For this test category, typical tests performed were the element test using RC plate specimens, the simultaneous horizontal and vertical loading test using box-type RC wall specimens, and the simultaneous horizontal cross directional loading test using box-type and cylindrical RC wall specimens. The dynamic loading test was performed using a 3-D shaking table to confirm whether or not the various characteristics of RC shear wall found under the static loading tests can also stand under the dynamic loading condition.

2.3.1.1. Outline of the test

2.3.1.1.1. Static loading test

Twelve specimens were tested with different reinforcement ratios and applied axial forces to study the shear transfer constitutive law of RC walls when they were damaged (parallel cracks) by the loads applied in the out of plane direction. Each specimen was 1.2 m × 1.2 m and 20 cm thick. The concrete design strength was 30 MPa and the yielding strength of the rebar was 345 MPa. The main rebar arrangement is double and orthogonal with a pitch of 150 mm. Each specimen was first out under a tension of up to 3.0 MPa to generate horizontal parallel cracks which simulating cracks generated by an out of plane bending moment then the tension load was unloaded and a specific axial stress was applied. Keeping the axial stress, the specimen was cyclically applied shear stress increasingly up to the concrete failure.

Two specimens were tested to evaluate the influence of vertical load on the horizontal restoring force characteristics of RC shear walls. The four faces of each specimen were 1.5 m wide, 1.0 m in height and 75 mm thick. The rebar of the specimens had yield strength of 345 MPa and was deployed in the vertical and transverse directions with a double-fold with a pitch of 70 mm. The reinforcement ratio was equivalent to 1.2%. The shear span ratios of two specimens tested were 0.8. The test was carried out by simultaneously applying static loads both in the horizontal and vertical directions.

Six specimens, four box-types and two cylindrical-types, were tested with different loading patterns to study the nonlinear behaviour of RC walls under simultaneous horizontal two directional loadings. The dimension, rebar ratio and concrete compression strength of the specimens were the same as those of the simultaneous horizontal and vertical loading test. Some loading patterns, e.g. rectangular and circular, were determined unrealistically severe to study the applicability of FEM analysis for the simulation. Figure 5 shows a loading test run before wall collapse.

The vertical stiffness of the specimens decreased under the fluctuating vertical load in the range of ±1 g. The decrement was mainly caused by cracking in the wall due to the horizontal load. The influence of the vertical load fluctuation on the stiffness in the horizontal direction was relatively small. Figure 10 exemplifies the effect of vertical load on the restoring force characteristics in the horizontal direction by showing test data and the skeleton curve calculated based on the method in JEAG-4601 [22] with the static vertical load of upper (2.94 MPa) and lower (0.0 MPa) bounds corresponding to a vertical response acceleration of +1.0 g and –1.0 g, respectively.

The shear deformation angles of the specimens at the ultimate states exceeded 4/1000 rad both in the horizontal two directions.

Although the restoring force characteristics of an RC wall deteriorated due to the damage in the crossing walls, the effects on the wall capacity was negligibly small when the damage was not so severe as to reach a concrete shear failure. Figure 11 shows envelope curves of the relationships between shear force and total deformation angle for three box-type specimens together with a typical 1-D loading test result for a box-type specimen. Each curve is normalized by ${}_{c}Q_{JEAG}$, calculated 1-D maximum shear strength based on JEAG-4601 [22]. The figure indicates that up to a deformation angle of 4×10^{-3} rad, the envelope curves of 2-D loading are higher than those of 1-D loading.

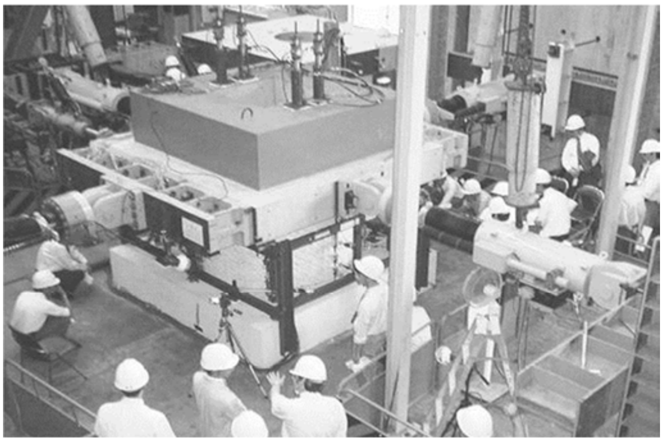


FIG. 5. Simultaneous horizontal two directional loading test.

FIG. 6. Dynamic loading test showing a specimen and the shaking table.

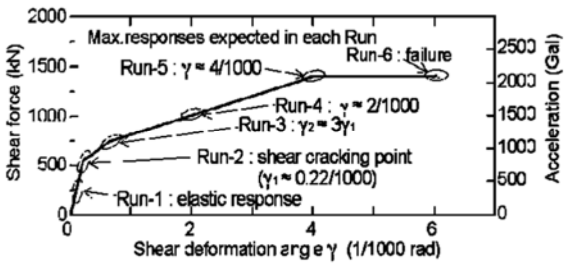


FIG. 7. Dynamic loading plan.

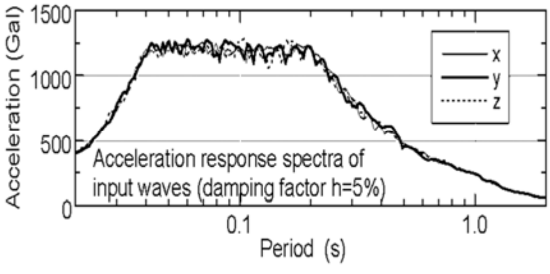


FIG. 8. Response spectra of the input motion used in the test.

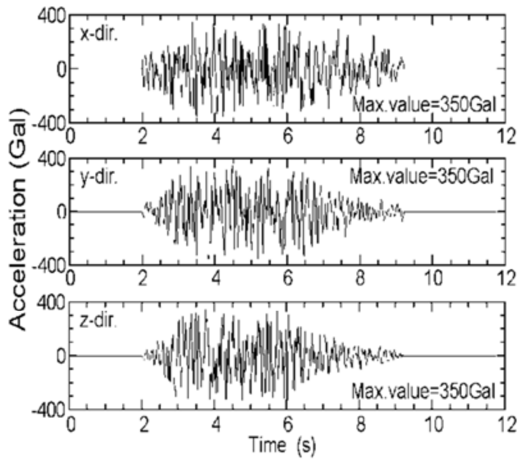


FIG. 9. Time histories of the input motion.

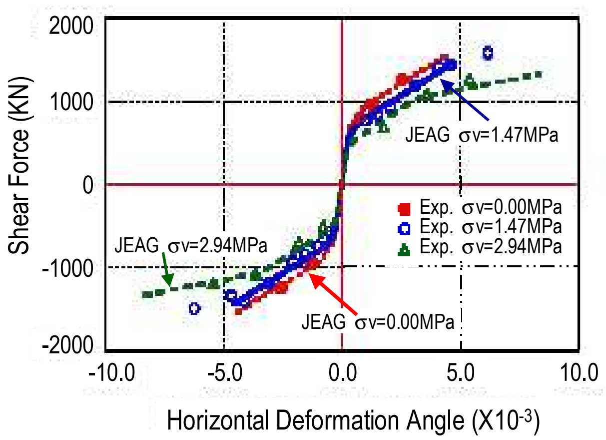


FIG. 10. Comparison of envelope curves.

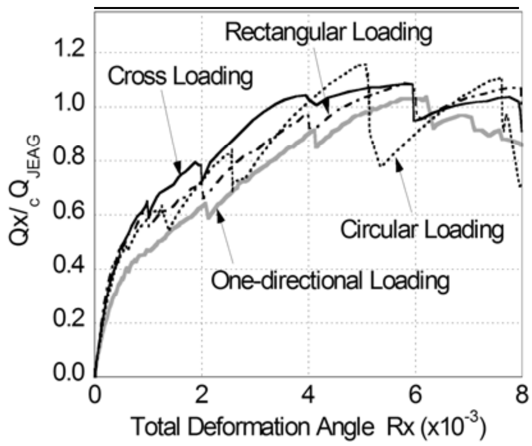


FIG. 11. Comparison of envelope curves.

2.3.1.1.2. Dynamic loading test

Three specimens, two box-types and one cylindrical-type, were tested. The basic specifications of the specimens were the same as those used in the simultaneous horizontal loading test. Figure 6 shows a photo of the dynamic loading test of a box-type RC wall. Figure 7 shows the basic dynamic loading plan for the test from elastic response to shear failure. Figures 8 and 9 show the response spectra and the applied input motions in the three different directions, respectively. These motions are applied simultaneously but the maximum acceleration of the vertical motion is set as the half level.

Figure 12 shows an example of a test result, namely the relationship between maximum acceleration and horizontal deformation angles. In the figure, three skeleton curves calculated with constant values of axial compressive stress of the walls σ_v = 1.47, 2.94 and 0.0 MPa corresponding to a vertical response acceleration of +1.0 g and –1.0 g, respectively, are shown for the purpose of easily imaging the varying axial force applied by vertical input motion. Good consistency was achieved between the test results and calculated skeleton curves in the region of weak nonlinearity. A summary of the test results is given below:

All three specimens reached the deformation angle of 6/1000 rad before collapse. The deformation angle of the cylindrical specimen was almost the same as that of box-type specimens, far smaller than that observed in the static loading test for the cylinder-type specimen. One reason for the smaller value is attributed to cumulative damages at the boundary between cylinder wall and its base slab due to repetitive loading tests.

The RC shear wall characteristics found in the 3-D static loading test were also found in the 3-D dynamic loading test.

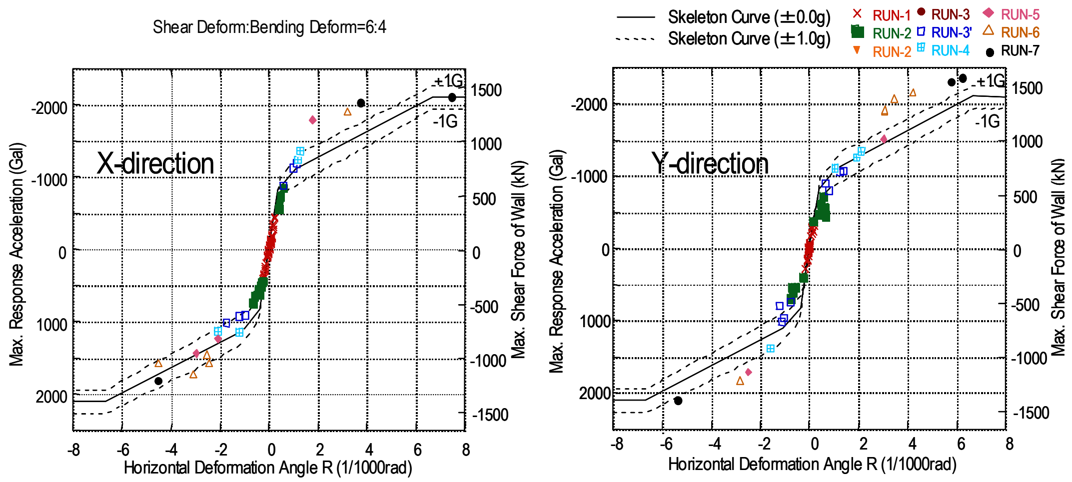


FIG. 12. An example of the dynamic loading test results compared to the skeleton curves under different compression stress calculated with the current Japanese practice.

2.3.1.1.3. Analytical results

Although the hysteretic loop of the restoring force became complex due to the randomness of the motions applied in the dynamic loading test, nearly the same hysteretic loop was obtained analytically by introducing an equivalent damping to the area of each step-by-step hysteretic loop to the analytical specimen model.

The dynamic analysis performed by applying simultaneous horizontal and vertical motions to the specimen model indicates that the influence of the vertical motion on the nonlinear behaviour of the specimen model in the horizontal direction is negligibly small. The reason is that the dynamic vertical motion (load) in the analyses is much faster than that of the static loading test.

An analytical methodology using the lumped mass model in JEAG-4601 [22], which applies the dynamic load in one direction, can be applied in the range of the shear deformation angle equal to or less than 2×10^{-3} rad.

In the range of a shear deformation angle exceeding 2×10^{-3} rad, the nonlinear response of an RC structure as well as its hysteretic curve for the restoring force to the multi-axes loading can be evaluated with satisfactory reliability by applying FEM analysis together with the four-way crack model [26]. In this range, a lumped mass model is also applicable by taking into account vector shear forces modelled by the skeleton curve in JEAG-4601 [22].

2.3.2. The US experience

The American Society of Civil Engineers (ASCE) motivated by various contentions that the actual stiffness of typical shear walls in nuclear power plant structures was less than, and some contended much less, than the theoretical stiffness, initiated an evaluation programme of all available experimental data to assess the effect of ‘reduced stiffness’ for low rise RC shear walls [27].

One of the initiating events of this effort was the reporting of results from the first stages of the US NRC Seismic Category I Structures Program, performed at Los Alamos National Laboratory, Los Alamos, NM, USA in 1984. The Seismic Category I Structures Program was initiated in 1980 and continued to a final report issued in 1993 [28].

The ASCE working group embarked on the following: (1) Collecting any available data on the stiffness of RC shear walls with an emphasis on data from real structures; (2) Careful review of the data; and (3) Developing a position paper addressing the issue of reduced stiffness for the seismic analysis of nuclear power plant structures. The focus was on seismic analysis and the calculation of in-structure response spectra (ISRS) — the question of the strength of the shear walls was not in doubt but rather the question of the frequency characteristics of the structure as modelled for the generation of ISRS for design, qualification and evaluation of sub-systems.

In the same time frame, the technical review group (TRG), formed to review the ongoing activities of the team performing the Seismic Category I Structures Program, provided valuable recommendations for the modification of the test plans post-1984 results. The extensive testing program of the Seismic Category I Structures Program is summarized in the Table 3.

TABLE 3. GEOMETRIC CHARACTERISTICS OF THE TEST STRUCTURES (Table V, Ref. [28])

Type of structure[a]	Wall thickness (in)	Aspect ratio[b] height:length	Types of tests[c] (No. tested)	Percent reinforcement each direction
ISW, 1-STOREY	1.0	0.42	SM(2), SC(3), SSW(4), SS(2)	0.56
ISW, 2-STOREY	1.0	0.42	SS	0.56
DGB, 1-STOREY, (1/30 SCALE)	1.0	0.40 0.73	SM(9), SC(2), SS(2)	0.56
DGB, 2-STOREY, (1/30 SCALE	1.0	0.73	SS(3)	0.56
DGB, 2- STOREY (1/10 SCALE)	3.0	0.73	SS(2)	0.56
AB-1/42, 3-STOREY (1/42 SCALE)	1.0	0.38	SS	0.56
AB-1/14, 3-STOREY (1/14 SCALE)	3.0	0.38	SS	0.56
TRG-1	1.0	1.0	EMA, SM, SS	0.56
TRG-3	4.0	1.0	EMA, SM, SS	0.61
TRG-4	6.0	1.0	EMA, SC	0.25
TRG-5	4.0	1.0	EMA, SC	0.61
TRG-6	6.0	0.27	EMA, SC	0.50
TRG-7 through -11	2.0	1.0	EMA(5), SS(5)	0.24
TRG-12	2.0	1.0	EMA, SC	0.24
TRG-13	2.0	1.0	EMA, SC, SS	0.24
TRG-14, -15	2.0	1.0	EMA(2), SS(2)	0.24
TRG-16	2.0	1.0	SC	0.24

[a] ISW = isolated shear wall; DGB = diesel generator building; AB = auxiliary building.
[b] For = multistorey structures — the aspect ratio reported is for an individual floor.
[c] SM = static, monotonic; SC = static, cyclic; EMA = experimental modal analysis; SS = simulated seismic input; SSW = sine-sweep input.

Initially, scale model tests were performed on individual walls (one and two storey) (ISW), and representations of a diesel generator building (DGB) and an auxiliary building (AB). Those structures and tests denoted TRG were the recommendations of the TRG, and define other model test structures and the associated static and dynamic tests performed. The results of these tests generally showed, for low stress levels denoted NBSS (nominal base shear stress) and very carefully controlled test specimens, that the initial stiffness of the RC scale model structures and structural elements approximated the linear strength of material or finite element approach to stiffness based on the design material properties. Two factors that were emphasized in the conclusions of this effort were the importance of any existing cracking (shrinkage, other curing, structure settlement, etc.) and the importance of foundation deformation and rotation.

The ASCE working group assimilated this information along with the widest range of experimental data known at the time to establish a position on the treatment of stiffness reduction for shear wall structures with an NBSS less than about 150 to 200 psi. Table 4 summarizes the extensive data that the ASCE working group considered.

Table 4 summarizes experimental data from the USA and other countries. All data were taken into account in the ASCE working group deliberations.

The result of this effort was to recommend upper and lower estimates of shear wall stiffness to be used in the seismic analysis of shear wall structures for the generation of seismic response, in particular, ISRS. Scale factors to be applied to the shear stiffness of shear walls were calculated, assuming the design values for f'c, were 1.25 and 0.75, for the upper and lower estimates, respectively.

TABLE 4. SUMMARY OF SHEAR WALL TEST STRUCTURES AND LOADING CONSIDERED BY THE ASCE WORKING GROUP

Summary of shear wall test structures and loading							
Structure	Static, cyclic	Static monotonic	Dynamic impulse	Dynamic random	Dynami csine-sweep	Dynamic, stimulated seismic	Dynamic ambient
Isolated shear walls	[34], [36], [38], [41], [42], [44], [48], [49], [50], [61], [62]	[34], [41], [48]		[48]	[48]	[48]	
Shear walls with end walls	[35], [42], [54], [55], [56], [57], [60]	[35], [52], [53]		[52], [53], [57]		[52], [53], [57]	
Shear walls framed into beams and columns	[37], [43], [45]	[29], [30], [31], [32], [33], [39]	[32], [33]				
3-D model of shear wall structures	[31], [40], [54], [60]	[50], [52], [53]		[50], [51]		[50], [51]	
Actual shear wall structures							[46]

A parallel effort by EPRI to evaluate the issue of reduced stiffness for shear walls was conducted by Sozen and Moehle [64]. Reference [64] presents a summary of their evaluations. Sozen and Moehle reviewed experimental data from a number of sources and correlated analysis results. Generally, comparing measured values of initial stiffness with linearly calculated values, the report concludes that:

- The ratio of measured versus calculated stiffness values were in a range about the median value of 0.7. This was based on measurements of lateral displacement at the point of load application.
- For a subset of experimental test data where strain data was captured and allowed another measure of initial stiffness to be made eliminating the effects of base rotation and minimal cracking at the base, the ratio of measured to calculated stiffness values was close to one. This emphasized the importance of base connectivity and possible very minor cracking at discontinuities of stiffness such as at the base. These considerations were emphasized as being crucial to the interpretation of test results.
- Generally, differences in initial stiffness were due to flexural behaviour rather than shear behaviour, especially the effects introduced due to boundary conditions at the support level of the experiment.

The study evaluated the effect of stiffness assumptions on calculated ISRS and the need to account for stiffness softening on the ISRS. Finally, the report presents data on the increase in compressive strength over time (up to 20 years) by a mean value of 30%. This fact partially offsets the stiffness softening phenomena. Significant interaction between the ASCE working group and the EPRI effort took place.

The evolution of the recommendation for the treatment of the stiffness of RC structure elements has been through a general cautionary note in ASCE 4-86 and ASCE 4-98 to the current requirements in ASCE 43-05 to explicitly account for cracking as a function of behaviour to be modelled (e.g. shear, flexure, axial) and element configuration. For shear walls, for cracked sections, shear and flexure stiffness should be modelled for dynamic seismic analyses assuming half of the design values, i.e. a reduced concrete Young’s and shear modulus equal to half of the code specified value.

2.3.3. The European experience

2.3.3.1. The SAFE programme

The European country that has invested most in the nuclear industry is France. EDF (Electricité de France) and COGEMA (Compagnie générale des matières nucléaires) commissioned an experimental programme on shear walls typical of nuclear power plants from the European Laboratory for Structural Assessment (ELSA) of the JRC of the EC [65].

The experimental campaign consisted of a series of 13 pseudo-dynamic tests on walls in shear. The boundary conditions are realized to assure that the upper beam remains horizontal during the tests in order to simulate the vertical continuity of the walls.

The stiff wall is placed in front of the reaction wall and attached to hydraulic actuators connected to the pumping system. The experimental set-up is shown in Fig. 13 and typical crack patterns

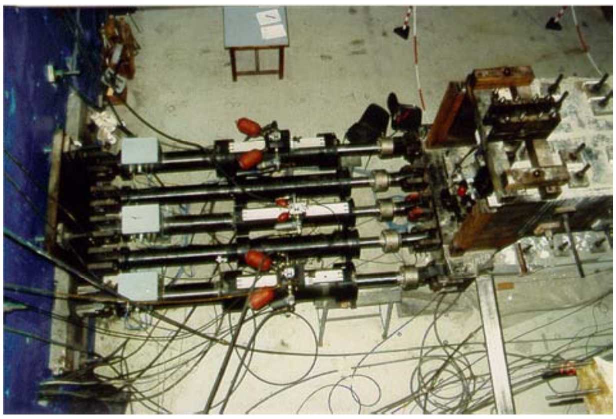


FIG. 13. SAFE experimental set-up.

FIG. 14. SAFE crack patterns after testing.

The first four tests were performed on 16 cm thick walls to optimize the system for the simulation of the effective conditions of the walls and to choose the most appropriate algorithm to be used for the temporal integration of the pseudo-dynamic method.

The following eight tests were performed on 20 cm thick walls with differences in reinforcement proportions in the walls, while elastic system eigenfrequencies and average vertical loads were maintained constant during the tests.

The final test is a repetition of one of the previous tests but for additional reinforcement obtained using vertical and horizontal bands of fibre composite material. This was performed to have preliminary information on the strengthening effectiveness of such material that could also be used for reinforcement/repairing purposes.

2.3.3.2. The ICONS programme

Walls consisting of two or more rectangular parts (i.e. a T-shaped, L-shaped or U-shaped section) are quite common in earthquake-resistant concrete buildings. Generally, they provide stiffness and resistance in both directions, and their behaviour under seismic actions is more complex than that of rectangular walls.

An intensive experimental programme [66, 67] and numerical investigations were carried out with the particular concern that modelling activities and design concepts complement each other and have some implications for the relevant provisions of Eurocode 8 (EC8).

The tests aimed at studying the effect of the arrangement and quantity of vertical and confining reinforcement on U-shaped wall cyclic deformation capacity in uniaxial and biaxial bending, and at assessing the U-shaped wall design process.

Shaking table and cyclic tests on EC8 designed U-shaped walls were conducted at the Commissariat à l’Énergie Atomique (CEA) in the Saclay Nuclear Centre in France (CEA-SACLAY) and ELSA laboratory as part of the TMR (Training and Mobility of Researchers) of the fifth topic ‘shear wall structures’ of the European research programmes ICONS and ECOEST II.

Overall, the seismic and cyclic tests on U-shaped walls confirmed the design and adequacy of the confinement rules of EC8, even if only partly in some cases, due to the insufficient ductility of the steel. The numerical study has demonstrated the capability of the numerical technique to essentially reproduce the observed experimental behaviour. Although the analysis failed to capture some local effects, such as bond slippage of the reinforcement and localized cracking, etc., the loss in accuracy is not severe.

2.3.3.3. The CAMUS programme

Since the beginning of the 1990s, several experimental campaigns on RC bearing walls under seismic loading have taken place in France in the framework of French and European research programmes.

The main objectives of the research related to the CAMUS (Conception et Analyse des Murs sous Séisme) programme were checking the ability of the existing numerical models to evaluate the global parameters of the response at different performance levels and supporting EC8 by studying the behaviour of the EC8 designed wall at different performance levels of loading.

The seismic tests were performed on the major Azalée shaking table of CEA-SACLAY [68, 69]. Between 1996 and 2002, four 1/3 scaled specimens named CAMUS I to IV made of two RC walls and six floors with different steel reinforcement and boundary conditions were tested under in-plane seismic loading.

The behaviour of structural walls in earthquakes is complex and the near collapse stage is very difficult to predict and many problems have yet to be solved. This limited knowledge has been reflected in the design codes, including the latest European seismic standard EC8.

Two benchmarks have been organized on the basis of the CAMUS I and CAMUS III experiments (walls fully fixed to the shaking table with a reinforcement designed in accordance with the French code PS92 and EC8, respectively) [70].

Benchmark prediction and post-experiment analyses of the inelastic seismic response of the RC structural walls have been performed. Blind predictions were conducted before the execution of the test campaign and, after the experimental results had been published, the models were calibrated for the best possible estimation of the results [71, 72] and the understanding of the limitation of the actual behavioural models and numerical codes.

2.3.3.4. The SEISPROTEC programme

Many experimental programmes have been run in Europe on masonry shear wall, which are typical of the European cultural heritage and historical buildings. Masonry shear walls are also of interest for buildings of the nuclear industry and the available experimental results can be helpful for the design and the vulnerability assessment.

One important experimental research programme was run at ELSA of the JRC of the EC [73]. The objective was to improve the understanding of the behaviour of masonry through laboratory tests on shear resistant masonry panels and accurate numerical simulations.

The research programme was intended to provide high quality output for the validation of theoretical models of damage for the improvement of the numerical models. It also contributed to the development and validation of European norms applicable to masonry walls.

The experimental campaign was performed on various panels in brick masonry about 115 cm in length and 150 cm high for a thickness of 25 cm as shown in Fig. 15.

The tests' set-up was organized to induce shear damage through vertical constant loading and horizontal variable loading. The boundary conditions imposed to the upper face of the wall were such that it remained horizontal during the test to simulate continuity of the structure.

The tests mainly focused on monotonic and cyclic horizontal loading to obtain, as far as possible, high precision results for the validation/calibration of damage behavioural models, which play an important role in the masonry structures vulnerability assessment.

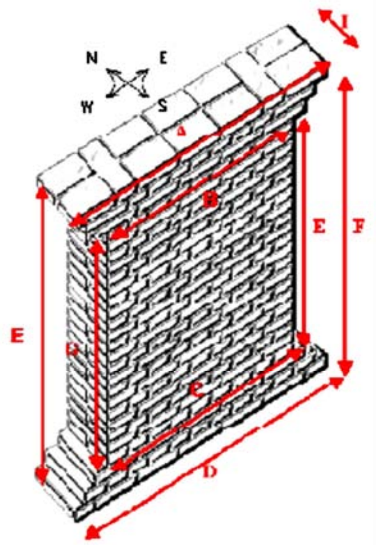


FIG. 15. SEISPROTEC: Geometry of the walls.

2.3.3.5. Validation of numerical models

Many theoretical and experimental studies have addressed the problem of including flexural failure modes in the modelling of structural shear wall (SW) elements [74, 75]. The determination of the shear wall model characteristics requires experimental results obtained from a cyclic test of SW walls and beams. Extensive experimental studies on the behaviour of walls with different aspect ratios subjected to various load conditions have been performed mainly for small/medium scale specimens, typically from 1/2 to 1/3 scale, while experimental evidence on the behaviour of full scale shear walls is presently scarce.

The design tools available for the analysis of RC SW are computationally expensive and require high expertise because of their complexity. Therefore, a simple easy to use methodology is desirable for the design of such elements. Intensive research has been performed to create a realistic conceptual model that captures the nonlinear stiffness and energy adsorbing characteristics of RC structures subjected to strong earthquake motions. The problem of including both crack generation and energy dissipation for cyclic loads in the model, as in the case of earthquakes, has not yet been solved satisfactorily.

2.4. Experimental data for the IAEA CRP

2.4.1. CAMUS experiment

2.4.1.1. Overview of the CAMUS programme

CAMUS is a generic name for a series of experiments that were carried out in France, on the Azalee shaking table at CEA-SACLAY [76]. Three specimens were tested; they were identical from a geometrical viewpoint, and were composed of two parallel five floor walls, representative of a 1/3 scale of conventional RC bearing walls as usually built in France. They differed by the reinforcement design: CAMUS I was reinforced according to the French building code at the time of design; CAMUS II had almost no reinforcement and CAMUS III was reinforced according to provisions of EC8, under preparation at that time [77]. Figure 16 shows a specimen.

In the frame of the IAEA CRP on the safety significance of near-field earthquakes, the terminology ‘CAMUS experiment’ or ‘CAMUS specimen’ refers to CAMUS I data and results which were provided by France as an input for the IAEA CRP.

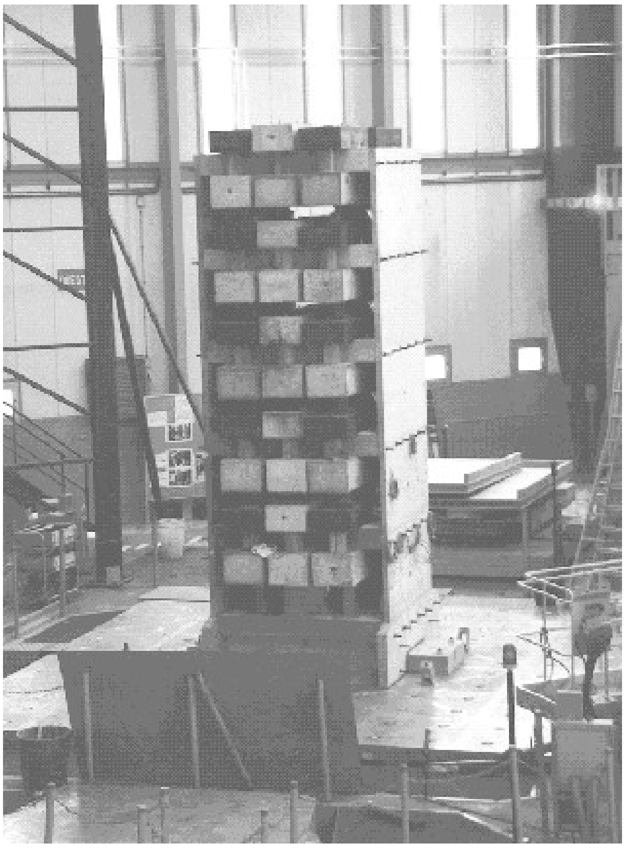


FIG. 16. The CAMUS specimen.

2.4.1.2. Objective and rationale of the CAMUS experiment

The CAMUS experiment was initially designed to provide experimental evidence of margins encompassed in the French practice for the anti-seismic design of conventional shear wall structures. The specimen was designed according to the mandatory design spectrum for the city of Nice (Spectrum S1 in Fig. 17, scaled at a 2 m/s^2 PGA), without accounting for a behaviour factor. The objective of the designers was to get evidence of a 3.5 margin in the design. Thus, the crucial run of the experiment was to be conducted under an input motion fitting the S1 spectrum scaled at 7 m/s^2, designated 'Nice 0.7 g'.

A further objective was to check the relatively low damaging capacity of near-field input motions, recognized by in-the-field experience feedback, and to reproduce the phenomenon in the laboratory so as to provide researchers with quantified data on the subject. Two types of input motions were consistently used:

- The 'Nice input motion' (artificial ground motion fitting Spectrum S1), representative of a far-field motion;
- The 'San Francisco input motion' (natural ground motion), representative of a near-field motion. It is shown in Fig. 18, scaled at 0.1 g in acceleration, without a scaling factor in frequency[8].

[8] See footnote 11.

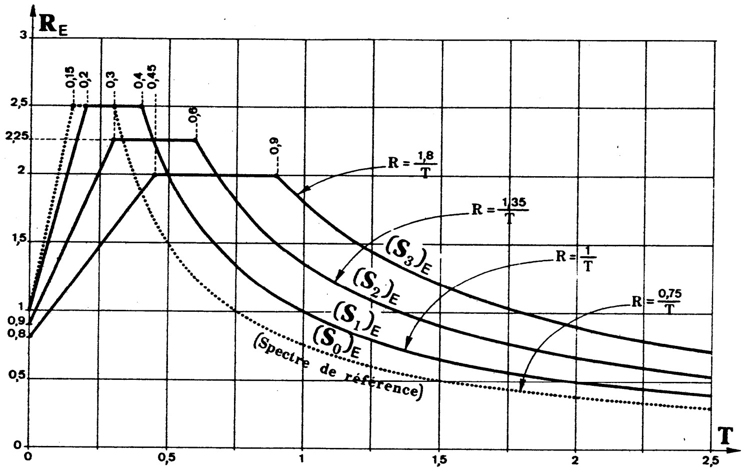


FIG. 17. French regulatory design spectra (valid in the 1990s) for conventional buildings; Spectrum S1 was considered for the CAMUS design.

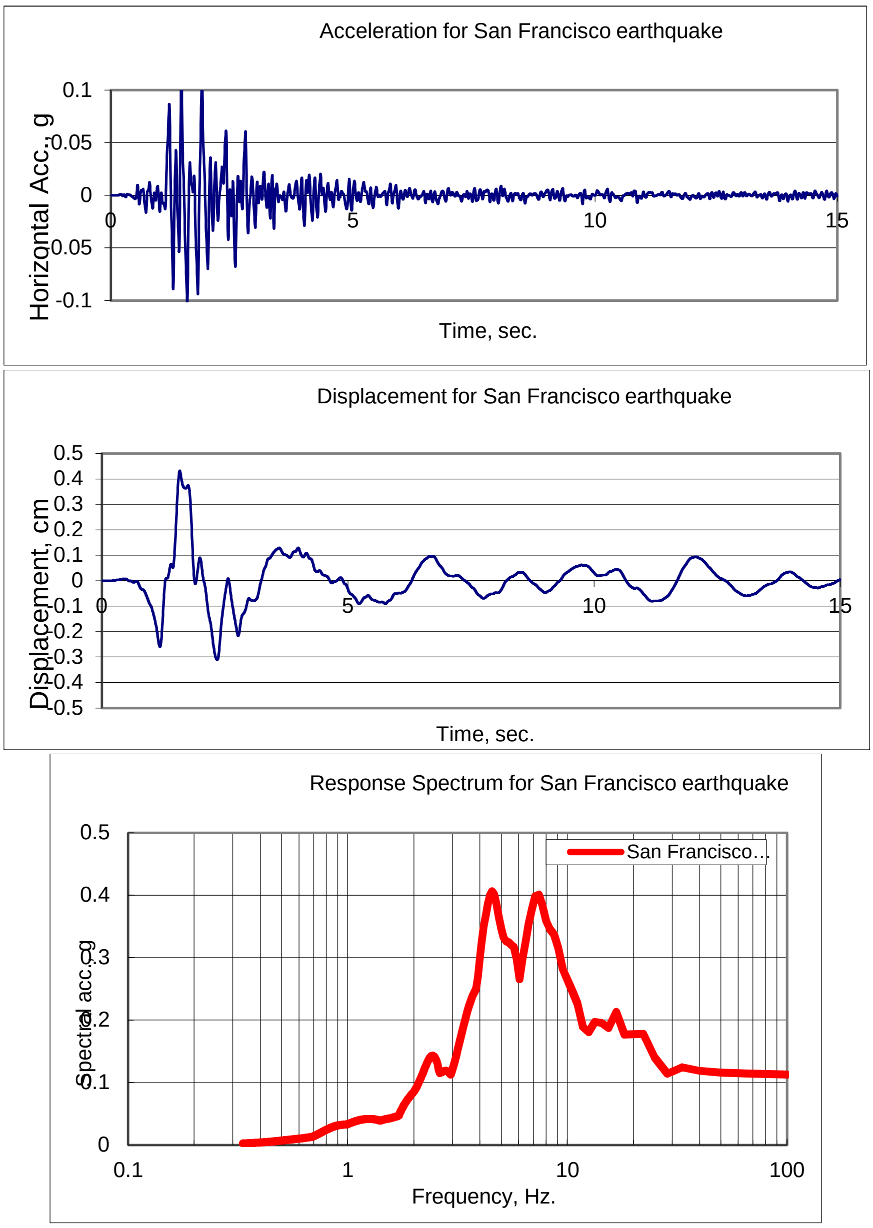


FIG. 18. San Francisco input motion, acceleration, displacement, response spectrum.

According to conventional nuclear practice, and scaled at the same PGA, the San Francisco input motion is deemed to be more damaging than the Nice input. The intention of the CAMUS experiment designers was to provide evidence that this statement is wrong and that, on the contrary, for a given PGA, the near-field type input is significantly less damaging than the far-field type. More precisely, the objective was to substantiate the following statement: "A near-field input scaled at a $\Gamma \times 1.5$ PGA is less damaging than a far-field input scaled at Γ."

It was, therefore, decided to conduct a run with the San Francisco input motion scaled at $0.7 \times 1.5 = 1.05$ g, rounded to 1.1 g.

To conduct the experiment, it was necessary to have a reasonably reliable prediction of the respective damaging capacities of 'San Francisco 1.1 g' and 'Nice 0.7 g'. Two indicators of damaging capacity can be introduced:

- Indicator A: Spectral value of the specimen, which reflects conventional nuclear practice;
- Indicator B: Estimated top displacement of the specimen.

At the time of CAMUS design, the specimen top displacement was estimated on the basis of a methodology that is now codified as a DBA with secant stiffness (presented in Section 4.2.1.4 as the 'substitute structure' method). The resulting estimates were substantiated by time history analyses.

The relative damaging capacity of San Francisco 1.1 g versus Nice 0.7 g is presented in Table 5 according to both indicators. It is clear that they lead to totally contradictory conclusions. According to Indicator A, San Francisco 1.1 g is more damaging than Nice 0.7 g. In contrast, according to Indicator B, San Francisco 1.1 g is much less damaging than Nice 0.7 g.

TABLE 5. PREDICTED RELATIVE DAMAGING CAPACITIES OF SAN FRANCISCO 1.1 g AND NICE 0.7 g

	$\frac{\text{Damaging capacity of San Francisco 1.1 g}}{\text{Damaging capacity of Nice 0.7 g}}$
Indicator A, nuclear practice	1.30
Indicator B, DBA	0.27

Finally, the experiment designers were convinced that the nuclear approach was wrong and they decided to launch San Francisco 1.1 g before Nice 0.7 g. An additional run, Nice 0.4 g, was chosen as a control run before launching Nice 0.7 g. Views of CAMUS designers on the respective effects of far-field and near-field input motions are presented in Ref. [78].

2.4.1.3. Description of the specimen and monitoring

A detailed description of the experiment, as provided to the participants during the kick-off meeting, is presented in Annex III. It includes data on specimen design, specimen construction, monitoring, shaking table features, input motions and outputs of the experiment (displacements, accelerations, forces and moments as well as crack pattern).

The CAMUS experiment [76] consists of two similar parallel shear walls, strongly clamped on a shaking table and subjected in their plane to 1-D horizontal seismic input motions. The specimen is a mock-up at a 1/3 scale of typical shear walls of a five storey conventional building (Fig. 16). A stiff bracing system, which does not carry any vertical load, was installed in the direction perpendicular both to the walls and input motion.

The total mass of the mock-up is 36 t (6 floors × 6 t). The compressive stress in walls under static load is 1.60 MPa. The main wall characteristics are as follows: height = 5.10 m, length = 1.70 m, thickness = 0.06 m. Micro-concrete is used, the Young modulus of which varies in the range of 28 000 to 32 500 MPa and the strength of which in the range of 30 to 38 MPa. The R-bar system is designed in compliance with the French regulation for conventional buildings[9].

Sixty-four channels are available during each run. The horizontal displacement, velocity and acceleration of the shaking table are recorded, as well as its possible rocking and vertical undesired motions. On every floor, displacement and acceleration are recorded in the direction of the input motion. Some additional sensors are also installed so as to capture the vertical and out-of-plane movements. Some transducers are installed to measure crack opening; they provide values of strains on the average length of the transducer (25 cm). A series of R-bars are also equipped with strain gauges.

2.4.1.4. Input motions selected for the IAEA CRP

Practically, the implemented input motion did not exactly meet the experiment designers' intention. After a series of low level runs, strong motions were applied to the specimen. Four strong motion runs were selected for the IAEA benchmark, as shown in Table 6. A last one, Nice 0.72 g, which led to extensive cracks and some R-bar ruptures without collapse, was not selected in the frame of the IAEA CRP because, consistent with considerations developed in Section 2.2.3, the OC decided to "…focus on those small nonlinearity effects that occur without exceeding design criteria."[10] However, it is important to note run 5 because it gives a good idea of the 'ultimate' capacity of the specimen.

TABLE 6. STRONG MOTIONS ACTUALLY APPLIED ON THE CAMUS SPECIMEN

Runs selected for the IAEA CRP				Not selected
Run 1	Run 2	Run 3	Run 4	Run 5
Nice 0.24 g	San Francisco 0.13 g	San Francisco 1.11 g	Nice 0.41 g	Nice 0.72 g

Run 1 may be considered as a typical design input motion. Runs 1 and 2 may be regarded as rather 'low level' inputs. Runs 3 and 4 are moderately damaging level inputs. Response spectra of input motions 1 to 4 are shown in Fig. 19[11].

As reflected in the run titles, the original intention of the designers of the CAMUS experiment was that Runs 1, 4 and 5 be carried out with input motions that have the same spectral shape; only the scaling PGA was supposed to be different. It was the same for runs 2 and 3 with a different shape. This intention was reasonably met for runs 2 and 3 (Fig. 19); their spectral shapes may be regarded as sufficiently close for the purpose of analyses carried out in the frame of this benchmark. Unfortunately, this is not the case for runs 1 and 4 (Fig. 19), apparently due to difficulties encountered with the control system of the shaking table during run 4. The run 1

[9] With some adaptations relating to possible improvements of security factors that were under discussion when the mock-up was under design.
[10] Due to large nonlinearity developed during run 5 and to its specific failure mode (shear failure at level 3), its interpretation would have required a lot of efforts from research teams without real benefit for the IAEA CRP.
[11] The specimen being representative of a wall at 1/3 scale, input ground motions applied on the shaking table are scaled by a factor $3^{1/2}$ in frequency. Response spectra of these scaled input motions, as applied on the shaking table, are shown in Fig. 19.

spectrum exhibits a peak at the natural frequency of the CAMUS specimen while the run 4 spectrum exhibits a valley.

2.4.1.5. Major outputs

Minor damages were reported during runs 1 to 4. It is worth observing that during run 4, which cumulates the damage effect of previous runs, strain values provided by transducers are around 0.1–0.5%, while strains provided by gauges on R-bars are around 15–25%. This illustrates strain concentration that appears in R-bars in relation with crack opening.

Extensive cracking, as well as R-bar ruptures at level 3, finally occurred during run 5. However, the rupture mode was not of the type expected by the designers. Instead of a classical bending moment effect, a shear rupture appeared. This makes run 5 outputs difficult to interpret.

A global indicator of damage is the evolution of the first eigenfrequency of the specimen. It decreases from 7.24 Hz before run 1 to 6.60 Hz after run 4. An ‘apparent’ natural frequency during runs may also be derived from records; it was estimated to be as low as 4 Hz during run 4.

Regarding the response of the specimen, the most representative output is naturally the top displacement, shown in Table 7.

TABLE 7. TOP DISPLACEMENTS OBSERVED ON THE CAMUS SPECIMEN

Runs selected for the IAEA CRP				Not selected
Run 1 (Nice 0.24 g)	Run 2 (San Francisco 0.13 g)	Run 3 (San Francisco 1.11 g)	Run 4 (Nice 0.4 g)	Run 5 (Nice 0.72 g)
7.0 mm	1.54 mm	13.2 mm	13.4 mm	43.3 mm

2.4.1.6. Interest of CAMUS for the resolution of the NFE issue

It was pointed out in Section 2.1.2 that the type of near-field input motion considered in the IAEA CRP is principally characterized by its rather high frequency content. This means that for the purpose of the IAEA CRP, the frequency content of the CAMUS runs is a key factor. This frequency content appears in Fig. 19; however, the concept needs some clarification.

It is clear that under rather strong input motion, some damage appears in the specimen, resulting in a loss of stiffness and consequently in a reduced apparent frequency. This means that, regardless of whether it is qualified as near-field or far-field, the relevant part of a response spectrum corresponds to a range of frequency situated just below the first eigenfrequency of the specimen, practically in the case of the CAMUS specimen in the range of 4–5 Hz to 7.2 Hz.

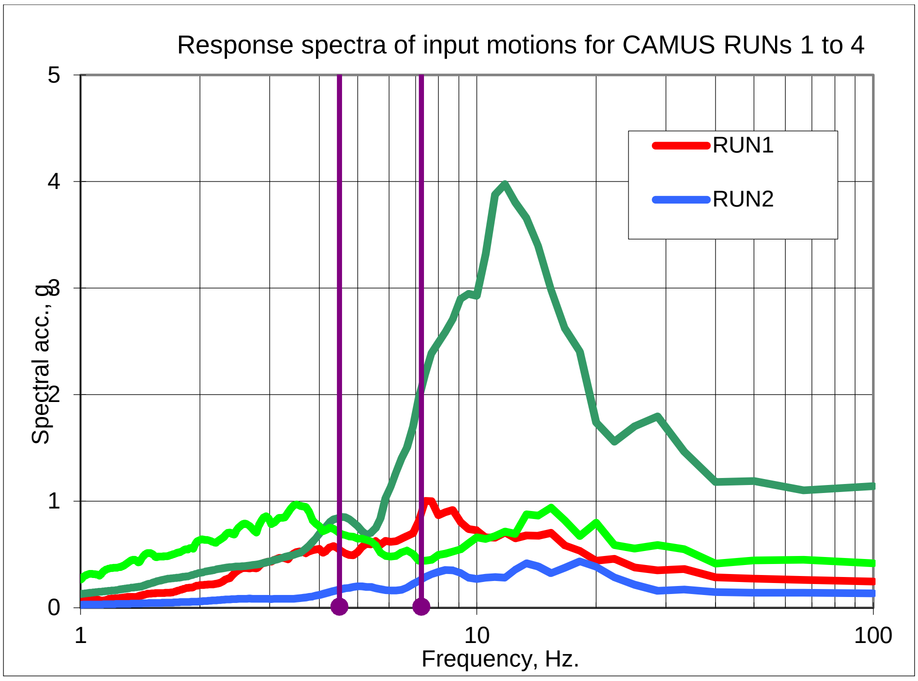
Taking into account this range of interest, it can be seen in Fig. 19 that runs 1, 2 and 3 should be considered as ‘high frequency’ runs (spectral acceleration is an increasing function of the frequency), as opposed to run 4 that should be regarded as a ‘low frequency’ input motion (spectral acceleration is a decreasing function of the frequency). This classification of runs, which pertains to the purpose of the CRP, is summarized in Table 8. In the table, runs 2 and 3 are put in the same column because their spectral shapes are very similar in the frequency range of interest.

In this regard, because of its slope in the frequency range of interest, interpretation of the run 3 outputs presents a major interest. As it is developed further, according to nuclear practice, the acceptable PGA of this input motion is limited to 0.45 g. At the opposite extreme of this prediction, the experiment has provided evidence that the acceptable PGA is higher than 1.11 g.

This experiment illustrates both the low damaging capacity of the near-field input motions (in spite of their possible high PGA) and the very bad predictive capacity of the current nuclear power plant engineering practice in this regard. It is expected that interpretation of CAMUS outputs will lead to an evolution of nuclear power plant engineering practice that will properly predict the damaging capacity of the type of near-field input motions considered in the frame of the IAEA CRP.

TABLE 8. CLASSIFICATION OF CAMUS RUNS THAT PERTAIN TO THE IAEA CRP

Run 1	Runs 2 and 3 (similar spectral shapes)	Run 4
High frequency	High frequency	Low frequency



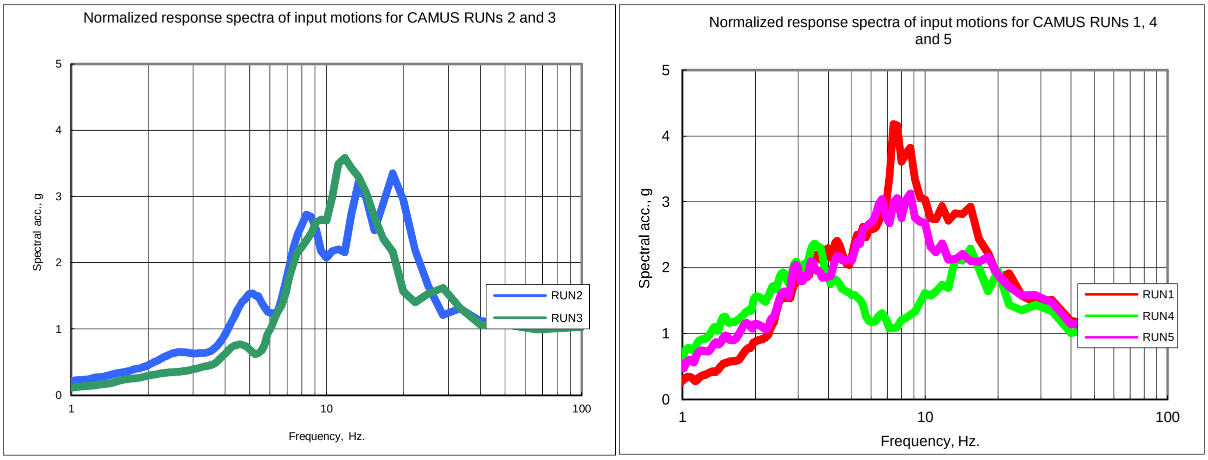


FIG. 19. Response spectra of CAMUS input motions as measured on the shaking table (top) and corresponding non-dimensional response spectra (bottom).

2.4.2. Japanese input motions

2.4.2.1. Japanese database

Since the Great Kanto earthquake disaster in 1923, which killed about 142 000 people, Japan has experienced more than 50 major earthquakes (M > 6.7). The strong motion accelerometer committee (SMAC) was organized in 1951 after the 1948 Fukui earthquake disaster, which killed 3769 people, to develop strong motion measuring instruments, and a SMAC strong motion seismometer was developed in 1953. Since then, many strong earthquake ground motion records have been observed including the 1995 Hyogoken Nanbu earthquake (Kobe earthquake). Some of the observed records were recognized as near-field ground motions. Most of the observations were on soft soil sites, but 23 of them, listed in the Annex IV, were on rock or stiff soil sites.

After the tragic disaster of the Kobe earthquake, several projects were planned and conducted to reinforce the strong motion network. In total, about 2600 seismometers were newly installed. This dense network provided numerous records that can be regarded as near-field ground motions; for instance, the 2000 Tottori-ken Seibu earthquake and the 2004 Niigata Chuetsu earthquake.

Figure 20 shows the peak acceleration contour in the Chugoku district due to the 2000 Tottori-ken Seibu earthquake, the moment magnitude (Mw) 6.6, produced by the National Research Institute for Earth Science and Disaster Prevention (NIED), Japan.

In addition, the 2004 Niigata Chuetsu earthquake of Mw 6.6, had struck 80 km to the south of the city of Niigata, on the west coast of Honshu, Japan. The epicentre of the earthquake was in-land and a PGA recorded at a point of so called near-field (about 10 km from the epicentre) was about 1.7 g.

Furthermore, due to the specific purpose of the CRP, a record of the 1989 Ito-Oki earthquake (Mw 5.3, 5 km to epicentre, recorded on basalt rock site) was selected.

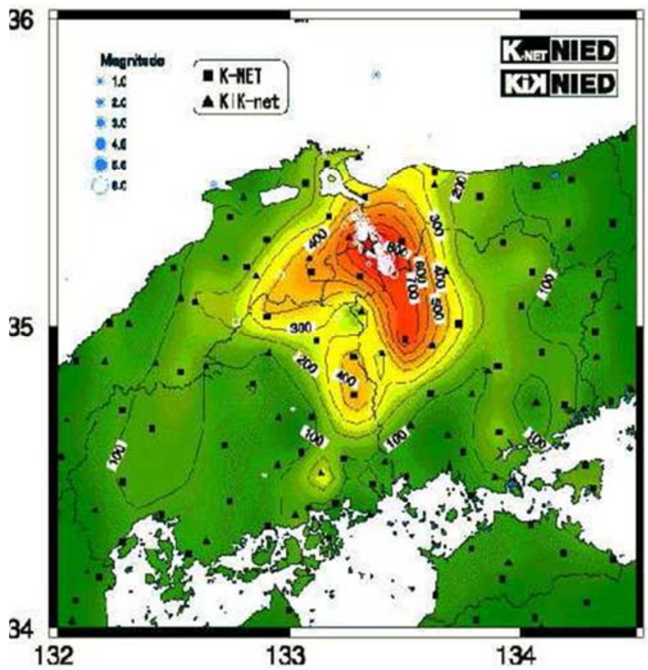


FIG. 20. Distribution of surface peak acceleration during the 2000 Tottori-ken Seibu earthquake [79].

2.4.2.2. Selection of input motions for the IAEA CRP

The set of gathered ground motion records was extensive. A screening procedure was applied on magnitude, PGA, distance to epicentre and focal depth. In addition, records were eliminated because they were recorded in depth, or with a sampling time lag that was too large, or on the basis of other criteria. A first series was discussed during the kick-off meeting (see Annex IV). As a follow-up of those discussions, a new series of 31 records were screened out by JNES (see Annex IV), eight of which, shown in Table 9, were recommended to the organizing committee as candidate records for the CRP.

TABLE 9. RECOMMENDED NFE GROUND MOTION CANDIDATES FOR USE IN THE IAEA CRP

Earthquake	Magnitude Mw	Depth (km)	Observation Ssystem and Point	Epicentral Distance	Seismometer Location(m)	Max. acc.(Gal.)		Max.V.(cm/sec.)	
						NS	EW	NS	EW
1989.7.09 Ito-Oki	5.3	5	Shiofukizaki at rock (Basalt) surface	3(km)	at surface	189.0	189.0	13.8	25.3
1995.01.17 Hyogoken Nanbu (Kobe Earthquake)	6.9	14	JMA Kobe	Seismic Source Area	at surface	818.0	617.3	90.9	74.2
			Kanshinkyo Kobe Unv.		-10.0	270.1	301.1	55.1	31.0
			Shin-Kobe Substation KEPCO*1		at surface	510.7	584.3	62.8	77.7
2000.10.06 Tottoriken Seibu	6.6	11	Kik-NET, Hino	7(km)	-100.0	357.4	574.7	44.1	22.6
			Kik-NET, Hakuta	8(km)	-100.0	185.4	274.2	16.4	26.8
			Tottriken, Gashyo Dam	3(km)	at surface	528.5	531.1	53.3	50.5
1997.03.26 Kagoshima pref.	6.0	12	Sendai NPP KEPCO*2	22(km)	Bldg. Basemat	64.1	63.4	8.4	4.9

*1:KANSAI ELECTRIC POWER CO.,INK
*2:KYUSHU ELECTRIC POWER CO.,INK

After examining the respective spectral shapes and time history patterns of the different accelerograms and discussing their respective interests, the OC selected two of them:

(a) The N–S component of the Ito-Oki earthquake record;

(b) The E–W component of the Tottoriken earthquake, recorded at Kashyo dam.

The interest of (a) is that this input was generated by a medium magnitude earthquake, such as the type aimed at by the IAEA CRP; however, the g level is rather low. The interest of (b) is that it presents a g level comparable to the level of runs 3 and 4 of step 1; the fact that it originated from a rather high magnitude earthquake was discussed and regarded as acceptable and as an opportunity to examine possible effects linked to this feature. The two selected Japanese input motions (acceleration, displacement and response spectrum) are shown in Figs 21 and 22, and their main characteristics are summarized in Table 10.

TABLE 10. MAIN CHARACTERISTICS OF THE SELECTED JAPANESE INPUT MOTIONS

	PGA (g)	PGV (m/s)
N–S component, Ito-Oki	0.19	0.25
E–W component, Kashyo dam	0.53	0.51

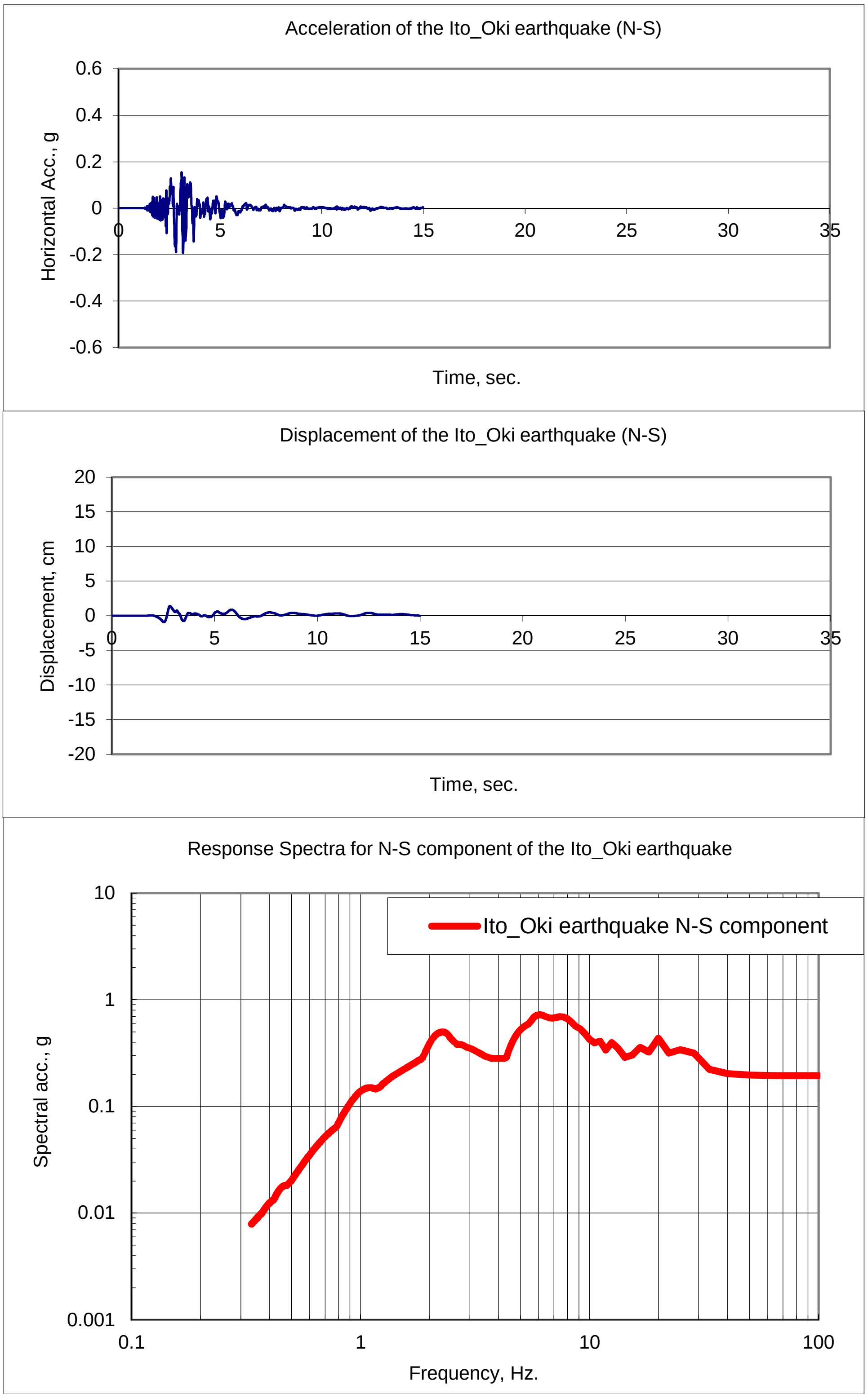


FIG. 21. Ito-Oki input motion: acceleration, displacement, response spectrum.

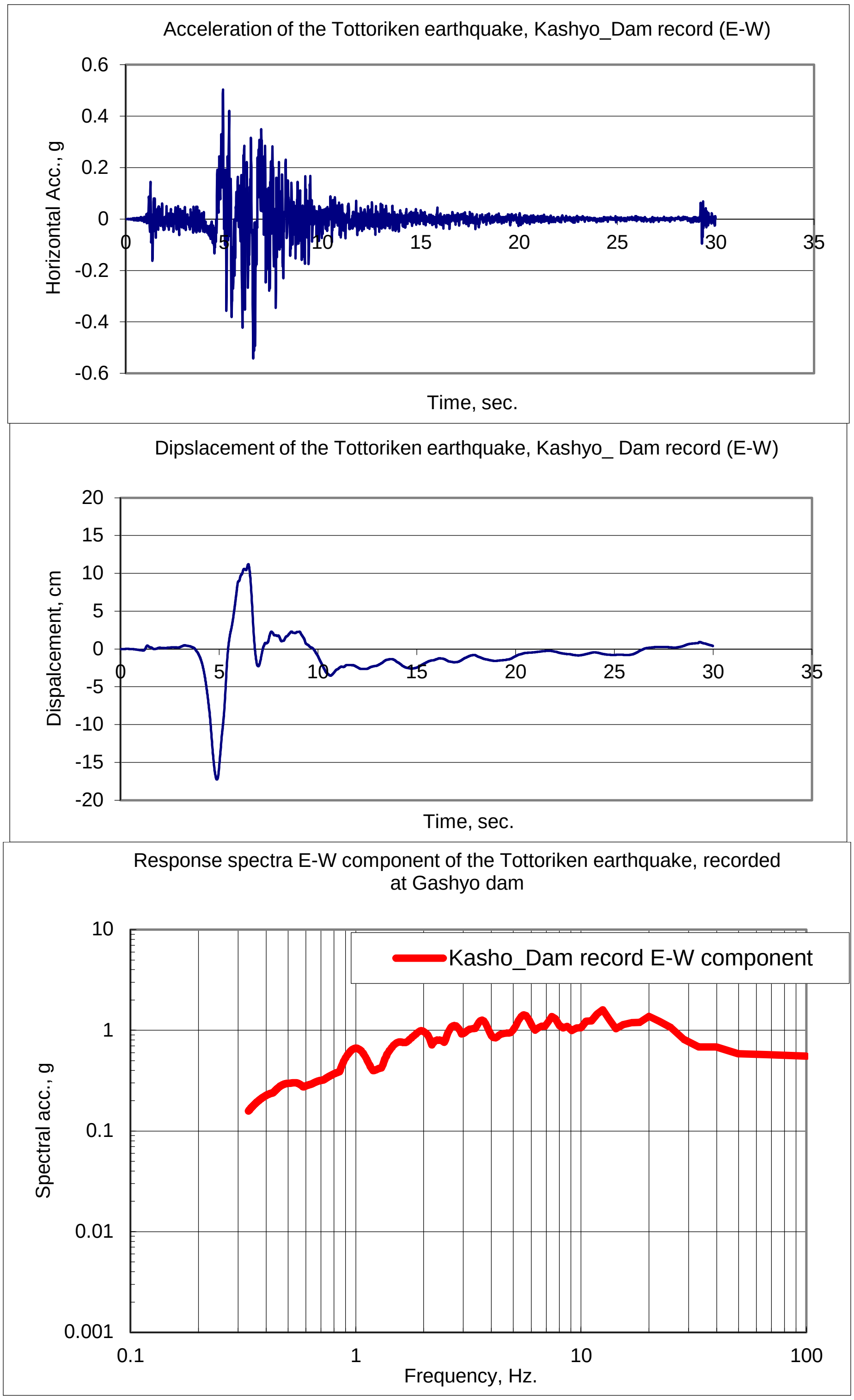


FIG. 22. Kashyo dam input motion: acceleration, displacement, response spectrum.

3. OUTPUTS OF THE BENCHMARK EXERCISE

As mentioned in the introduction, the benchmark was organized in the form of a three step exercise. It resulted in a series of 34 analyses of the CAMUS specimen that participants were requested to carry out. The main characteristics of these 34 different analyses are summarized in Table 11. Along main lines, they consist of the following:

(a) Step 1: Interpretation of the CAMUS experiment.

- (i) A four step pushover analysis;
- (ii) Four conventional response spectrum analyses;
- (iii) Eight displacement based approach analyses;
- (iv) Four time history analyses.

(b) Step 2: Numerical simulations with Japanese input motions.

- (i) Two time history analyses.

(c) Step 3: Effects on floor response spectra generation.

- (i) Six time history analyses ('far-field' input, scaled from 0.1 to 0.6 g);
- (ii) Six time history analyses ('near-field' input, scaled from 0.1 to 0.6 g).

For each step, terms of references of the expected outputs are summarized in Sections 3.1 to 3.3. Details of expected outputs are shown in Annex V in the form that was available to the participants.

Benchmark outputs are presented below in the form of tables and figures with some evaluation and comments. Only a synthesis, meaningful for comparison is shown in the body of the document; a detailed presentation can be found in the Annexes. For the sake of clarity, in every table of results, and where necessary in the narrative, the analysis identification is repeated, as it appears in column 1 of Table 11.

The outputs were initially processed as provided by the participants. Mean values and standard deviations were computed. In a second step, they were processed again in order to reduce some strong discrepancies, resulting in non-representative large standard deviation. In the tables, the maximum and minimum values have been ignored when computing means and standard deviations.

3.1. Step 1: CAMUS experiment

3.1.1. Terms of reference of step 1

Outputs expected from participants, as well as the corresponding format in the form of a benchmark output format (BOF), were specified by the OC. Regarding step 1, the expected output consisted of (1) a description and justification of the models (choice of finite elements, constitutive relationships, boundary conditions, assumptions regarding the shaking table and its modelling) and (2) a series of outputs that were divided into sections as follows (see details in Annex V):

TABLE 11. MATRIX OF REQUESTED ANALYSES AND MAIN CORRESPONDING REQUESTED OUTPUTS

Identification	Spectral shape	PGA (g)	Top displ. (mm)	Top acc.	Level 1 bending moment	Level 1 shear force	Level 1 to 4 strain	Top response spectrum
Step-1								
Step-1-A: Static analysis								
P-01			1		X	X	X	
P-10			10		X	X	X	
P-15			15		X	X	X	
P-20			20		X	X	X	
Step-1-B: Conventional response spectrum method								
S1	Sh1	0.24	X	X	X	X	X	
S2	Sh2	0.13	X	X	X	X	X	
S3	Sh3	1.11	X	X	X	X	X	
S4	Sh4	0.41	X	X	X	X	X	
Step-1-C: Displacement based approaches								
DBA1	Sh1	0.24	X	X	X	X	X	
DBA2	Sh2	0.13	X	X	X	X	X	
DBA3	Sh3	1.11	X	X	X	X	X	
DBA4	Sh4	0.41	X	X	X	X	X	
DBA-01	Sh3 / Sh4	X / X	1					
DBA-10	Sh3 / Sh4	X / X	10					
DBA-15	Sh3 / Sh4	X / X	15					
DBA-20	Sh3 / Sh4	X / X	20					
Step-1-D: Time history analysis								
TH1	Sh1	0.24	X	X	X	X	X	X
TH2	Sh2	0.13	X	X	X	X	X	X
TH3	Sh3	1.11	X	X	X	X	X	X
TH4	Sh4	0.41	X	X	X	X	X	X
Step-2								
J1	Ito-Oki	0.19	X	X	X	X	X	X
J2	Kashyo dam	0.53	X	X	X	X	X	X
Step-3								
Far-field (FF) series								
FF1	Sh1 (Nice)	0.1	X	X	X	X		X
FF2	Sh1	0.2	X	X	X	X		X
FF3	Sh1	0.3	X	X	X	X		X
FF4	Sh1	0.4	X	X	X	X		X
FF5	Sh1	0.5	X	X	X	X		X
FF6	Sh1	0.6	X	X	X	X		X
Near-field (NF) series								
NF1	Sh2 (SF)	0.1	X	X	X	X		X
NF2	Sh2	0.2	X	X	X	X		X
NF3	Sh2	0.3	X	X	X	X		X
NF4	Sh2	0.4	X	X	X	X		X
NF5	Sh2	0.5	X	X	X	X		X
NF6	Sh2	0.6	X	X	X	X		X

ShY: Spectral shape of the input motion of the corresponding CAMUS run Y.
X: requested output.
shaded: not relevant or not requested output.

(a) Static analysis

The purpose of this section was to conduct a pushover analysis under an inverse triangular load pattern over the height of the specimen that was specified by the OC (details in Annex V) and to provide principally force–displacement curves.

(b) Modal and spectral analysis

The purpose of this section was to conduct a conventional RSA according to the engineering practice in the country of each participant. The input spectra considered are shown in Fig. 19, without broadening or smoothing.

(c) DBAs

As exposed in the rationale and objective of the IAEA CRP, the development of DBAs has been a major improvement in the earthquake engineering of conventional buildings in the past decade. The purpose of this section was to make all the participants familiar with these methods, so as to later discuss their applicability in the context of the nuclear industry.

(d) Time history analyses

Finally, the participants were invited to compute the time history response of the specimen under run 1 to 4 input motions and to plot the more significant results.

3.1.2. Presentation of the models and outputs provided by the participants

3.1.2.1. Finite element models

Analyses were performed with various computer codes based on the finite element method (FEM). The geometry has been described with a significant variety of elements. Depending on the type of analysis performed (static, response spectrum, time history), participants individually decided whether or not to take into account the flexibility of the shaking table. The material characteristics, in particular as regards nonlinear behaviour, had relevant differences.

FEM modelling was performed with 3-D solid, 2-D plane stress, plate, shell, beam, fibre and other elements. Various computer codes were used (ANSYS, NASTRAN, ABAQUS, CASTEM2000, STARDYNE, IDARC, SAP2000, etc.).

The modelling choice and computer codes used by the participants are summarized in Table 12. This table shows static (pushover) analyses; a few participants made other choices for response spectrum and/or time history analyses. The complete details relating to modelling (including material behaviour and boundary conditions) are reported in Annex VI. As an example of modelling variability, values of Young’s modulus selected by participants vary from 11 500 to 30 700 MPa.[12]

[12] Some participants decided to account for the consequences of expected cracking by introducing a reduced E modulus of concrete.

TABLE 12. OUTLINES OF NONLINEAR STATIC (PUSHOVER) ANALYSES

Participant	Model	Type of finite element	Longitudinal reinforcement consideration	Lateral reinforcement consideration	Shaking table consideration	Computer code
ANRA-Armenia	Stick	Beam			Yes	SAP2000
BAS-Bulgaria	Stick	Beam		No	No	IDARC
AECL-Canada	Fibre	6 node panel	Uniaxial fibre	No	No	CANDY
BINE-China	2-D FEM	Reinforced solid	Embedded reinforcement	No	No	ANSYS
FORTUM-Finland	2-D FEM	4 node plane stress	2 node truss	No	No	MSC, NASTRAN, ABAQUS
INSA-France	2-D FEM	4 node membrane	2 node truss	No	Yes	CASTEM
IRSN-France	2-D FEM	8 node plane stress	3 node beam	No	Yes	DIANA
AERB-India	3-D FEM	Reinforced solid	Embedded reinforcement	Yes	Yes	ANSYS
POLIMI-Italy	Fibre	3 node RCIZ fibre (fibre beam)	Fibre beam	Yes	Yes	NONDA
JNES-Japan	2-D FEM	8 node plate	Embedded reinforcement	No	No	COM3
KINS-Republic of Korea	2-D FEM	4 node shell	4 node shell	No	Yes	RCAHEST
KOPEC-Republic of Korea	Stick	Beam	–	Yes	No	IDARC
PAEC-Pakistan	3-D FEM	4 node shell	–	No	No	ANSYS
UTCB-Romania	Stick	Beam	–	No	No	IDARC
CKTI-Vibroseism-Russian Federation	2-D FEM	4 node plane stress	2 node beam	No	Yes	SOLVIA
SAS-Slovakia	Stick	Beam	–	No	No	SAP2000
IDOM-Spain	Stick	Beam	Embedded reinforcement	No	Yes	ABAQUS
METU-Turkey	3-D FEM	Reinforced solid	Embedded reinforcement	No	Yes	ANSYS
TAEA-Turkey	3-D FEM	Reinforced solid	Embedded reinforcement	No	Yes	ANSYS
HSE-UK	2-D FEM	8–6 node plane stress	Embedded reinforcement	Yes	Yes	DIANA
BNL-USA	3-D FEM	Reinforced solid	Embedded reinforcement	No	No	LS-DYNA

3.1.2.2. Static analysis

Pushover analysis is based on nonlinear static assessment of the behaviour of the structure under an increasing pattern of lateral loads simulating the inertia forces due to an earthquake. This procedure accounts for the nonlinear load deformation characteristics of individual components and elements of the structure, and provides an integral load–displacement curve representative of the structure capacity.

Pushover curves (top displacement versus applied force) of all the participants are plotted in Fig. 23. It can be observed that there are four curves corresponding to a significant higher capacity and one corresponding to a significant lower capacity as compared to the vast majority of curves. This majority group (16) exhibits good agreement with experimental results derived from CAMUS runs, which are also plotted in the figure. The question of whether this pushover curve is applicable to interpretation of dynamic runs is discussed in Section 3.4. The mean pushover curve resulting from participants' outputs is shown in Fig. 24 as well as an equivalent bilinear curve.

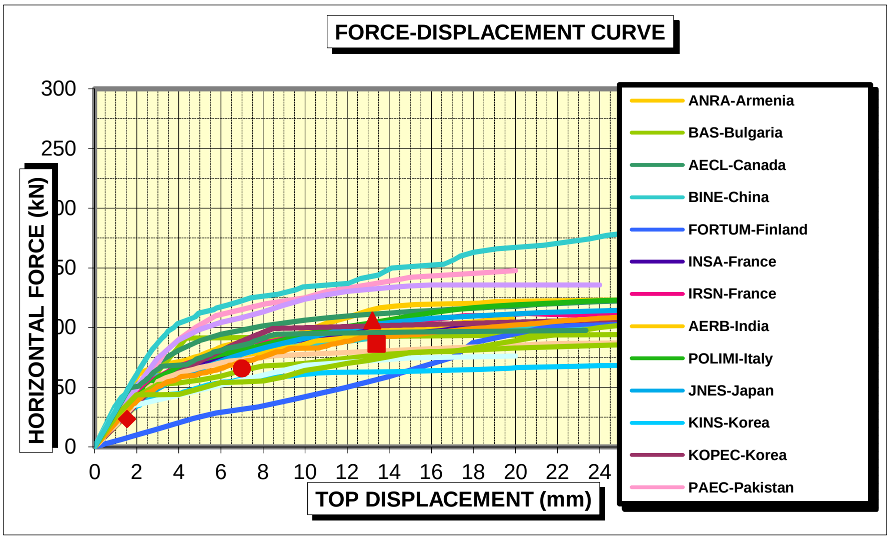


FIG. 23. Pushover curves provided by participants.

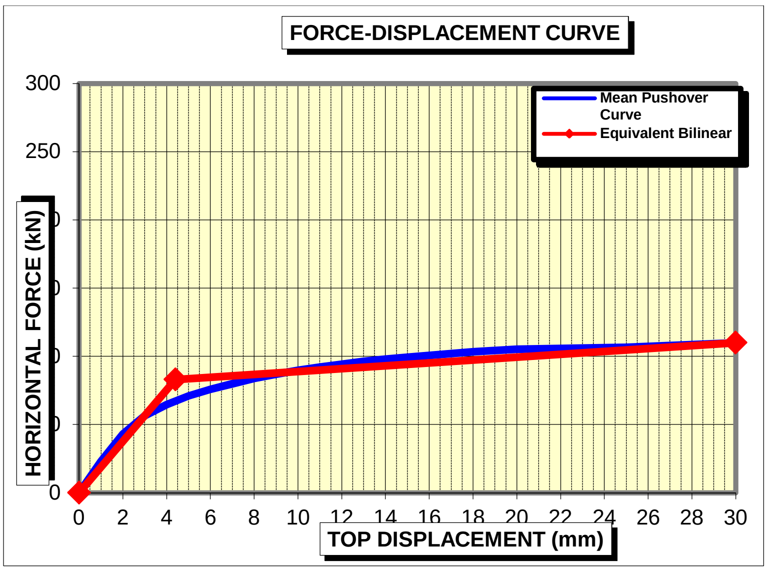


FIG. 24. Mean pushover curve.

Level 1 shear force and bending moment are shown in Table 13 for 1 mm and 20 mm top displacements. For instance, for a 1 mm top displacement, the minimum and maximum values of shear force are 5.1 and 35.3 kN, respectively, and the corresponding mean value and standard deviation are 23.3 and 4.6 kN, respectively. Interesting to note is that there is no evidence that the COV is larger in the nonlinear regime (P-20) compared to the linear one (P-01).

3.1.2.3. Modal and spectral analysis

The most significant outputs are shown in Tables 14 and 15, and a full description is given in Annex VII.

Regarding the input data selected by participants, the following are presented: (1) the concrete Young modulus; (2) whether the flexibility of the shaking table was taken into account or not; and (3) the damping ratio (a few participants used a damping value other than 5%). The first two points directly affect the (first) natural frequency, while the third also affects the magnitude of the dynamic response (see the first columns of Table 14).

TABLE 13. MAJOR OUTPUTS PROVIDED BY THE PARTICIPANTS FOR PUSHOVER ANALYSES

Participant		1 mm top diplacement *(P-01)*		20 mm top diplacement *(P-20)*	
		Level1 Shear Force (kN)	Level1 Bending Moment (kN m)	Level1 Shear Force (kN)	Level1 Bending Moment (kN m)
1	ANRA-Armenia	31.4	118.3	121.8	464.9
2	BAS-Bulgaria	20.9	66.4	90.1	292.0
3	AECL-Canada	24.7	81.4	117.7	390.0
4	BINE-China	35.3	114.0	100.3	324.0
5	FORTUM-Finland	5.1	17.7	80.2	268.5
6	INSA-France	22.1	71.0	102.0	326.6
7	IRSN-France	21.1	64.1	111.8	340.5
8	AERB-India	17.5	57.8	106.0	349.6
9	POLIMI-Italy	21.8	69.6	119.0	380.0
10	JNES-Japan	18.8	55.3	113.0	337.5
11	KINS-Korea	18.3	60.4	66.5	219.5
12	KOPEC-Korea	18.0	59.4	104.6	345.2
13	PAEC-Pakistan	25.9	85.5	147.8	488.8
14	UTCB-Romania	18.3	60.2	84.9	279.2
15	CKTI-Vibroseism-Russia	21.5	70.5	83.0	274.0
16	SAS-Slovakia	31.1	102.5	76.1	251.2
17	IDOM-Spain	24.4	80.6	135.6	447.5
18	METU-Turkey	21.0	64.5	102.1	328.8
19	TAEA-Turkey	25.5	81.6	88.9	287.7
20	HSE-UK	30.0	99.0	97.0	320.0
21	BNL-USA	30.1	135.5	167.0	751.5
Mean		23.3	77.0	104.3	341.9
Standard deviation		4.6	19.2	18.9	66.9
Coefficient of variation		0.20	0.25	0.18	0.20
Note: Max. and Min. values are not included in the mean and standard deviation computations. Max and Min values are hihglighted.					

TABLE 14. MAJOR OUTPUTS PROVIDED BY THE PARTICIPANTS FOR THE CONVENTIONAL SPECTRAL METHOD — MAXIMUM AND MINIMUM VALUES ARE HIGHLIGHTED

Participant		Young modules of	Damping ratio	Shaking table flexibility	First frequency	Top displacement (mm)			
						S1	S2	S3	S4
1	ANRA-Armenia	22400	5	Yes	7.51	5.39	1.85	13.70	2.85
2	BAS-Bulgaria	28000	5	No	6.46	5.68	1.55	13.23	9.70
3	AECL-Canada	17500	5	No	7.92	4.80	1.84	13.90	2.80
4	BINE-China	28000	5	Yes	8.18	4.87	1.79	13.50	2.69
5	FORTUM-Finland	30000	2	No	8.50	7.42	1.27	14.10	12.20
6	INSA-France	30000	4	Yes	7.24	6.06	1.72	13.50	2.87
7	IRSN-France	28000	5	Yes	7.22	5.20	1.57	11.90	2.48
8	AERB-India	30650	5	Yes	7.24	6.03	1.48	11.60	3.06
9	POLIMI-Italy	30650	5	No	8.77	3.26	1.40	11.73	2.46
10	JNES-Japan	26600		Yes	7.30				
11	KINS-Korea	28000	5	Yes	8.23	4.62	1.81	13.37	2.68
12	KOPEC-Korea	30642	5	Yes	7.64	6.26	1.98	15.29	2.94
13	PAEC-Pakistan	28000	5	Yes	7.45	5.10	1.79	13.80	3.16
14	UTCB-Romania	11500	5	Yes	5.65	7.23	3.29	28.10	12.30
15	CKTI-Vibroseism-Russia	30600	5	Yes	7.20	5.87	1.73	12.99	2.69
16	SAS-Slovakia	14000	5	No	5.83	6.67	1.78	10.74	5.19
17	IDOM-Spain	28000	5	Yes	8.24	5.09	1.98	14.17	2.88
18	METU-Turkey	30000	5	Yes	7.28	6.13	1.47	12.07	3.20
19	TAEA-Turkey	30000	5	Yes	8.19	5.40	1.80	13.83	2.90
20	HSE-UK	30700	5	Yes	8.34	4.29	1.63	12.37	2.48
21	BNL-USA	28000	2	No	8.92	3.42	1.27	11.70	2.45
Mean					7.62	5.45	1.69	13.15	3.85
Standard deviation					0.73	0.91	0.20	1.04	2.69
Coefficient of variation					0.10	0.17	0.12	0.08	0.70
Test					7.24	7.01	1.54	13.19	13.43

Maximum ▒ Minimum ▒

* METU's top displacement result for RUN4 taking into account cumulative damage is 12.20 mm

Note: Max. and Min. values are not included in the mean and standard deviation computations.

TABLE 15. CONVENTIONAL SPECTRAL ANALYSIS, SHEAR FORCES AND BENDING MOMENTS AT LEVEL 1

	Shear Force (kN)				Bending Moment (kN m)			
	S1	S2	S3	S4	S1	S2	S3	S4
Mean	108.5	36.5	287.7	76.5	353.9	119.1	927.7	254.7
Standard deviation	16.0	6.8	47.9	29.3	66.4	33.3	251.1	107.2
Coefficient of variation	0.15	0.19	0.17	0.38	0.19	0.28	0.27	0.42
Test	65.9	23.5	105.5	86.6	211.1	75.5	279.7	279.3

Considering the first natural frequency, the minimum and maximum values obtained by the participants are 5.65 and 8.92 Hz, respectively, with a 7.62 Hz mean value and a 0.73 Hz standard deviation. An unexpectedly large number of participants provided a first natural frequency very close to the experimental value of 7.24 Hz. This clearly reveals that several participants tuned their model in order to get this result. This approach is not consistent with the spirit of this part of the benchmark, the objective of which was to assess and compare national engineering practices[13]. In spite of this bias, it is notable that the scattering of RSA outputs is rather large, likely reflecting the modelling diversity.

On average, top displacement outputs are comparable with experimental ones for runs 1 to 3. It is noteworthy that this output results from linear analyses although run 3 (and also to a certain extent run 1) brought the specimen into a nonlinear regime. On the contrary, for those three runs, and especially for runs 1 and 3, the level 1 shear force and bending moment are strongly overestimated (by a factor of about 1.5 to 3).

[13] For this purpose, participants were requested to apply standard engineering procedures and to refrain from tuning the model.

As opposed to other runs, the predicted top displacements for run 4 are significantly under estimated, while the forces and moments are correctly predicted. This apparent divergence with the performances observed on the other runs is due to the fact that most of the participants did not take into account the run 3 pre-damaging effect (see Section 3.1.3.6). Taking this into account in a linear analysis would imply evaluating an equivalent degradation of the specimen stiffness, which is rather difficult. Participant METU carried out an additional run 4 accounting for the cumulative damage by modifying the stiffness of the model and obtained a larger top displacement as would be expected.

3.1.2.4. DBAs

Most of the participants used the displacement coefficient method (FEMA 273) and/or the capacity spectrum method (ATC-40) as a DBA. Three participants used other methods, namely the constant ductility spectra method, EC8 and the brute force method.

In the displacement coefficient method (FEMA 273), an effective elastic stiffness and associated fundamental period is derived from the pushover curve. A target displacement is then calculated using response spectrum, acceleration, effective fundamental period and modification factors (C_0, C_1, C_2 and C_3).

In the capacity spectrum method (ATC-40), on the basis of the pushover analysis, the curve of spectral accelerations versus spectral displacements is calculated for each loading step (capacity spectrum). Equivalent periods and equivalent damping ratios for a given earthquake response spectrum are computed and used for computing the acceleration–displacement curve (demand spectrum). The intersection of the capacity spectrum with the demand spectrum allows the prediction of the top displacement and shear force induced by the earthquake on the structure.

Detailed outputs are presented in Annex VII. Regarding the top displacements provided by the participants (Table 16), there is a similarity with response spectrum outputs: the mean value is in agreement with the test output for runs 1 to 3, while the run 4 top displacement is underestimated for reasons already mentioned in Section 3.1.2.3. Regarding forces and moments, and as opposed to the response spectrum approach, on average they are correctly predicted (Table 17). However, the standard deviation is quite large, which likely reflects the inevitable lack of maturity of the engineering community when dealing with a new approach.

Associated to the spectral shapes of runs 3 and 4 (respectively referenced as San Francisco and Nice in the tables), estimated PGA levels leading to 1, 10, 15 and 20 mm top displacement are shown in Table 18. For a given top displacement, PGA levels associated with the San Francisco spectral shape are higher than those associated with the Nice spectral shape. These outputs are of great interest for the interpretation carried out in Section 3.1.4.

3.1.2.5. Time history analysis

Outputs of time history analyses are shown in Tables 19 and 20. All participants used models that account for a possible material nonlinear behaviour. Depending on the circumstances, this nonlinear effect was either active or not.

Regarding run 1 (Nice 0.24 g), run 2 (San Francisco 0.13 g) and run 3 (San Francisco 1.11 g), computed top displacements are satisfactory and COVs are rather small. Forces are also correctly estimated although COV is relatively larger for run 2.

TABLE 16. DBAS, TOP DISPLACEMENTS (MM) PROVIDED BY THE PARTICIPANTS

Participant		Top displacement (mm)				Displacement Based Method	Notes
		DBA1	DBA2	DBA3	DBA4		
1	ANRA-Armenia	6.53	2.20	15.82	3.44	FEMA	(2)
2	BAS-Bulgaria	6.90	1.70	17.40	12.50	FEMA	(1)
3	AECL-Canada	5.60	1.68	8.40	4.20	ATC-40	(1)
4	BINE-China	11.27	2.23	16.62	7.35	FEMA	(1)
5	FORTUM-Finland	11.90	1.25	10.70	17.95	Brute Force	(1)
6	INSA-France	3.70	1.85	8.20	2.70	ATC-40	(2)
7	IRSN-France	6.60	2.30	9.90	7.90	FEMA	(2)
8	AERB-India	7.94	1.73	13.44	3.61	FEMA	(2)
8	AERB-India	2.80	1.30	6.00	2.90	ATC-40	(2)
9	POLIMI-Italy	6.09	1.84	14.22	7.63	FEMA	(2)
10	JNES-Japan						
11	KINS-Korea	6.76	1.85	14.50	4.05	FEMA	(2)
11	KINS-Korea	2.91	1.69	6.96	4.92	ATC-40	(2)
12	KOPEC-Korea	7.55	1.76	16.50	3.62	FEMA	(1)
13	PAEC-Pakistan	5.56	1.39	12.50	3.82	FEMA	(1)
14	UTCB-Romania	6.60		14.60	4.50	FEMA	(1)
14	UTCB-Romania	6.90		9.40	4.90	ATC-40	(1)
15	CKTI-Vibroseism-Russia	7.22	1.99	12.49	6.56	FEMA	(2)
16	SAS-Slovakia	4.31	1.77	12.78	11.21	EUROCODE	(1)
17	IDOM-Spain	4.53	1.50	15.48	3.23	FEMA	(2)
18	METU-Turkey	7.52	1.60	8.43	12.07 ** +	FEMA	(2)
18	METU-Turkey	6.16	1.50	9.52	4.06	CDSM	(2)
19	TAEA-Turkey	7.14	1.84	13.00	9.10	FEMA	(2)
20	HSE-UK	6.20	1.90	14.40	2.80	FEMA	(2)
21	BNL-USA	5.75	4.13	16.47	4.94	FEMA	(1)
Mean		6.35	1.78	12.47	5.88		
Standard deviation		1.69	0.26	3.05	3.01		
Coefficient of variation		0.27	0.15	0.24	0.51		
Test (top displ.)		7.01	1.54	13.19	13.43		

(**) Cumulative damage (modified pushover curve) considered. (+) Smooth input spectrum used.

Notes: (1) Fixed base model is used. (2) Shaking table flexibility is considered.

Max. and Min. values are not included in the mean and standard deviation computations.

TABLE 17. DBAS, SHEAR FORCES AND BENDING MOMENTS AT LEVEL 1

	Shear Force (kN)				Bending Moment (kN m)			
	DBA1	DBA2	DBA3	DBA4	DBA1	DBA2	DBA3	DBA4
Mean	73.6	36.7	90.4	69.4	244.0	117.1	294.8	225.9
Standard deviation	14.2	8.2	18.6	12.7	48.8	22.7	62.6	39.0
Coefficient of variation	0.19	0.22	0.21	0.18	0.20	0.19	0.21	0.17
Test	65.9	23.5	105.5	86.6	211.1	75.5	279.7	279.3

TABLE 18. DBAS, ACCELERATION LEVEL (g) CORRESPONDING TO A GIVEN DISPLACEMENT

	RUN3 spectral shape (SF)				RUN4 spectral shape (SF)			
	DBA-01	DBA-10	DBA-15	DBA-20	DBA-01	DBA-10	DBA-15	DBA-20
Top disp. (mm)	1	10	15	20	1	10	15	20
Mean	0.09	0.81	1.20	1.56	0.09	0.61	0.85	1.09
Standard deviation	0.01	0.12	0.15	0.51	0.04	0.37	0.57	0.79
C.O.V.	0.1	0.2	0.1	0.3	0.5	0.6	0.7	0.7

TABLE 19. TIME HISTORY ANALYSIS, TOP DISPLACEMENTS (MM) PROVIDED BY THE PARTICIPANTS

PARTICIPANT		Top displacement (mm)				
		TH1	*TH2*	*TH3*	*TH4/a*	*TH4/b*
1	ANRA-Armenia	5.61	1.13	9.92		2.97
2	BAS-Bulgaria	6.32	1.87	14.87	9.66	
3	AECL-Canada	6.04	1.85	10.56	12.58	
4	BINE-China	8.92	0.96	10.62		5.26
5	FORTUM-Finland	12.00	1.26	10.72	18.07	
6	INSA-France	5.52	1.90	12.50	13.04	
7	IRSN-France	4.19	1.54	7.92	4.41	
8	AERB-India	5.81	1.92	9.79	8.52	
9	POLIMI-Italy	6.78	1.61	11.00	9.24	
10	JNES-Japan	5.21	1.27	8.73	5.69	
11	KINS-Korea	6.62	2.17	11.73		4.11
12	KOPEC-Korea	5.68	2.06	10.45		3.52
13	PAEC-Pakistan	4.16	1.38	8.31	9.13	
14	UTCB-Romania	4.76	1.80	8.38	3.01	
15	CKTI-Vibroseism-Russia	6.24	2.00	10.96		4.83
16	SAS-Slovakia	6.24	1.50	8.25	12.70	
17	IDOM-Spain	5.71	2.03	10.77	4.91	
18	METU-Turkey	6.58	1.49	13.02	13.16	
19	TAEA-Turkey	6.51	1.84	11.00	9.48	
20	HSE-UK	4.21	1.86	9.68		2.78
21	BNL-USA	6.68	2.45	26.20	15.70	
Mean		5.98	1.71	10.59	9.86	3.86
Standard deviation		1.06	0.31	1.68	3.48	0.80
Coefficient of variation		0.18	0.18	0.16	0.35	0.21
Test		7.01	1.54	13.19	13.43	

Maximum ░ Minimum ▒

Notes for TH4: /a: Pre-damaging effect of previous runs is taken into account

/b: Pre-damaging effect of previous runs is not considered

Note: Max. and Min. values are not included in the mean and standard deviation computations.

TABLE 20. DBAS, SHEAR FORCES AND BENDING MOMENTS AT LEVEL 1

	Shear Force (kN)					Bending Moment (kN m)				
	TH1	*TH2*	*TH3*	*TH4/a*	*TH4/b*	*TH1*	*TH2*	*TH3*	*TH4/a*	*TH4/b*
Mean	79.1	30.5	112.5	83.1	74.2	245.3	103.4	290.9	242.8	208.7
Standard deviation	13.2	9.8	21.7	8.4	6.1	35.8	31.6	60.2	36.8	16.5
Coefficient of variatior	0.17	0.32	0.19	0.10	0.08	0.15	0.31	0.21	0.15	0.08
Test	65.9	23.5	105.5	86.6	86.6	211.1	75.5	279.7	279.3	279.3

Notes for TH4: /a Pre-damaging effect of previous runs is taken into account

/b Pre-damaging effect of previous runs is not considered

Regarding run 4 (Nice 0.41 g), it is clear that outputs have to be classified according to two groups of teams: (1) those that do take into account the pre-damaging effect of run 3 and (2) those that do not. Even though the first of these two approaches leads to better estimates of top displacement, it is still significantly underestimated. The level 1 bending moment is also underestimated by 15% while the shear force estimate is acceptable.

Due to the major role played by nonlinearity at least in runs 3 and 4, it is of interest to check the consistency between the predicted top displacement and the base shear (or bending moment) and the relationship between these two values as established by the pushover curve. This exercise is presented in Section 3.4, including the outputs of steps 2 and 3. Floor response spectra provided by participants (at the top of the model for a 5% damping) are shown in Figs 23 to 26, respectively for runs 1 to 4. They are compared to experimental ones.

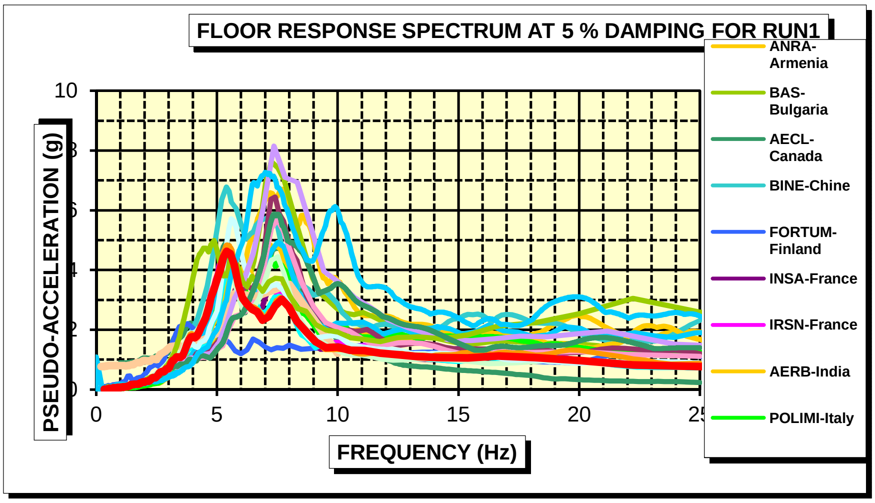


FIG. 25. Time history analyses: Top floor response spectra for TH1 (Nice 0.24 g).

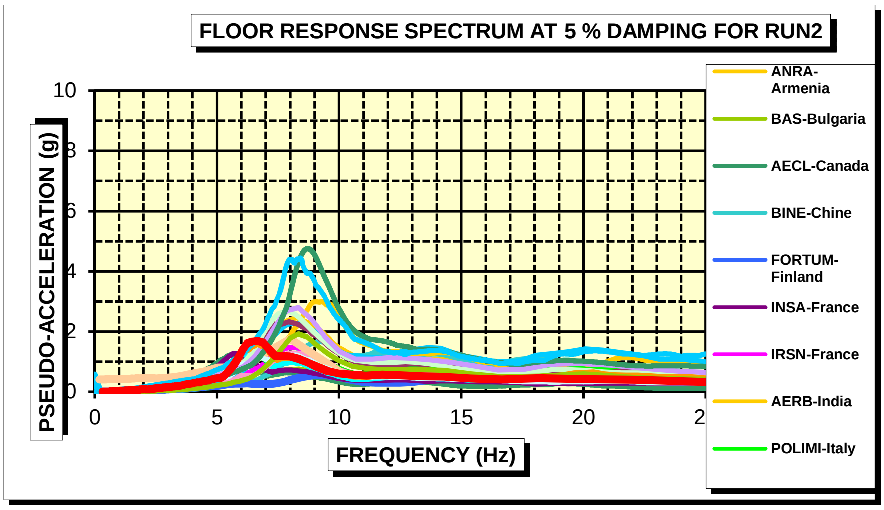


FIG. 26. Time history analyses: Top floor response spectra for TH2 (San Francisco 0.13 g).

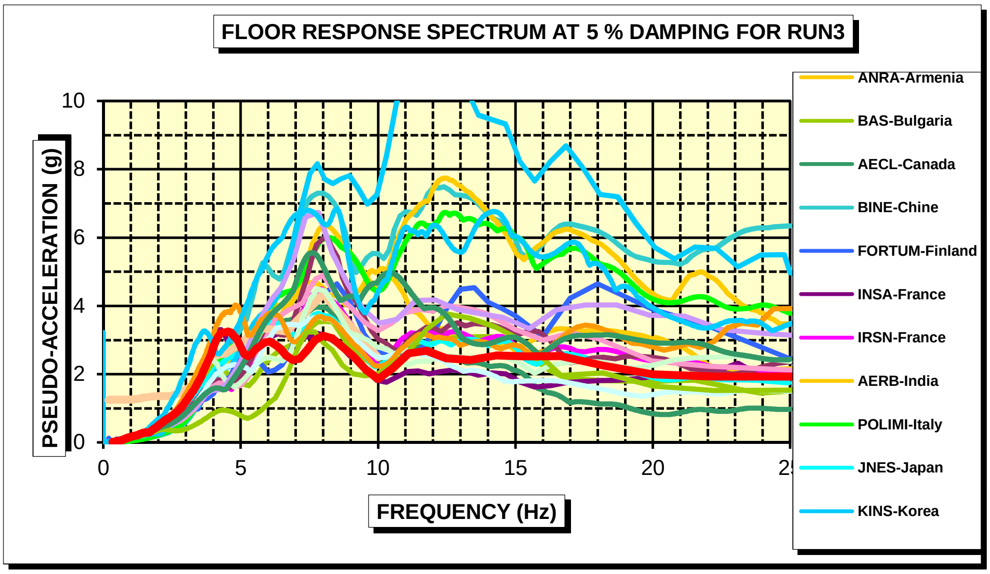


FIG. 27. Time history analyses: Top floor response spectra for TH3 (San Francisco 1.11 g).

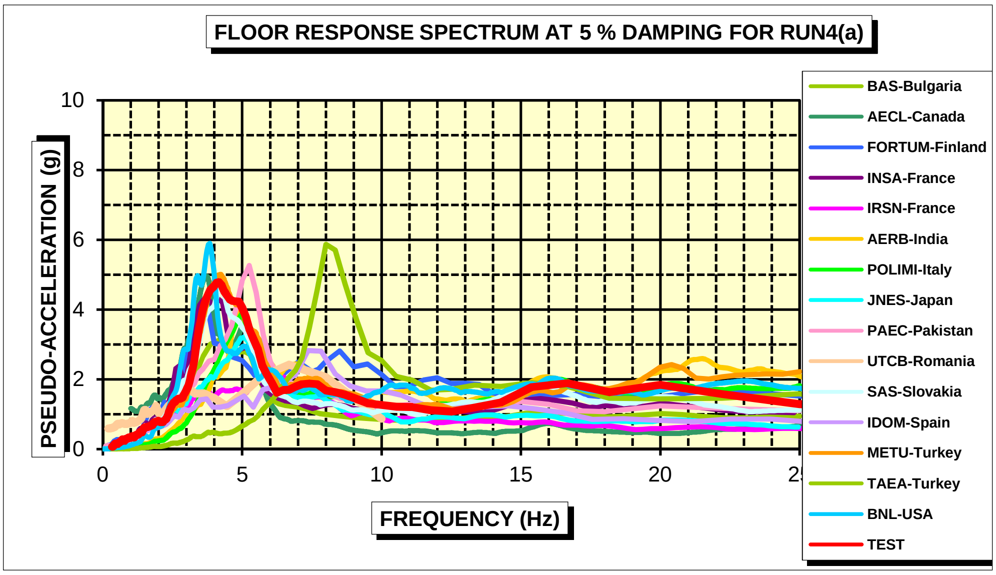


FIG. 28(a). Time history analyses: Top floor response spectra for TH4/a (Nice 0.41 g).

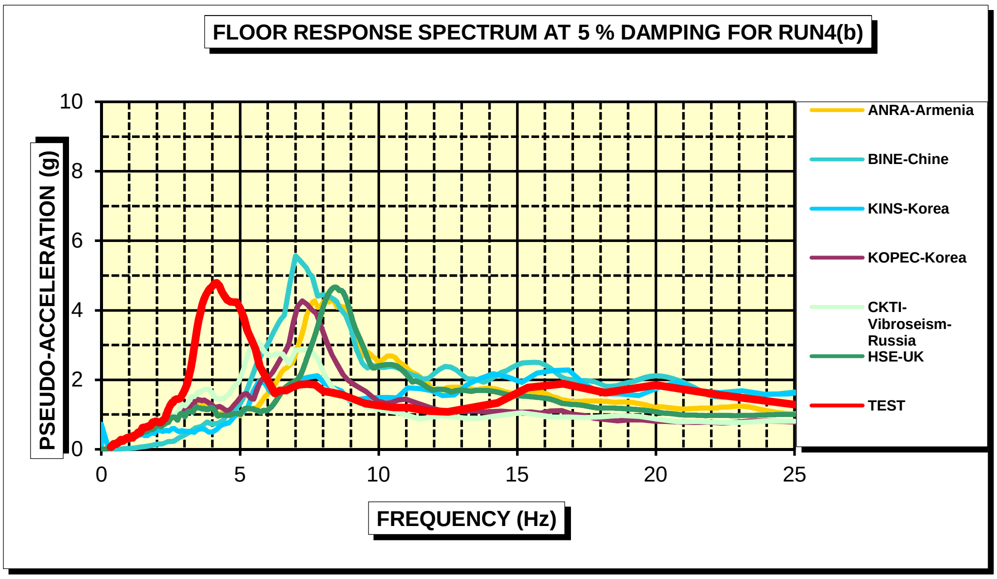


FIG. 28(b). Time history analyses: Top floor response spectra for TH4/b (Nice 0.41 g).

First of all, it should be put forward that even for the low level inputs (runs 1 and 2), the peak of the observed response spectrum does not fit the (first) natural frequency of the specimen; it is shifted towards low frequencies. This means that although damage developed during these runs is negligible[14] and the associated small nonlinearity has a negligible impact on displacements and forces, the impact on floor response spectra is significant. Most of the participants did not catch this peak shift or underestimated it to a large degree. For moderate nonlinearity (still not exceeding the conventional limit state), the phenomenon is obviously amplified as revealed by the outputs of runs 3 and 4.

This result seriously challenges nuclear industry practices. It motivated the benchmark stretch in the form of step 3, dedicated to the floor response spectra generation issue. This question is addressed in the corresponding Section 3.3.

[14] This is substantiated by the fact that the first eigenfrequency of the specimen did not evolve before run 3 was launched, as indicated in Table 7 of Annex III.

Other comments are common, with comments relating to outputs resulting from the response to Japanese input motions, and are presented in Section 3.2.2.

3.1.3. Comments on specific items

3.1.3.1. Modelling

The background of the participants is wide, including engineering companies, regulators and their technical support organizations, research institutes, universities, etc. Differences between academic practices and engineering practices appear clearly in the modelling choices made by the participants. Most of them used sophisticated modelling compared to common practice in earthquake engineering analysis and the design of nuclear facilities.

3.1.3.2. Initial stiffness of the specimen

The first requested output of the static analysis was the loading force leading to a 1 mm top displacement of the specimen. The intention was to capture the initial slope as representative of the elastic behaviour of the corresponding model. The discrepancy on this initial slope is unexpectedly high (it varies by a factor of two). This phenomenon may result from a number of causes.

One cause is the flexibility of the shaking table, which is discussed in Section 3.1.3.3.

Another cause is the diversity in professional backgrounds of the participants. The benchmark illustrates that, as compared to conventional building practice, the nuclear power plant engineering approach leads to stiffer models. For instance, in the spirit of conventional building practice, and as opposed to nuclear power plant engineering practice, some participants ignored the non-cracked stiffness of the specimen and deliberately adopted a reduced Young modulus.

It seems, however, that the main source of discrepancy comes from the large variability in the models adopted by the participants for describing the geometry of the specimen (3-D, 2-D, stick models, multi-fibre elements, etc.).

The initial stiffness of the different models can also be compared by considering the computed first natural frequency. The discrepancy here is also high, and surprisingly there is not excellent consistency between these two indicators: a participant may have one of the more flexible models according to the initial slope of the pushover curve and one of the stiffer according to the first natural frequency.

Finally, it was not possible to identify a root cause of this discrepancy. For instance, models based on the assumption of a fixed base are not significantly stiffer than models which account for the flexibility of the shaking table. It has to be concluded that the other modelling options such as the type of finite elements play a significant role in the initial stiffness of the models.

3.1.3.3. Flexibility of the shaking table and SSI effect

In the static analysis, the participants hesitated to use modelling that took into account shaking table flexibility. Clearly, the first thought that comes to mind is that the pushover curve should provide information about the behaviour of the structure itself, disregarding the possible effects of support flexibility. Similarly, a given real structure should have a pushover curve that does not depend on the flexibility of the foundation. On this basis, several participants presented pushover curves established with the assumption of a fixed base.

In the RSA, it was clear to most of the participants that the flexibility of the shaking table should be taken into account so as to get as correct as possible values of natural frequencies.

For the DBAs, two options could be discussed:

(a) On the one hand, the DBAs were initially developed for and by the conventional building industry. In this context, SSI effects are negligible or neglected. Consistently, the original intention of the designers of the method was to use fixed base pushover curves.

(b) On the other hand, it is clear when using ATC-40 type methods, for instance, that an equivalent natural frequency is directly linked to the performance point. If the structure stays in the elastic range, this equivalent frequency should be the (first) natural frequency. Consequently, in the case of CAMUS, the initial slope of the pushover curve should include the flexibility of the shaking table so as to lead to a correct estimate of the natural frequency.

As opposed to conventional buildings, it is a feature of the nuclear industry that SSI effects are taken into account. This specific item has already been identified as an issue to be resolved in view of a possible application of DBAs in the field of the engineering of nuclear power plants. The CAMUS test illustrates the case, and the research carried out in the frame of this IAEA CRP provides useful elements on the way it should be addressed.

3.1.3.4. Bifurcating collapse mode of the CAMUS specimen

Although predicting the collapse mode of the CAMUS specimen was beyond the scope of the benchmark, several participants investigated it and pointed out the practical impossibility of a reliable prediction. This is due to the fact that the design was not made according to capacity design principles. In the case of CAMUS, the detailed R-bar design is so that a homogeneous margin is achieved in every section of the wall. It is, therefore, not possible to decide which section should be regarded as the weak one. To a certain extent, the collapse occurrence at level 3 during run 5 can be regarded as a sample of a random process. For another specimen, hypothetically made in the same conditions as CAMUS and subjected to the same inputs, the collapse could occur at another level. The different crack patterns between the right wall and the left wall can also be regarded as evidence of this feature of CAMUS.

Also linked to this feature is the extreme sensitivity of moment–curvature relationships. As opposed to the pushover curve, which represents global data, these relationships are local data. The participants observed that the moment–curvature relationship just above a floor is significantly different from that just below the same floor. Therefore, it is very difficult to rely on these relationships in the analysis. For instance, deriving strains may be very sensitive.

3.1.3.5. Sensitivity to the input motion

The extreme sensitivity of the outputs of nonlinear analyses is a well known technical finding. Its root cause is elicited by the procedure of the DBA as developed by the ATC-40 (it would be the same with a similar method). The method is based on the identification of a performance point at the crossing between the pushover and the acceleration–displacement response spectrum (ADRS), as illustrated in Fig. 29.

In this figure, the run 4 ADRS is plotted. It is clear that a minor change in the pushover curve, or the choice of another input motion may result in a dramatically different performance point. This conclusion holds for both ATC-40 type and time history analyses.

From this observation, it can be concluded that nonlinear analyses should be carried out either with an input motion that has a smooth ADRS (or with a few input motions that have smooth ADRSs) or with a large number of input motions followed by an appropriate post-treatment (e.g. averaging of the outputs). The latter option is feasible when dealing with a DBA. It is hardly feasible when dealing with time history analyses as the natural input motion ADRSs are far from smooth. This conclusion questions the fashionable opinion that time history analyses should preferably be carried out with natural input motions.

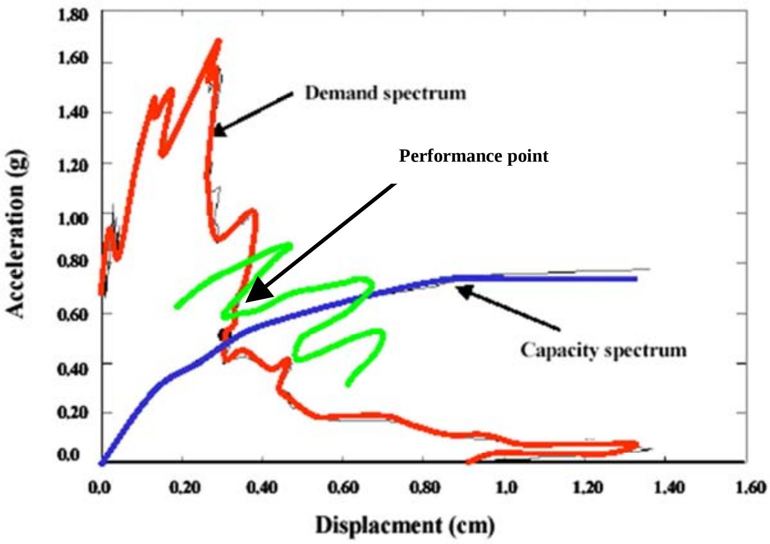


FIG. 29. Sensitivity of the ATC-40 output to the spectral shape of the input motion.

3.1.3.6. Pre-damaging effect of run 3 on run 4

RSA as well as DBA exhibit poor predictive performances when dealing with run 4. There is a consensus among participants that this is due to the run 3 pre-damaging effect that was not taken into account either by RSA or by DBAs.

On the contrary, most of the time history analyses were carried out taking into account this pre-damaging effect and lead to acceptable outputs for run 4. It is worth noting that some participants carried out (also in addition) a time history analysis of run 4 starting on a virgin structure (no pre-damaging effect). These computations led to outputs similar to those of RSA and DBA.

3.1.4. Compared performances of different approaches

The compared performances of the three approaches considered in step 1 of the benchmark are summarized in Figs 30 to 34. In these figures, 'S' stands for the classical response spectrum method, 'F' for the FEMA approach, 'A' for ATC-40 and 'T' for time history analyses (regarding time history outputs of run 4, the set of teams that accounted for the pre-damaging effect of previous runs was considered). Means and standard deviations calculated from the participants' results are plotted in these figures. More precisely, x_0 being the experimentally observed value of the quantity under consideration in the figure, X its predicted mean, and σ_x its predicted standard deviation, the following ratios are plotted in Figs 30 to 34: X/x_0 and $\sigma_{x/}x_0$.

Considering top displacement, it can be observed that all the methods tested in the frame of the benchmark lead to comparable outputs. There is no definite evidence that one of them performs better than the other ones. Regarding run 4, it can be noticed that all of the results underestimate the test output. This is likely a consequence of the run 3 pre-damaging effect. It is noticeable that, although the history of the specimen is rather complicated, time history analyses that account for the pre-damaging effect of the previous runs perform more satisfactorily than any other approach. To a large extent, similar comments are applicable to the mean of tensile strain outputs. This was expected because there is a strong correlating relationship between displacements and strains.

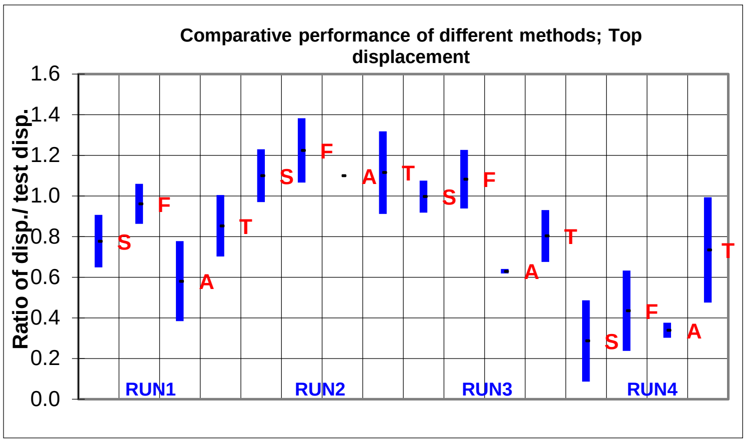


FIG. 30. Comparison of top displacements obtained by different methods.

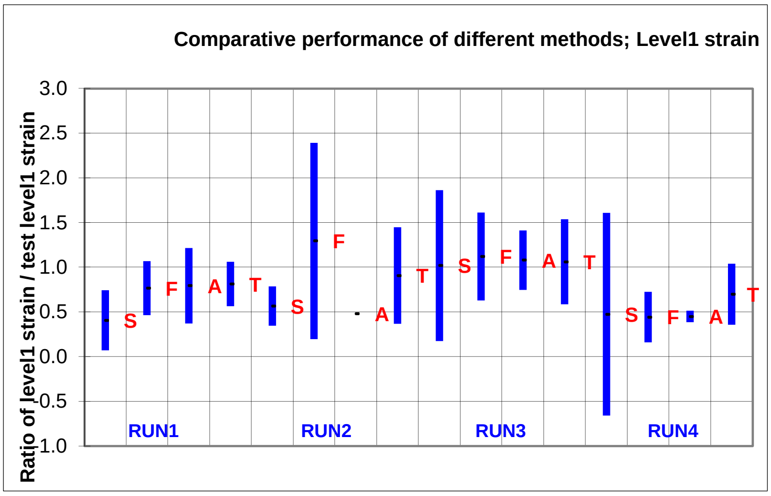


FIG. 31. Comparison of level 1 strains obtained by different methods .

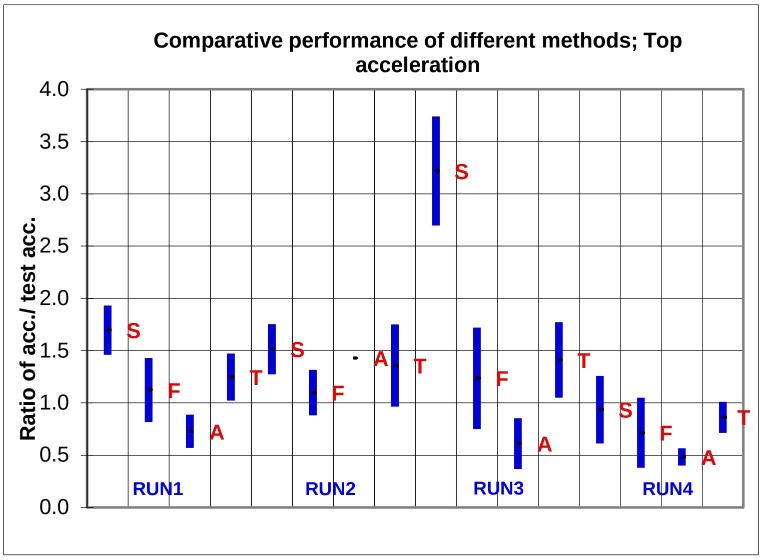


FIG. 32. Comparison of top acceleration obtained by different methods .

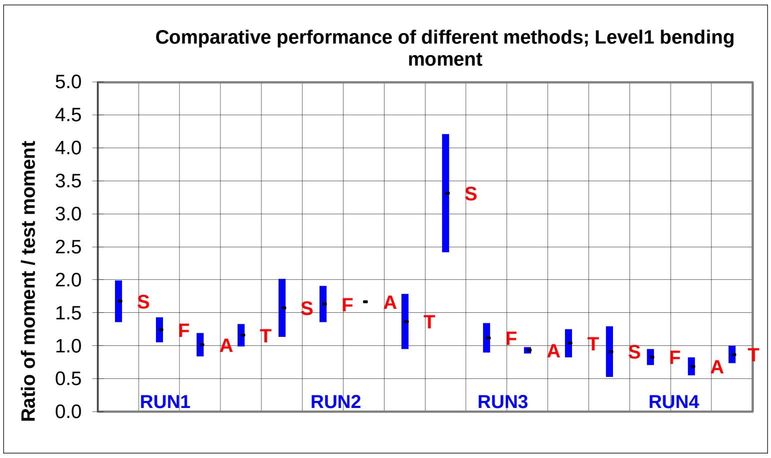


FIG. 33. Comparison of level 1 bending moment.obtained by different methods

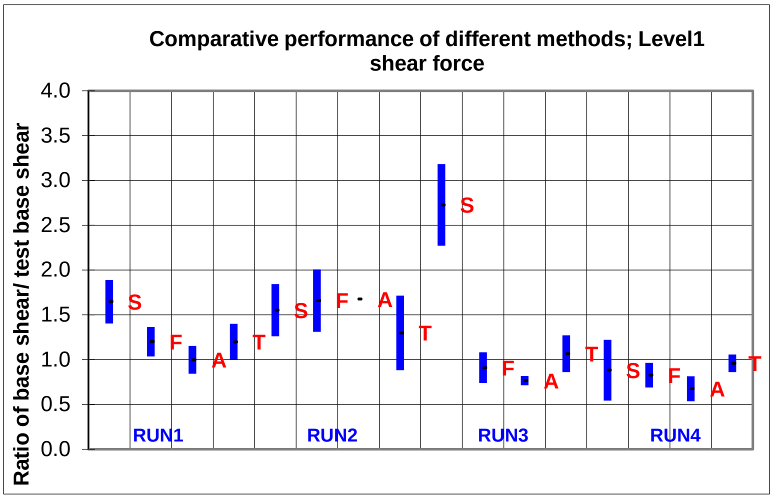


FIG. 34. Comparison of level 1 shear force.obtained by different methods .

Regarding top acceleration, bending moments and shear forces, there is likely still an impact of the run 3 pre-damaging effect; however, it has to be mentioned that if restricted to those that account for this effect, time history analyses provide very good results.

For these three items, the most noticeable output is the following: Generally speaking, all the outputs are in the range between half and two times the test result, with the remarkable exception of the classical response spectrum method when dealing with run 3 (San Francisco 1.11 g). This phenomenon is a clear illustration of the 'near-field earthquake effect'. In particular, it is clear that conventional criteria based on bending moments strongly overestimate the damaging capacity of this type of input motion.

Although run 2 is also of the same type, the phenomenon does not appear because of the very low PGA. In such a case, the response is practically elastic and all methods lead to similar outputs. To a certain extent, this remark also applies to run 1, which is a 'high frequency' input motion at a rather low PGA (as compared to the conventional admissible PGA[15]).

Another significant and unexpected result is the output dispersion. Due to the vast choice of available constitutive relationship and difficulties generally encountered to keep this type of analyses under control, it was expected that the COV of time history outputs would be

[15] See developments in Section 3.4.2.

significantly higher than the COV of response spectrum outputs. This is not the case; even for those runs that correspond to larger nonlinear effects (runs 3 and 4), the COVs generated by response spectrum analyses are larger than those generated by time history analyses. This question of dispersion is further developed in Section 3.2.2.2.

3.2. Step 2: Simulations with Japanese input motions

3.2.1. Terms of reference of Step 2 and outputs provided by the participants

The outputs requested from the participants were the same as for the time history analyses carried out in step 1. Detailed outputs are presented in Annex VIII. The basic idea in step 2 was that each participant should use the same model as in step 1 and run both input motions (Ito-Oki and Kashyo dam) on a supposed virgin specimen. The objective was to observe the dispersion of outputs (measured by the COV) and to compare it with the dispersion of outputs in step 1.

The participants' attention had been drawn to the fact that the Japanese input motions were provided 'as recorded' (but base-line corrected). Consequently, for the sake of similarity, before applying them on the specimen, the time path of the input motions should be divided by a factor of $\sqrt{3}$, so that the frequency content of the signal is multiplied by a factor of $\sqrt{3}$ as compared to the natural frequency content.[16]

Top displacements provided by the participants are shown in Table 21 for both input motions. Mean and standard deviation of forces and moments at level 1 are shown in Table 22. Top floor response spectra are shown in Figs 35 and 36.

Regarding floor response spectra, it is clear that some participants have not captured the response. One may point out some similarity between Ito-Oki (PGA 0.19 g) and run 2 (San Francisco 0.13 g) outputs; although these two runs do not result in significant nonlinear effects, there is a significant dispersion of the top floor response spectra.

It is interesting to note that although the Kashyo dam PGA is significantly higher than the Ito-Oki PGA, it does not apparently result in a higher dispersion of the participants' contributions. This question of dispersion is addressed in more detail in Section 3.2.2.2.

3.2.2. Comments and lessons learned

3.2.2.1. General comments

Practically, Ito-Oki and Kashyo dam input motions cannot be regarded as really representative of the type of near-field input motions aimed at in the IAEA CRP because they both have a non-negligible energy content in a rather low frequency domain. For instance, Ito-Oki components have peaks between 2 and 2.5 Hz (the Ito-Oki spectral shape is very similar to the run 4 spectral shape).

Regarding Ito-Oki, the top displacement mean value provided by participants is 2.34 mm, meaning that the response basically remains in the elastic domain. On the contrary, in the case of Kashyo dam, the 11.38 mm top displacement means that the nonlinear regime plays a significant role in the response of the specimen.

[16] Similarly, the specimen eigenfrequency is equal to the scale 1-building eigenfrequency multiplied by $\sqrt{3}$.

TABLE 21. JAPANESE INPUT MOTIONS: TOP DISPLACEMENTS (MM) PROVIDED BY PARTICIPANTS

PARTICIPANT		Top displacement (mm) Ito_Oki *(J1)*	Top displacement (mm) Kashyo_Dam *(J2)*	Shaking table consideration	Time interval scaling
1	ANRA-Armenia	2.68	4.94	Yes	Yes
2	BAS-Bulgaria	5.19	6.34	No	No
3	AECL-Canada	1.70	7.23	Yes	Yes
4	BINE-China	2.72	14.55	No	Yes
5	FORTUM-Finland	8.02	25.14	No	
6	INSA-France	1.69	10.12	Yes	Yes
7	IRSN-France	1.46	7.14	Yes	Yes
8	AERB-India	6.22	15.39	Yes	No
9	POLIMI-Italy	2.00	15.05	Yes	Yes
10	JNES-Japan	2.06	17.84	Yes	Yes
11	KINS-Korea	1.83	21.07	Yes	Yes
12	KOPEC-Korea	1.94	7.63	Yes	Yes
13	PAEC-Pakistan	2.22	14.47	Yes	Yes
14	UTCB-Romania	1.90	9.54	No	
15	CKTI-Vibroseism-Russia	1.69	11.49	Yes	Yes
16	SAS-Slovakia	1.87	8.76	No	Yes
17	IDOM-Spain	2.59	5.60	Yes	Yes
18	METU-Turkey	1.78	19.46	Yes	Yes
19	TAEA-Turkey	1.68	17.92	Yes	Yes
20	HSE-UK	2.70	4.65	Yes	Yes
21	BNL-USA	2.99	19.60	No	Yes
Mean		2.50	12.31		
Standard deviation		1.22	5.32		
Coefficient of variation		0.49	0.43		

Maximum (dotted shading) Minimum (grey shading)

Note: Max. and Min. values are not included in the mean and standard deviation computations.

TABLE 22. JAPANESE INPUT MOTIONS: SHEAR FORCES AND BENDING MOMENT AT LEVEL 1

	Ito_Oki *(J1)*		Kashyo_Dam *(J2)*	
	Shear Force (kN)	Bending Moment (kN m)	Shear Force (kN)	Bending Moment (kN m)
Mean	53.7	158.8	117.2	308.3
Standard deviation	16.8	51.7	22.5	39.1
Coefficient of variation	0.31	0.33	0.19	0.13

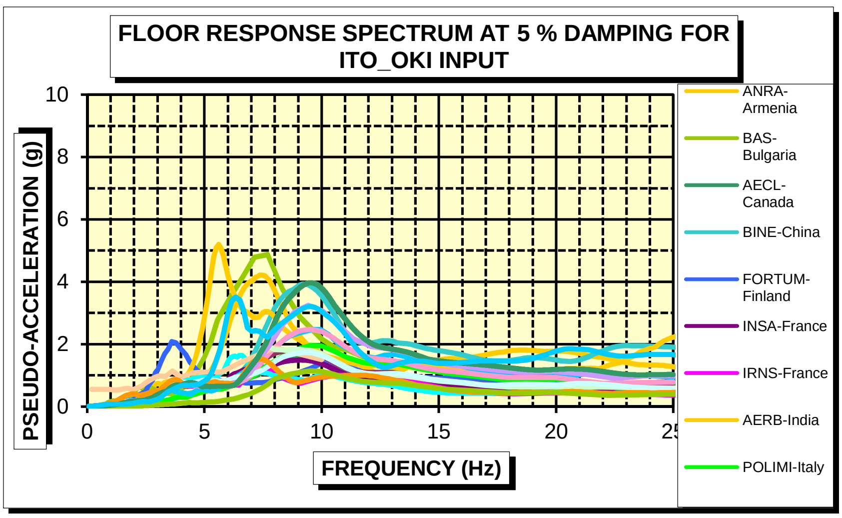


FIG. 35. Top floor response spectra provided by participants for the Ito-Oki input motion (J1), PGA = 0.19 g.

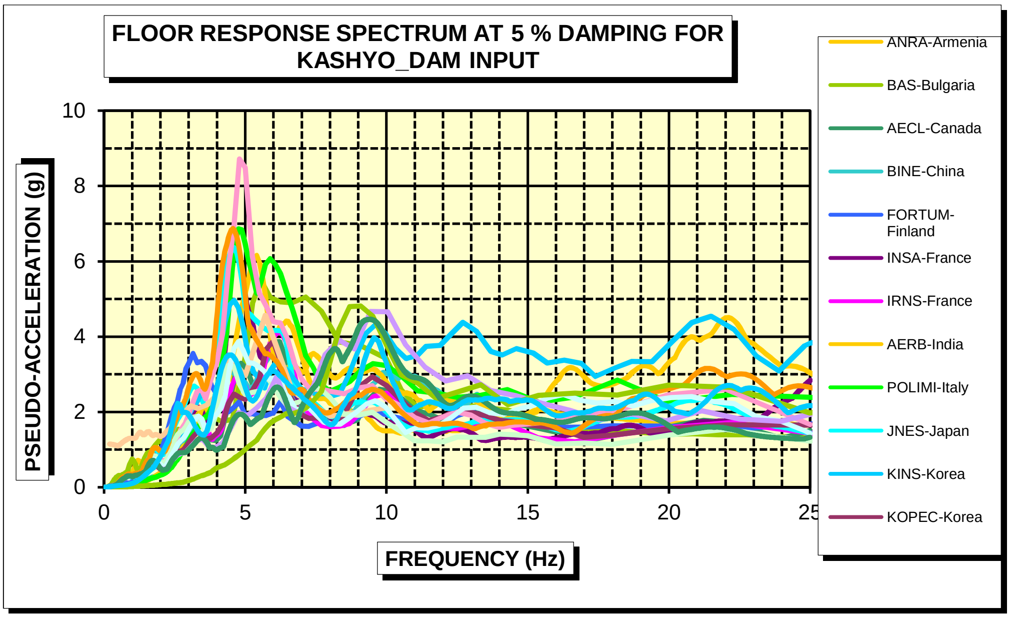


FIG. 36. Top floor response spectra provided by participants for the Kashyo dam input motion (J2), PGA = 0.53 g.

3.2.2.2. Dispersion of outputs

As opposed to step 1, the major interest of step 2 is that participants could not calibrate their respective outputs against experimental results. Therefore, examining the COVs and comparing them to step 1 output COVs is meaningful. COVs of top displacement and acceleration as well as of level 1 shear force and bending moments are plotted in Fig. 37 versus the top displacement of the corresponding run (the top displacement is considered as an indicator of the 'nonlinearity degree' of the run under consideration). Dispersion coefficients of step 3 outputs are also plotted in the same figure.

It is interesting to note that as opposed to what is frequently presented as obvious, there is no trend that the COV increases with nonlinearity. It is even the opposite with the Japanese input motions: the Kashyo dam input motion is stronger than Ito-Oki and leads further in the nonlinear regime; however, the COVs are smaller. This was already observed in step 1 outputs.

Furthermore, it was expected that the COV would be much higher under J1 and J2 analyses ('blind' calculation) than under TH, FF and NF[17] analyses (outputs of CAMUS runs were at the disposal of the participants). This is effectively the case for the rather low PGA Ito-Oki input motion, but surprisingly not for the high PGA Kashyo dam input motion.

Generally speaking, one should not be surprised by the low dispersion of forces and moments for large nonlinearity. It does not mean that participants concur on the response of the specimen; it just reflects the dispersion of the 'asymptotic' value of the pushover curve. Regardless of its exact value, to the extent the displacement is large, the associated force (or moment) is governed by this flat part of the curve.

[17] See Table 11 for the features of TH, FF and NF analyses.

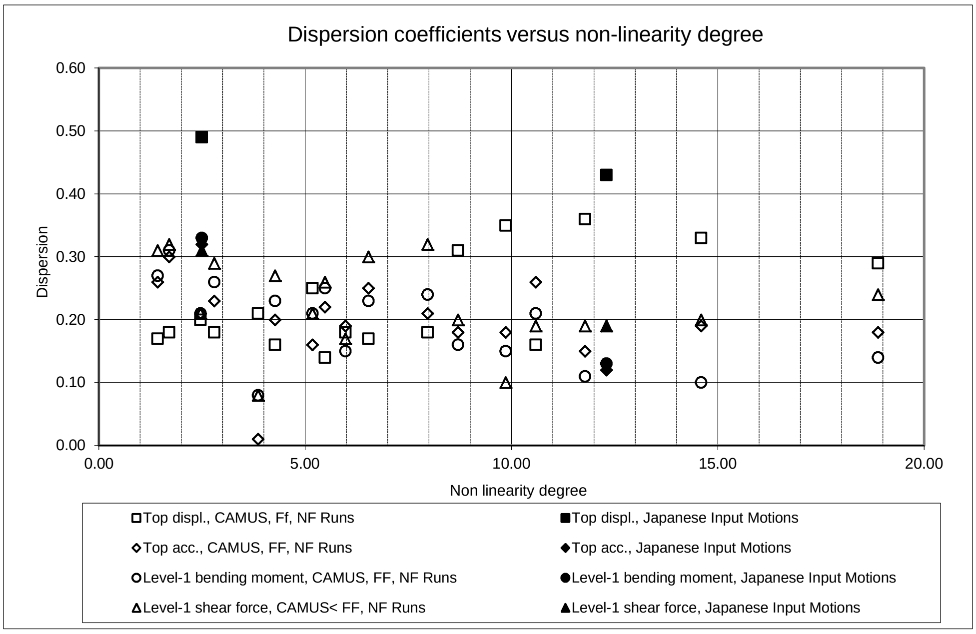


FIG. 37. COVs versus nonlinearity.

3.3. Step 3: Effects on floor response spectra generation

3.3.1. Terms of reference of step 3

According to conclusions of the second RCM, held in Trieste, the main purpose of step 3 is to investigate the effects of concrete nonlinear behaviour on floor response spectra generation. For this purpose, it is assumed that analyses carried out during steps 1 and 2 offered the participant the opportunity to polish their models of the CAMUS mock-up, and that consequently those models could be run so as to carry out 'numerical experimentations'.

In the wake of the initial intention of the IAEA CRP, this exercise is conducted on the basis of two CAMUS input motions, namely run 1 (Nice) and run 2 (San Francisco), regarded as respectively representative of a far-field and a near-field input motion, at least in the restriction of near-field input considered in the frame of this IAEA CRP. Two series of six time history analyses (runs) are carried out, one series with each type of input motion, as shown in Table 23.

TABLE 23. INPUT MOTIONS CONSIDERED IN STEP 3

Runs carried out in step 3 of the benchmark			
Far-field (FF) type (Nice)		Near-field (NF) type (San Francisco)	
Analysis identification	Input: CAMUS run 1 scaled at	Analysis identification	Input: CAMUS run 2 scaled at
FF1	0.1 g	NF1	0.1 g
FF2	0.2 g	NF2	0.2 g
FF3	0.3 g	NF3	0.3 g
FF4	0.4 g	NF4	0.4 g
FF5	0.5 g	NF5	0.5 g
FF6	0.6 g	NF6	0.6 g

Each run is carried out on a virgin model of the CAMUS specimen. No pre-damaging effect of a run on the following one is taken into account.

Basically, the expected outputs are a selection of previous time history outputs. They consist of:

- Top relative horizontal displacement versus time;
- Top absolute horizontal acceleration versus time;
- Shear force at level 1 versus time;
- Bending moment at level 1 versus time;
- Horizontal top pseudo-acceleration response spectrum at 5% damping.

3.3.2. Presentation of outputs provided by the participants

3.3.2.1. General observation

A comprehensive presentation of step 3 outputs is available in Annex IX. The more significant of them are presented here. Top displacement versus g-level of FF and NF types of input motions are shown in Fig. 38. It is clear in this figure that, for rather large PGAs, both relationships can be regarded as practically linear. This is a consequence of the fact that both signals were recognized as ‘high frequency’ input motions.

Of course, as opposed to top displacement, level 1 shear force and bending moment are not proportional to the PGA, neither for the FF nor for the NF series, as illustrated in Fig. 39.

From the outputs shown in Figs 38 and 39, it is clear that extrapolating top displacement outputs is possible to a reasonable extent, while extrapolating forces and/or moments would be very inappropriate.

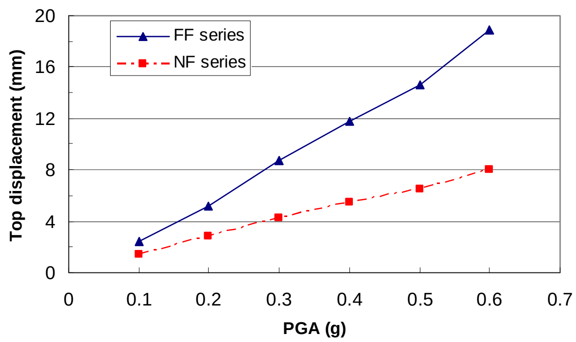


FIG. 38. Top displacement versus PGA for FF and NF series.

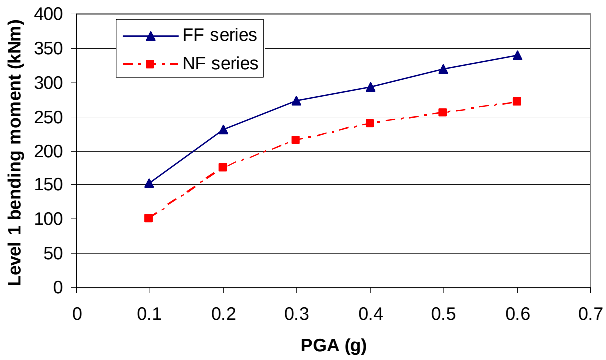


FIG. 39. Level 1 bending moment versus PGA for FF and NF series.

3.3.2.2. Floor response spectra

A comprehensive set of floor response spectra generated by the step 3 analyses is presented in Annex IX. As a matter of illustration, spectra corresponding to NF1 and NF6 runs are shown in Figs 40 and 41.

The interest of NF1 is that it leads to the lowest top displacement reported during the benchmark exercise, and consequently, the nonlinear effect is minimized. The dispersion observed in the figure reflects the model dispersion as presented in Section 3.1.2 However, a deeper investigation of participants' contributions would be necessary to discuss the large discrepancies in peak accelerations, which is not expected on the basis of the selected damping values.

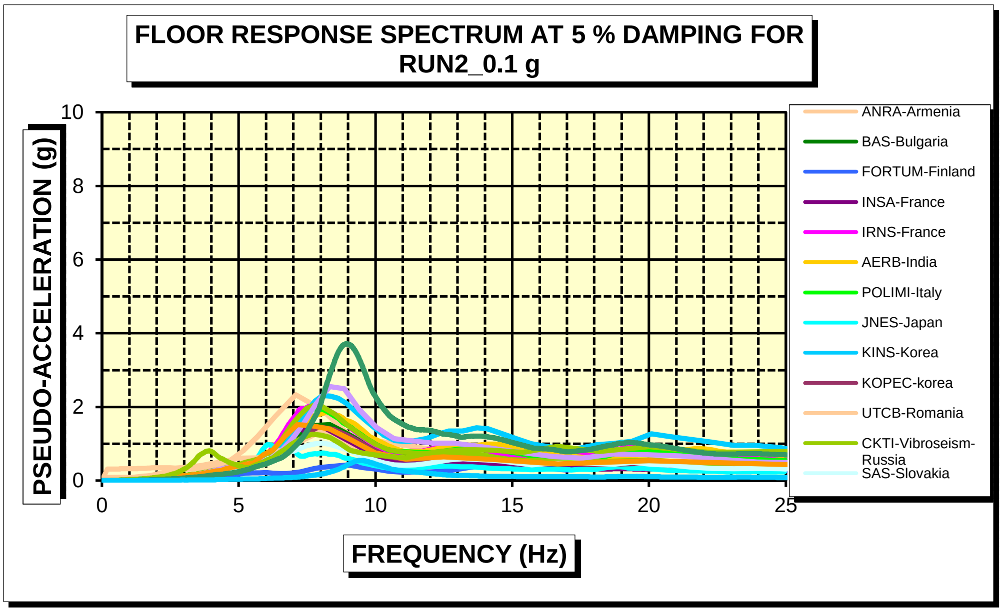


FIG. 40. NF1 top floor response spectra.

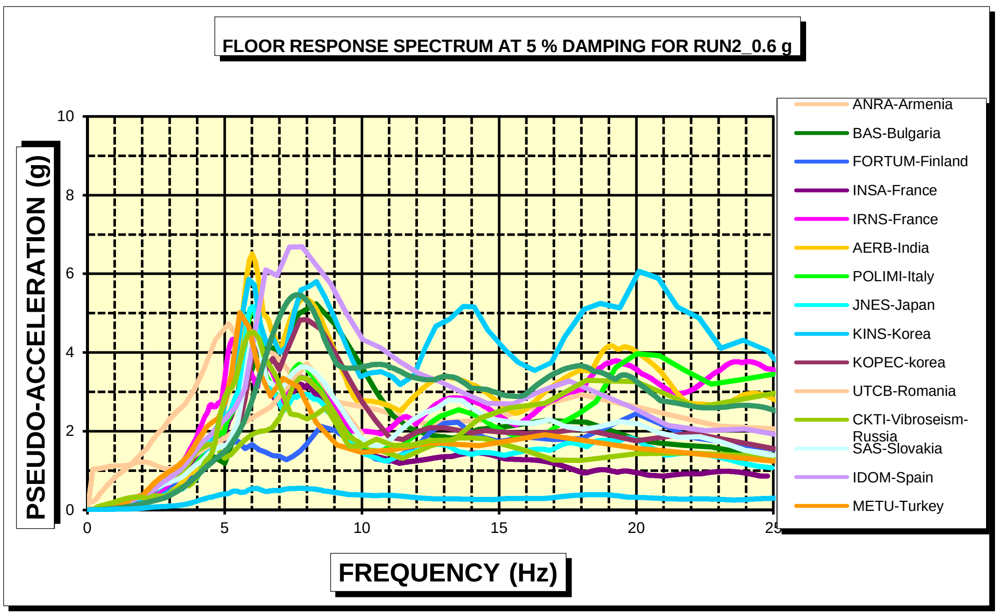


FIG. 41. NF6 top floor response spectra.

Comparison of NF1 and NF6 also leads to the conclusion that a significant saturation effect of the peak acceleration can be observed as well as a shift of the peak frequency in relation with the input PGA increase. The same phenomenon is observed in a similar manner on the FF series outputs. As a matter of fact, for most participants, nonlinear effects appear for very low PGAs, between 0.1 and 0.2 g for the NF series and around 0.1 g for the FF series.

It is difficult to present more deeply the set of response spectra as a whole because deriving average response spectra does not make sense. For instance, let us consider a set of narrow spectra, all of them corresponding to a 5% damping value, but with different frequencies. The average spectrum will be an apparently wide-band spectrum corresponding, for instance, to a 20% damping value. Therefore, it is more interesting to select some representative participants' outputs and to present them.

Two participants made their choice of a linear constitutive relationship, and consequently the top floor spectral shape they obtain does not depend on the PGA. Top floor response spectra are exactly proportional to the PGA. However, most of the participants accounted for a nonlinear behaviour of the specimen in their respective analysis. Typically, two types of consequences on the top floor response spectra were obtained.

(a) A first type consists practically only of a 'saturation' effect on the peak acceleration (the peak of the response spectrum does not increase with the g-level for strong input motions), surprisingly without any significant impact on the peak frequency; this type is exemplified in Fig. 42;

(b) In a second type of output, both effect of saturation of the peak acceleration and of shift of the peak frequency can be observed. This type is exemplified in Fig. 43 (spectra corresponding to 0.3 and 0.5 g are deleted for clarity).

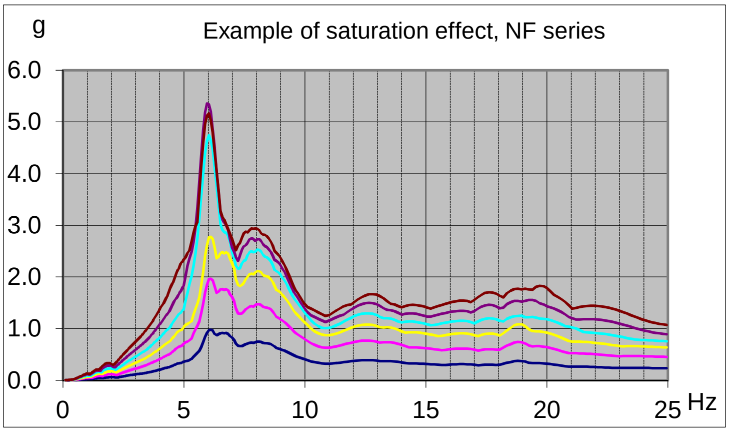


FIG. 42. Example of saturation effect on the peak acceleration.

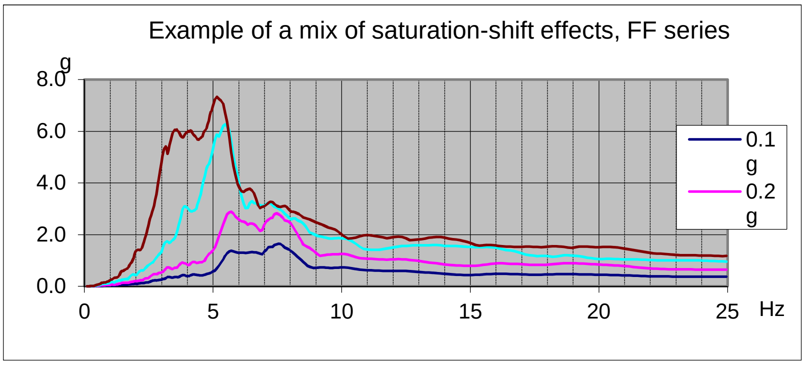


FIG. 43. Example of a mix of saturation of the peak acceleration and shift of the peak frequency.

The type-1 and type-2 effects are not governed by the model under consideration. The outputs shown in Figs 42 and 43 were obtained by the same participant, namely JNES.

Other effects are also reported by some participants. In particular, it can happen that the peak acceleration is far from increasing versus PGA. In such a case, floor response spectra concurrently undergo a peak reduction and a significant band broadening.

3.3.3. Comments and lessons learnt

3.3.3.1. Damaging capacity of near-field versus far-field input motions

Figure 38 illustrates, for a given PGA, the low damaging capacity of the NF type of input motion as compared to the FF type. As an example, we can assume that the CAMUS design has to be checked against the following input motions:

- A conventional input motion represented by FF2 (Nice 0.2 g);
- A near-field input motion represented by NF4 (San Francisco 0.4 g).

According to conventional nuclear power plant engineering practice (see Section 3.4), it would be concluded that FF2 is acceptable and NF4 is not. However, it is confirmed by the series of time history analyses that NF4 is not more damaging than FF2 and should be regarded as acceptable.

3.3.3.2. Computation of displacements and forces

Figure 39 illustrates that nonlinear effects appear for bending moments much lower than acceptable moments governed by the conventional limit state, and consequently for PGA levels much lower than acceptable conventional PGA (see Section 3.4.2). It is interesting to discuss the impact of this nonlinear behaviour of the most relevant outputs in terms of earthquake engineering.

From the step 1 outputs, it can be derived that these nonlinear effects do not have a major influence on computation of displacements, and at a first approximation could be neglected in displacement estimate. This conclusion is confirmed by the step 3 outputs: extrapolating the top displacement for a 0.6 g PGA from the top displacement computed at 0.1 g does not result in a major error. This is a consequence of the fact that for both signals were recognized as ‘high frequency’ input motions, or in other words as ‘displacement controlled’ inputs according to mechanical engineering terminology.

It was also observed at the end of step 1 that bending moment and shear force estimates based on a linear behaviour assumption are not as satisfactory as displacement estimates. However, they are acceptable for low frequency input motions (run 4) and are overestimated for high frequency input motions (with excess in the case of run 3), which does not jeopardize safety. This conclusion is also confirmed by the step 3 outputs. The same conclusion is valid for top accelerations.

3.3.3.3. Floor response spectra generation

It is frequent in Member States that, in the conventional nuclear approach framework, structural behaviour is regarded as (equivalent) linear, and floor response spectra are computed on this basis.

It is possible to simulate floor response spectra generated by this approach and to compare them to experimental results as well as to floor response spectra generated by more sophisticated methods such as those used by participants in step 3 of this benchmark. The simulation exercise is carried out as follows: An FF1 spectrum provided by a participant is selected and multiplied by a factor of 2.4 so as to get the floor response spectra that, assuming linear behaviour, would have been generated with the same input scaled at 0.24 g. This new floor response spectrum is then comparable to the run 1 experimental output. The results of this exercise are shown in Fig. 44 on the basis of FF1 spectra provided by two participants (Participants a and b in the figure), who made different assumptions for CAMUS specimen modelling.

The issues posed by floor response spectra generation are, to a large extent, not comparable to those posed by displacement and/or force assessment, and are certainly more complicated. For displacement and/or force evaluation, the assumption of linear or quasi linear behaviour may lead to acceptable outputs (either the prediction is correct or it is overestimated without risk of jeopardizing safety). Regarding floor response spectra generation, the same assumption provides a reasonable margin on the ZPA.[18] However, this margin does not make sense when dealing with the floor response spectra as a whole, because the crucial point is to capture the peak frequency shift, as can be observed in Fig. 44.

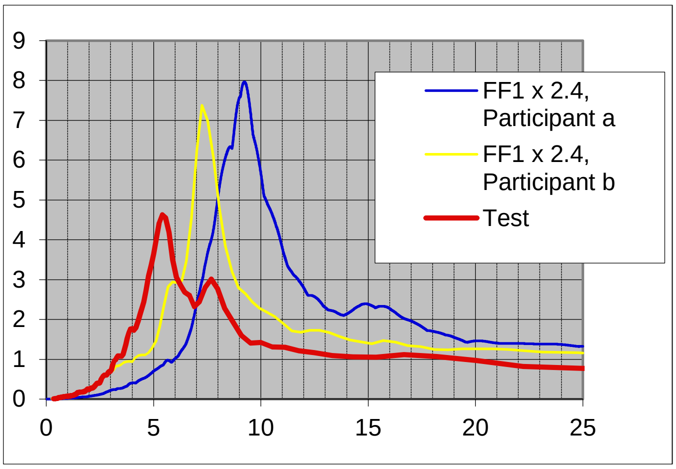


FIG. 44. Simulation of run 1 top floor response spectra generated by the conventional nuclear approach and comparison to experimental output.

In this regard, not accounting for any cracking effect leads to an excessive stiffness and the (first) eigenfrequency is significantly overestimated (Participant a in the figure). In such a case, classical peak broadening (±15% on the peak frequency) would not result in a spectrum that envelops the observed run 1 spectrum. The situation is better if the impact of expected cracking on structural flexibility is accounted for through a reduced E modulus of concrete; it results in significantly lower eigenfrequencies (Participant b in the figure). In this case, the conventional peak broadening could be sufficient to envelop the run 1 spectrum.

From the run 1 example, it is clear that a significant frequency shift is possible in a concrete structure, even without exceeding the conventional limit state (see Section 3.4.2). It is, therefore, necessary to account for this phenomenon. It is clear that this phenomenon cannot be captured without modelling the small nonlinearity effects that occur without exceeding design criteria. In this regard, it has to be mentioned that the predicted run 1 top floor response spectra (outputs of TH1 analysis) provided by half of the participants give evidence that capturing this phenomenon is achievable (Fig. 45). As a conclusion, it is possible to state that regarding floor response spectra generation, an evolution in nuclear power plant engineering practice is desirable and feasible.

[18] It is worth mentioning that this margin is correctly predicted by the response spectrum method (see Fig. 32), which is not surprising because both calculations are carried out on the same assumption of linear behaviour.

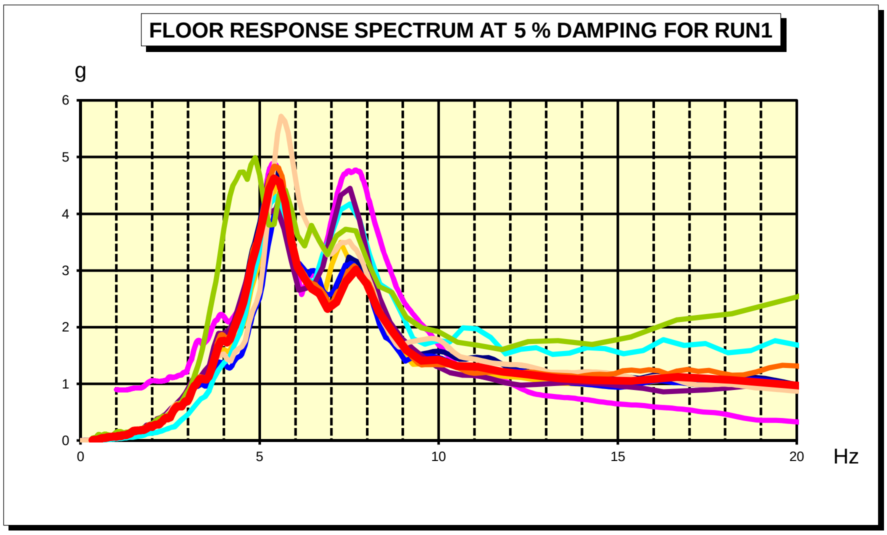


FIG. 45. Selection of run 1 top floor response spectra provided by participants (selected from Fig. 25).

3.4. Analysis of the benchmark outputs

3.4.1.Consistency and consolidation of the outputs

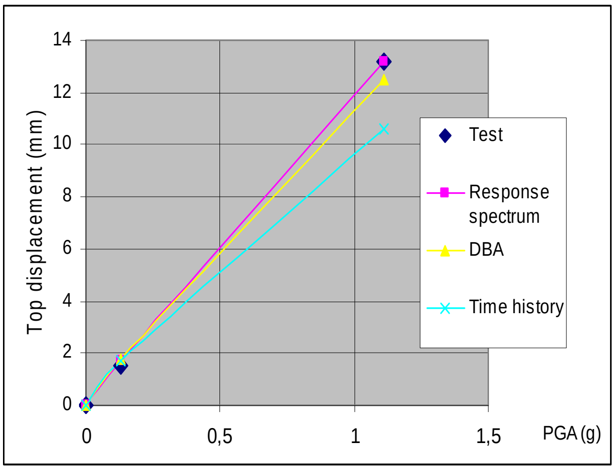
3.4.1.1. Families of runs

The CAMUS designers' intention was that runs 2 and 3 be two samples of the same input motion scaled at two different PGAs. As mentioned in Section 2.4.1, this objective was not fully met. However, the run spectral shapes are reasonably similar, particularly in the range of frequency of interest for the benchmark exercise, so that comparing test outputs as well as analysis outputs makes sense.

The corresponding data are gathered in Table 24. A first and major output is that the top displacements observed on the shaking table are strictly proportional to the input PGAs. This validates experimentally the assumption that such 'high frequency' seismic input motion acts as a 'displacement controlled' input. This result is confirmed by the different types of analyses carried out in the frame of the benchmark as illustrated in Fig. 46. In conclusion, for the purpose of the CRP, their outputs confirm that runs 2 and 3 may practically be regarded as two samples of the same input motion, scaled at two different PGAs.

TABLE 24. DATA OF CAMUS RUNS 2 AND 3

	Run 2		Run 3	
Inputs				
PGA	0.13 g		1.11 g	
Outputs				
Source of data	Top displacement (mm)	Level 1 bending moment (kNm)	Top displacement (mm)	Level 1 bending moment (kNm)
Test (Annex III)	1.54	75.5	13.19	280
S2 and S3	1.69	119.1	13.15	927.7
DBA2 and DBA3	1.78	117.1	12.47	294.8
TH2 and TH3	1.71	103.4	10.59	290.9



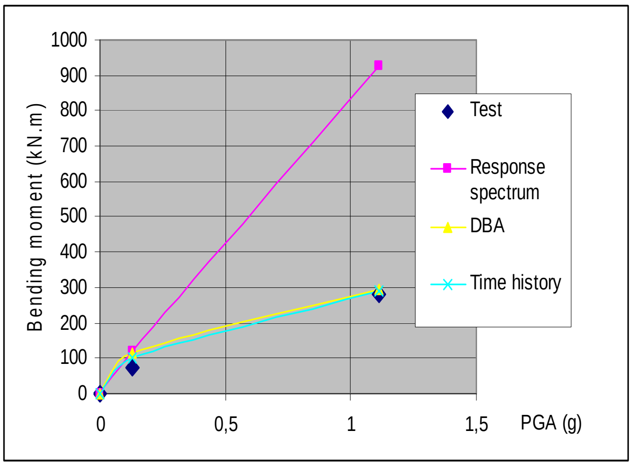


FIG. 46. Comparison of run 2 and 3 outputs: Top displacement (left) and level 1 bending moment (right).

Regarding level 1 bending moment, it is not surprising that response spectrum method outputs are proportional to input PGAs because this method assumes a linear behaviour of the specimen. Test outputs and other method outputs enable us to estimate, on the CAMUS example, margins generated by the response spectrum method when dealing with this type of 'high frequency, high PGA' input motions.

Furthermore, it should be mentioned that FF and NF series are based respectively on the run 1 and run 2 input motions scaled at different PGAs. Consequently, run 1 and 2 outputs should respectively be consistent with FF and NF series outputs. This is the case and consistently, in the remainder of Section 3.4, the outputs of the FF series and TH1 as well as the outputs of the NF series and TH2 are plotted together

3.4.1.2. Constitutive relationship of CAMUS wall level 1 section

As opposed to a pushover curve that integrates the structural response from the base to the top and is consequently affected by the type of input[19], a constitutive relationship, such as a bending moment–curvature relationship should not depend on the type of input under consideration. This means that all available outputs, either from static or dynamic analyses, should be consistent. This consistency is examined here, focussing on level 1 outputs.

For the sake of comparison with experimental outputs, the participants were requested to provide tensile strain estimates. Consequently, the analysis developed in this paragraph refers to tensile strains, and not directly to curvature. Processing strains also provides a more direct estimate of the conventional ultimate state, assuming that it is governed by the acceptable tensile strain in R-bars. In this section, a tensile strain of 1% is adopted as being acceptable.

The set of available [level 1 bending moment–level 1 tensile strain in R-bars] couples is shown in Table 25. Obviously, it is necessary to put aside conventional response spectrum analysis outputs because the relationship between moment and strain is assumed to be linear. In addition, it is clear that run 4 outputs, both in DBA and in time history analyses, result in a lack of regularity in the relationship and should be eliminated. Certainly, the puzzling pre-damaging effect of run 3 is the root cause of these unreliable run 4 outputs. Finally, the retained [bending moment–tensile strain] couples are shown in Fig. 47. An obvious consistency is achieved, regardless of the type of input under consideration.

[19] As developed in Section 3.4.1.4, the static pushover curve does not pertain for interpretation of dynamic outputs.

TABLE 25. BENDING MOMENTS AT LEVEL 1 AND ASSOCIATED TENSILE STRAINS PROVIDED BY PARTICIPANTS (AVERAGE VALUES)

Reference	Moment (kNm)	Tensile strain (10^{-3})
Pushover		
P-01	77.0	0.051
P-10	292.6	1.614
P-15	326.0	2.234
P-20	341.9	3.303
Spectral		
S1	353.9	0.581
S2	119.1	0.141
S3	927.7	1.580
S4	254.7	1.074
DBA		
DBA1	244.0	1.047
DBA2	117.1	0.255
DBA3	294.8	1.953
DBA4	225.9	1.131
Time history, CAMUS runs		
TH1	245.3	1.163
TH2	103.4	0.227
TH3	290.9	1.646
TH4	242.8	1.579
Time history, Japanese inputs		
J1	158.8	0.410
J2	308.3	1.929

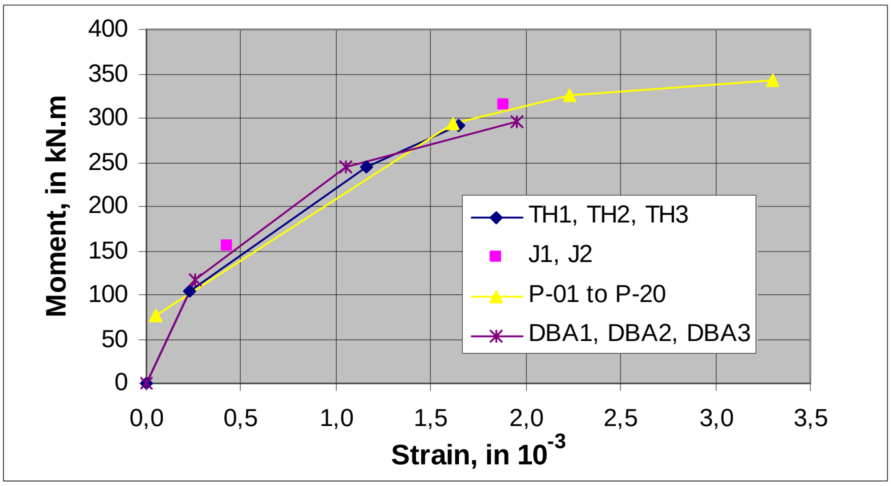


FIG. 47. Bending moment versus tensile strain in R-bars at level 1.

Concurrently, it is interesting to consider the level 1 bending moment–curvature relationship provided by the participants. A series as well as their mean curve are plotted in Fig. 48. In order to check the consistency between the graphs shown in Figs 47 and 48, it is necessary to have a reliable relationship between curvature and tensile strain. Such a relationship may be derived from the classical RC approach, as developed in Section 3.4.1.3, and is shown in Fig. 51. With this additional information, a consistency check can be carried out, which consists of comparing strains within their common range:

- The bending moment–tensile strain outputs mentioned above;
- The theoretical bending moment–tensile strain relationship derived from the classical RC approach;
- The bending moment–tensile strain relationship derived from the combination of moment–curvature and curvature–strain relationships.

This comparison is shown in Fig. 49. It is clear that again a very good consistency is achieved. A major consequence of this positive result is that it validates the theoretical RC approach and, on this basis, provides solid ground for extrapolating available outputs towards larger moments and tensile strains, as developed in the next paragraph.

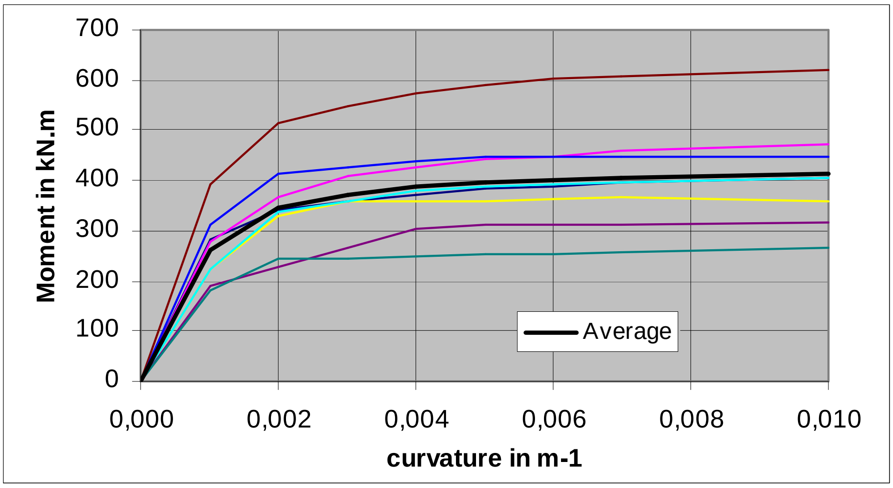


FIG. 48. Bending moment versus curvature at level 1: a selection of participants' outputs and corresponding mean value.

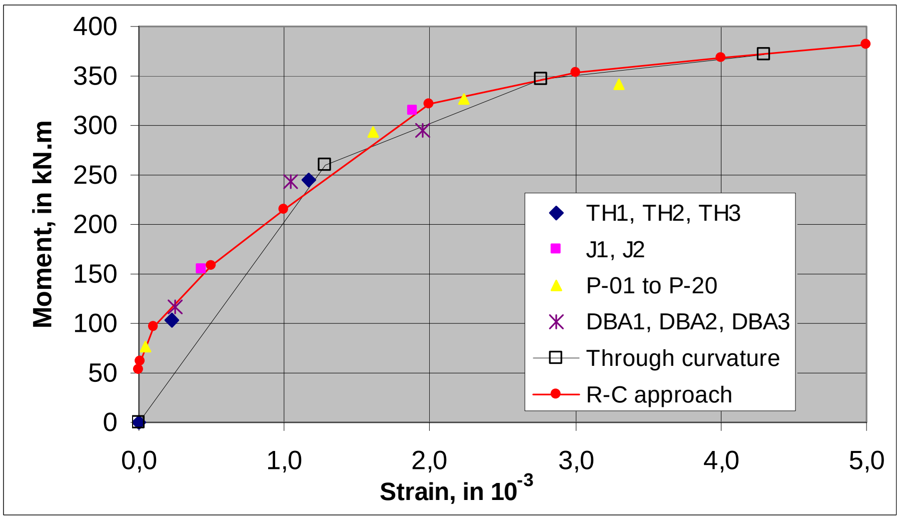


FIG. 49. Consistency of outputs shown in Fig. 47 with moment–curvature outputs and with the RC approach.

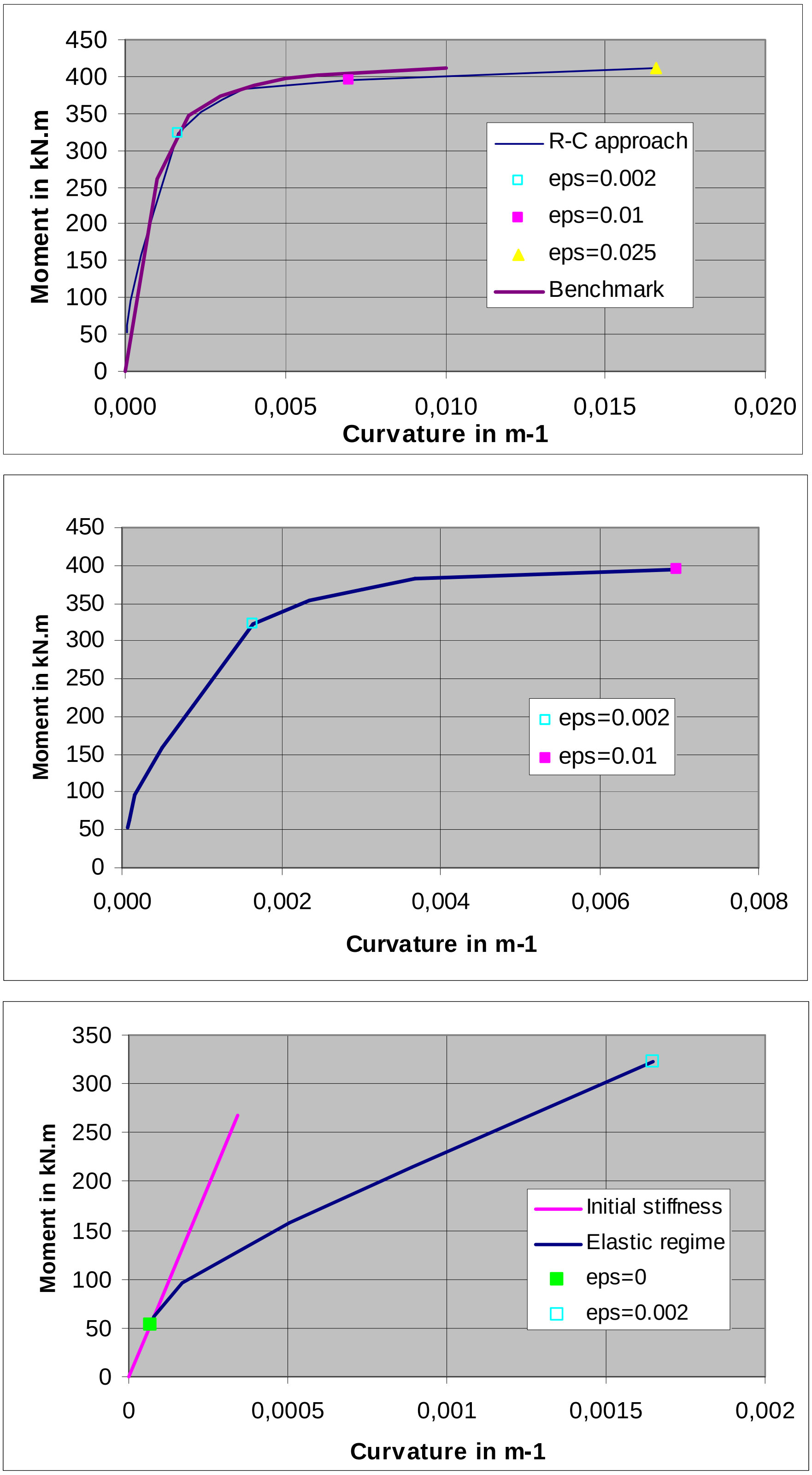


FIG. 50. Bending moment–curvature relationship at different strain scales according to the classical RC approach, for CAMUS wall level 1 (eps indicates the corresponding tensile strain in R-bars).

3.4.1.3. RC classical approach

It is possible to conduct an analysis of the level 1 section according to the conventional RC approach. Basically, it consists of adopting the Saint-Venant principle (cross section remains plane under lateral loading) and considering that concrete does not exhibit any tensile capacity. On this basis, relations between moment, curvature and tensile strains can be established.

The bending moment–curvature relationship is shown in Fig. 50 at different scales. At the larger scale, it is compared to the average of the participants' outputs; consistency is excellent and provides confidence in further using the RC approach, if necessary, for interpretation of benchmark outputs. At the smaller scale, the very narrow range of low moments and curvature governed by the initial section stiffness, EI, is also plotted.

The curvature–tensile strain and moment–tensile strain relationships are shown in Figs 51 and 52. An advantage of the former is that it is practically insensitive to uncertain inputs such as steel, plastic yield or hardening modulus (it is sensitive to dead weight, which is known).

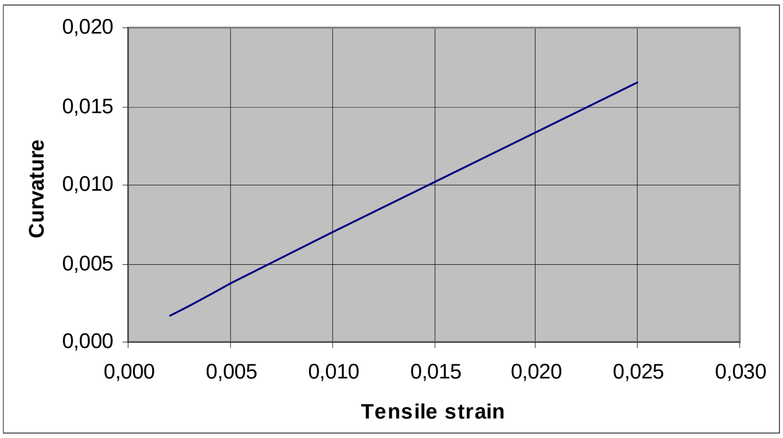


FIG. 51. Curvature–tensile strain relationship for CAMUS wall level 1 according to the classical RC approach.

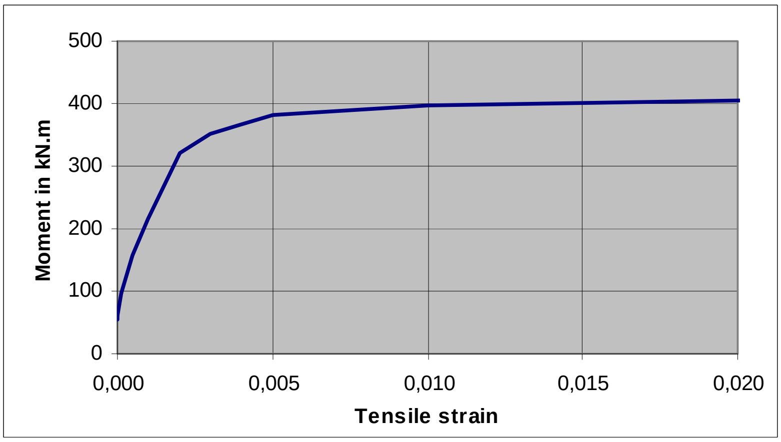


FIG. 52. Bending moment–tensile strain relationship for CAMUS wall level 1 according to the RC concrete approach.

It can be observed in Figs 50 and 52 that, due to the dead weight, a nil tensile strain does not correspond to a nil moment or a nil curvature.

Finally, on the basis of the RC approach, consolidated by the benchmark outputs, it is possible to conclude that, for level 1, the conventional ultimate state, governed by a 1% tensile strain in R-bars, is associated to a 0.007 m^{-1} curvature and a 396 kNm bending moment.

3.4.1.4. Consistency of time history outputs

Every time history output of the benchmark provides a set of [top displacement, level 1 shear force and level 1 bending moment], shown in Table 26. The purpose of this paragraph is to check consistency of these outputs and to derive features of the conventional limit state that will be useful in Section 3.4.2.

TABLE 26. OUTPUTS OF TIME HISTORY ANALYSES CARRIED OUT BY PARTICIPANTS

Inputs motions	PGA (m/s^2)	Top displ. (mm)	Base shear (kN)	Base moment (kNm)	Strains (10^{-3})
Run 1	2.4	5.98	79.1	245.3	1.163
Run 2	1.3	1.71	30.5	103.4	0.227
Run 3	11.1	10.59	112.5	290.9	1.646
Run 4	4.1	9.86	83.1	242.8	1.579
Ito-Oki	1.9	2.50	53.7	158.8	0.410
Kashyo_dam	5.4	12.31	117.2	308.3	1.929
Run 1 spectral shape (Nice)					
	1.0	2.47	45.9	152.1	
	2.0	5.18	72.7	231.9	
	3.0	8.71	92.7	274.2	
	4.0	11.79	102.9	293.8	
	5.0	14.60	117.0	318.7	
	6.0	18.89	123.7	339.1	
Run 2 spectral shape (San Francisco)					
	1.0	1.43	29.9	99.6	
	2.0	2.80	52.9	174.6	
	3.0	4.27	70.3	216.0	
	4.0	5.48	84.2	239.6	
	5.0	6.54	91.5	256.2	
	6.0	7.97	100.8	271.9	

In Fig. 53, [top displacement–level 1 bending moment] couples are plotted. It is clear that a remarkable consistency in the comprehensive set of outputs is achieved. Extrapolating the curve until the conventional limit of the level 1 bending moment seems reasonable. It leads to a corresponding top displacement that can be estimated between 25 and 30 mm. For the following, a 27 mm top displacement is regarded as representative of the ultimate limit state.

Results leads to the interesting conclusion that the ratio, level 1 bending moment: level 1 shear force, may vary significantly as illustrated in Fig. 54. The static pushover value of 3.3 m (bending moment/shear ratio) is confirmed only for low level inputs. A clear trend is that this ratio decreases for high level inputs. This evolution certainly reflects a slight change in the dynamic behaviour of the specimen in relation to some nonlinearity effects.

In other words, it might be concluded that the pushover profile is not representative of the CAMUS wall dynamic response. A profile such as $F(h) = h^{\alpha}$, $\alpha < 1$ would result both in a larger shear force associated with a given top displacement and in a lower value of the bending moment:shear force ratio. For instance, $\alpha = 0.5$ leads to a 3.03 m ratio and $\alpha = 0$ to a 2.7 m ratio, which is approximately the lower value reported in Fig. 54.

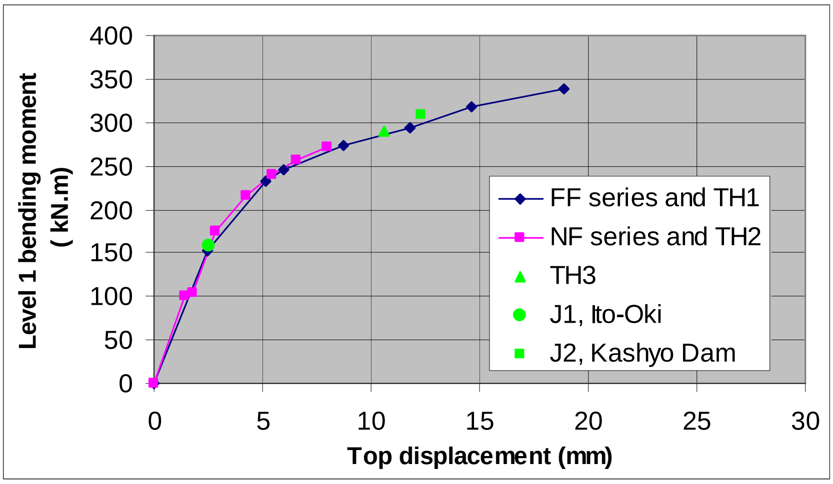


FIG. 53. Pushover curve resulting from time history analyses.

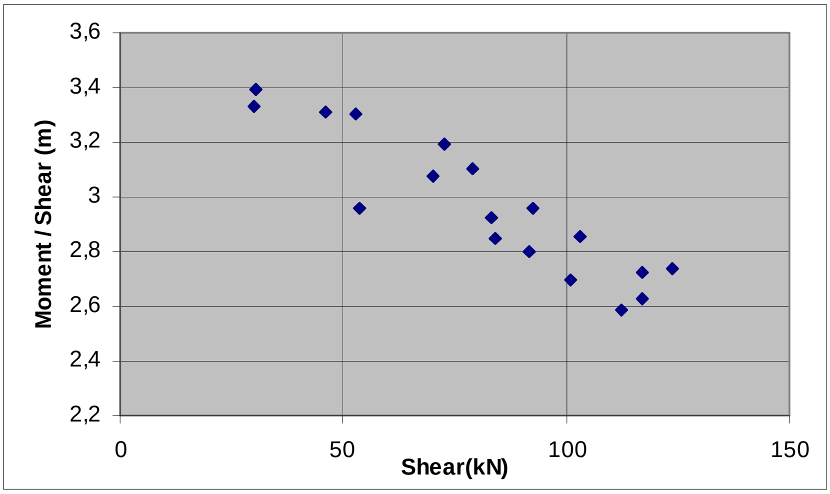


FIG. 54. Bending moment:shear force ratio at level 1 from time history outputs.

3.4.2. Conventional limit PGA and assessment of conventional nuclear approach

3.4.2.1. Interest and definition of a conventional limit PGA

The performances of different methods were presented and discussed at the end of Section 3.1 by examining different outputs: displacements, accelerations, forces, moments and tensile strains. In order to synthesize all of these performances, it is interesting to discuss the following question: For the spectral shape under consideration, what would be the acceptable PGA scaling value? This means the PGA that would be deemed to lead to the conventional limit state of the specimen, as determined at the end of Section 3.4.1. This PGA is defined as the 'conventional limit PGA'.

TABLE 27. MAIN FEATURES OF THE CONVENTIONAL LIMIT STATE FOR DYNAMIC RUN ANALYSIS

Tensile strain in R-bars	0.01
Level 1 bending moment	396 kNm
Top displacement	27 mm

In particular, in the context of nuclear safety, it is of major interest to discuss what the conventional limit PGA would be according to nuclear practice and to compare it to a best estimate of this conventional limit PGA. As a basis for discussing this question, the main features of the conventional limit state established in Section 3.4.1 are summarized in Table 27.

3.4.2.2. Conventional limit PGA according to nuclear practice

In conventional nuclear practice, the structural response is frequently assumed to be linear and is governed by the initial stiffness of the structure, and the response is computed according to the conventional response spectrum method. Computing CAMUS response according to this practice is the purpose of Section B of step 1 of the benchmark.

This type of response being assumed, the acceptance criterion for the level 1 section simply reads as follows: the computed bending moment should not exceed the conventional limit M_0 = 396 kNm (see calculation of M_0 in Section 3.4.1).

Consequently, the acceptable PGA is calculated as follows: For a given run X scaled at level γ_X (for instance run 3 is scaled at γ_3 = 1.11 g), M_X is the mean value of the bending moment resulting from SX analysis. The conventional limit PGA for this run X, denoted Γ_X, is then given by:

$$\Gamma_X = \gamma_X \times M_0/M_X$$

Considering for instance S3 analysis, the mean value of the bending moment at level 1 is M_3 = 940.4 kNm. Consequently, an estimate of the conventional limit PGA for an input with a response spectrum of run 3 shape is:

$\Gamma 3 = 1.11 \times 396/940 = 0.47$g

It is interesting to note that:

- Runs 2 and 3 result in a close PGA, which reflects the fact that, according to the experiment designers’ intention, the spectral shapes of those two runs are similar, as already observed in Section 2.4.1. An average value of 0.45 g can be retained;
- Run 1 and 4 analyses lead to significantly different PGAs, which reflects the fact that, as opposed to the experiment designers’ intention, the run 4 spectral shape was totally different from the run 1spectral shape, as also mentioned in Section 2.4.1.

TABLE 28. CONVENTIONAL LIMIT PGA ACCORDING TO NUCLEAR PRACTICE

	Run 1	Run 2	Run 3	Run 4
γ (g)	0.24	0.13	1.11	0.41
M (kNm)	351	119	940	265
$\Gamma = \gamma \times M_0/M$ (g)	0.27	0.43	0.47	0.61

3.4.2.3. Best estimate of conventional limit PGA

A best estimate of the conventional limit PGA can be derived from CAMUS experimental outputs, with the additional support of step 3 outputs. Data processed for this purpose are shown in Table 29. FF6 and Test 1 (Resp. NF6 and Test 2) results are shown in the same column because they correspond to the same input motion with two different scaling PGAs.

TABLE 29. CALCULATION OF BEST ESTIMATE ACCEPTABLE PGAs

<table>
<tr><td colspan="5">Experimental results (Annex III)</td></tr>
<tr><td>Test</td><td>Test 1</td><td>Test 2</td><td>Test 3</td><td>Test 4</td></tr>
<tr><td>PGA (g)</td><td>0.24</td><td>0.13</td><td>1.11</td><td>0.41</td></tr>
<tr><td>Level 1 moment (kNm)</td><td>211</td><td>75.5</td><td>280</td><td>276</td></tr>
<tr><td>Top displacement (mm)</td><td>7.0</td><td>1.54</td><td>13.2</td><td>13.4</td></tr>
<tr><td>Acceptable PGA/force controlled assumption (g)</td><td>0.45</td><td>0.68</td><td>1.57</td><td>0.59</td></tr>
<tr><td>Acceptable PGA/displacement controlled assumption (g)</td><td>0.93</td><td>2.28</td><td>2.27</td><td>0.83</td></tr>
<tr><td colspan="5">Benchmark step 3 results</td></tr>
<tr><td>Analysis</td><td>FF6</td><td>NF6</td><td>–</td><td>–</td></tr>
<tr><td>PGA (g)</td><td>0.60</td><td>0.60</td><td>–</td><td>–</td></tr>
<tr><td>Top displacement (mm)</td><td>18.89</td><td>7.97</td><td>–</td><td>–</td></tr>
<tr><td>Acceptable PGA/extrapolation (g)</td><td>0.86</td><td>2.03</td><td>–</td><td>–</td></tr>
<tr><td colspan="5">Synthesis</td></tr>
<tr><td>Acceptable PGA/retained values</td><td>0.9 g</td><td colspan="2">2.1 g</td><td>0.6 g</td></tr>
</table>

Regarding the tests, there are two possible options, both shown in Table 29, for extrapolating the outputs in order to derive an acceptable PGA estimate. Either the input motion is regarded as a force imposed input — then extrapolation is based on the bending moment value (this approach provides a default estimate of the acceptable PGA) — or it is regarded as a displacement controlled input — then extrapolation is based on the top displacement value (this approach provides an excess estimate of the acceptable PGA).

Regarding step 3 outputs, as mentioned in Section 3.3.2.1, top displacement outputs of both FF and NF series can be regarded as linear functions of the input PGA. Therefore extrapolating them seems reasonable. The corresponding estimates of acceptable PGAs are also shown in Table 29. Estimates derived from both test and step 3 outputs can be commented on as follows: Regarding runs 2 and 3, it was already stated in Section 3.4.1.1 that they are of the displacement controlled type. In spite of their very different PGAs, extrapolating test top displacements leads to very close estimates of acceptable PGAs: 2.28 and 2.27 g, respectively. This result is confirmed by step 3 outputs: the acceptable PGA is estimated as 2.03 g. As a compromise, a 2.1 g acceptable PGA is adopted for both runs 2 and 3.

Regarding run 1, the acceptable PGA derived from the displacement controlled assumption, 0.93 g is also consistent with extrapolation of the step 3 output, 0.86 g. As a compromise and rounded value, a 0.9 g acceptable PGA is adopted.

Regarding experimental outputs, the same procedure applies to run 4 and is shown in Table 27. According to the rationale developed in Annex X, and due to its 'low frequency' content, it is reasonable to think that this run is of the 'force controlled' type and that consequently the conventional limit pga should be closer to 0.58 g than to 0.83 g. Therefore, 0.60 g is retained as a round value.

3.3.4.2. Confirmation of Newmark's rule

For the three different spectral shapes (run 1, runs 2 and 3, and run 4 shapes as already identified at the end of Section 2.4.1) of input motions, conventional limit PGAs as estimated by the conventional nuclear approach are compared to best estimate values in Table 30. The overestimate of the damaging capacity of each type of input motion is also calculated as the ratio of the best estimate PGA to the nuclear approach estimated PGA.

TABLE 30. OVERESTIMATE OF THE DAMAGING CAPACITY OF HIGH FREQUENCY INPUT MOTIONS IN CONVENTIONAL NUCLEAR PRACTICE

Spectral shape	Run 1	Runs 2 and 3	Run 4
Low/high frequency input	HF	HF	LF
Conventional limit PGA, response spectrum method	0.27 g	0.45 g	0.61 g
Conventional limit PGA, best estimate	0.9 g	2.1 g	0.60 g
Overestimate of the damaging capacity	3.3	4.7	1.0

This table clearly illustrates and confirms that, when dealing with high frequency input motions, the conventional nuclear approach significantly overestimates their damaging capacity. The root cause of the phenomenon was recognized by Newmark early on and confirmed by other scientists — it is the fact that high frequency input motions act as and should be regarded as displacement controlled loads. In such cases, the conventional nuclear approach overestimates the damaging capacity of the input motion by a factor that equals the available ductility.

This overestimate is confirmed by outputs of the CAMUS experiment. The available ductility of the specimen is shown in Fig. 55 and its value is estimated to be 4.3 (on the basis of linear behaviour, the top displacement corresponding to the conventional limit of the bending moment is 6.25 mm; consequently, the available ductility is 27/6.25 = 4.3). This available ductility is in excellent agreement with the overestimate for runs 2 and 3, which can be regarded as purely of the displacement controlled type. This is not exactly the case for run 1; the overestimate is lower than the available ductility, meaning that run 1 input should not be regarded as 100% of the displacement controlled type. The phenomenon can be explained by the response spectrum slope in the range of frequencies of interest.

Historically, the conventional nuclear approach was established in order to deal with the conventional design situation, which consists of evaluating effects of rather low frequency input motions (such as represented by the NRC response spectrum) on stiff buildings such as reactor buildings of nuclear power plants. An analysis method (response spectrum method) and criteria (conventional limit state) were selected accordingly. This approach proved to be effective and reliable in the context of a conventional design situation, i.e. situations before the recording of high frequency ground input motions.

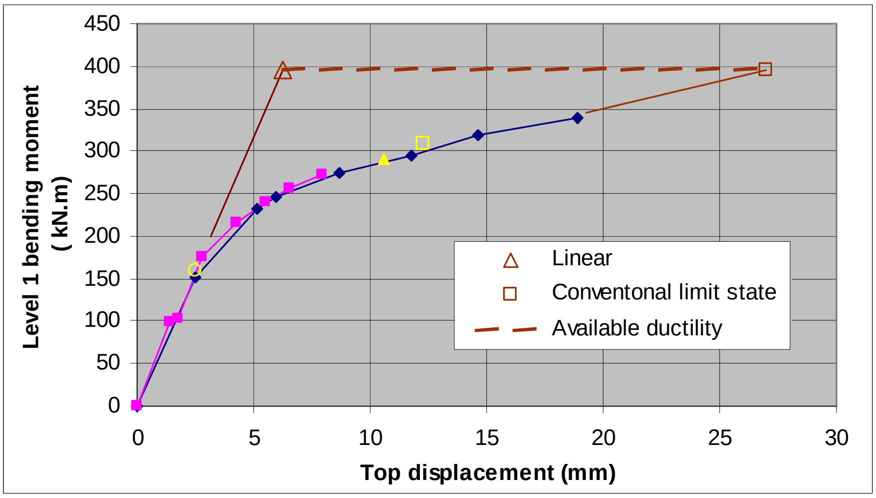


FIG. 55. Available ductility without exceeding the conventional limit state.

4. STRUCTURAL ENGINEERING PRACTICES AND THEIR POSSIBLE EVOLUTION

4.1. Current structural engineering practice in the nuclear industry

4.1.1. General considerations on the diversity of the available methods

4.1.1.1. Force based versus displacement based approaches

For historical reasons, conventional structural engineering practice is founded on two pillars:

(a) The elastic behaviour assumption;

(b) The force based approach of acceptance criteria.[20]

In this conventional practice, forces (or stresses) under given load cases are estimated assuming an elastic behaviour of the structure and are compared to acceptable values. This comparison is the substance of the acceptance criteria. It is noticeable that criteria are generally based on strain considerations[21]; this fundamental lack of consistency is a major source of difficulties in the implementation of the force based approach. However, the education of engineers (at least until very recently), design codes and computer tools are based on the force based approach.

This conventional practice is adequate for dealing with force controlled load (loads such that the induced stress is proportional to the load). In the case of displacement controlled loads (for instance thermal loads) for which the strains, and not the stresses, are proportional to the load, the conventional practice does not work properly. In order to cope with this difficulty, concepts of primary and secondary loads (or stresses) have been introduced. They are presented and commented on in the Safety Reports Series No. 28 [19]. A major difference between primary and secondary loads is that primary loads can lead to catastrophic failure and consistently the acceptance criteria associated with primary loads are much more severe than those associated with secondary loads.

The case of the seismic load is a complicated one, depending on the respective frequency content of the structure and the input motion, it should be regarded either as a primary load or as a secondary load, as explained by Newmark as early as 1978 [8] and illustrated in Ref. [9]. In

[20] The reference Method M1 includes these two features.
[21] Civil engineering criteria such as ULS, as well as mechanical engineering criteria.

Section 4.1.2, a series of methods is presented and the way they (implicitly) classify the seismic load is discussed.

As opposed to the force based approaches, the displacement based (or strain based) approaches are characterized by their consistency between the structural analysis, based on the analysis of strain or (relative) displacements and the acceptance criteria, also based on strains or (relative) displacements. They require skills in modelling of the material nonlinearity, which is the main difficulty for their use. As they are the subject of this document, they will be discussed in more detail later in this section.

4.1.1.2. Design of new structures

The acceptance criteria should characterize the safety of a structure in a manner independent of the type of analysis performed. Nonetheless, they are necessarily linked to the type of analysis, as the analysis should provide the information required for the application of the criteria.

The most frequently used method in the design process of new nuclear structures has been well established for a long time. In this document, it is referred to as the 'standard procedure' or the 'reference method' and will be called M1. Basically, M1 is a conventional structural engineering practice as introduced in the previous paragraph, based on the assumption of elastic behaviour of the structure and on the conventional response spectrum method. Apart from this method, it should be noted that the conventional Japanese method for the design of nuclear facilities is based on time history analyses, which enlarges the range of the possible methods.

Although, for the vast majority of Member States that have adopted it, it seems reasonable to go on with the standard procedure for the design of new structures, there are also needs for further developments at the design stage, for instance:

- It is more and more frequent that the design stage is supplemented by a margin assessment stage so as to check that there is no cliff-edge effect in the seismic response of the installation (due for instance to brittle elements);
- As demonstrated by the benchmark exercise, it is clear that, even in the frame of the conventional design criteria there is a necessity to address the nonlinear effects when deriving the floor response spectra.

Obviously, these questions cannot be answered by the standard procedure. Therefore, more sophisticated methods, such as presented in the next subsection, should be envisaged, even at the design stage.

The use of behaviour factors (Method M4 in the next subsection) is well established by design standards for conventional buildings. It implies the implementation of detailed design features and of identified structural configurations. On the one hand, these provisions are not necessarily appropriate for the design of some nuclear structures, such as nuclear power plant or reprocessing plant structures that are often massive, with large over-strength, but often less ductility than more conventional elements. Therefore, using this approach for the design of nuclear power plants or reprocessing plants is not envisaged.

On the other hand, there are also nuclear facilities with low nuclear inventories, such as some laboratories and/or research reactors. For these facilities, it is envisaged to develop a graded safety approach, with criteria (based on the nuclear inventory and other data) less severe than for the design of nuclear power plants but more severe than for the design of conventional buildings. An interesting feature is that, in general, these facilities have structures similar to conventional building structures. Therefore, it is also of interest to investigate the possible use/adaptation of the methods presented in the next subsection in view of dealing with the design of such facilities.

As a general comment to the development of methods applicable at the design stage of nuclear facilities, we should remember a major conclusion of the August 2003 IAEA symposium: The interest and the objective of the nuclear industry is to refine the modelling and understanding of the behaviour of structures in the immediate post-elastic domain, in the limits of the conventional design criteria, without the necessity to evaluate the ultimate state of structures. Therefore, the development of methods should avoid unduly sophisticated developments.

4.1.1.3. Assessment of existing structures

In the case of existing structures that have to be reassessed either because the earthquake has been poorly taken into account at the initial design stage or because the seismic event has been re-evaluated, two situations may occur:

(a) The structure can be proved to be acceptable on the basis of standard design procedure, which is rarely the case;

(b) The acceptability of the structure can be substantiated by taking into account its nonlinear behaviour, with adapted acceptance criteria.

We focus now on the second case, the first one being of little interest in the frame of the present document.

This situation is addressed in Safety Reports Series No. 28 [19] for the case of nuclear power plants. It results in Method M3 presented in the next subsection. Basically, the method was designed in the framework of conventional structural engineering practice and includes reduction factors that apply to forces (or stresses). It is calibrated so that it is less demanding than for the design of new nuclear power plants, but more demanding than for the verification of conventional buildings. This method was widely used for the evaluation of existing nuclear power plants.

The behaviour factor method (M4 in the next subsection) was also designed in the frame of conventional structural engineering practice. This well established method for the design of conventional buildings may be useful for the evaluation of existing nuclear facilities and, with adaptations, was used for this purpose in some Member States.

It is generally the case that, when dealing with older facilities, detailing is not in accordance with the requirements of the codes for the use of a behaviour factor. Therefore, the ductility present in the structure is neither homogeneous nor established, and some elements should perhaps be considered as brittle. So, in such cases, the use of a displacement based analysis offers the advantage of a better assessment of the zones of the structure that may become critical and of their order of criticality. Moreover, as in the process of analysing the evolution of the structure, when it is submitted to an increasing action, the state of strain is known at each step of the analysis, the ductility demand is known in any critical zone and the potential brittleness can be more easily controlled in the appropriate zones. The displacement based methods (Method M5 in the next subsection) are now widely recognized as powerful tools for the assessment of existing conventional structures.

4.1.1.4. Comment on the link between safety and structural analysis

In the types of analyses considered above, it may happen at certain steps that very weak elements may jeopardize the continuation of the process, i.e. the analysis is stopped because only one or a limited number of elements has reached failure, for instance because it is brittle. It should be emphasized that there may be a strong interest in continuing the analysis beyond this point, so as to assess a realistic capacity of the structure to withstand the earthquake. Of course, in such situations, the remaining resistance and stiffness of the weak elements should be carefully assessed, or even it may be assumed that the weak elements have failed and this has resulted in the failure of some neighbouring elements, even in a local collapse. The acceptability of such an assumption relies solely on the safety analysis of the degraded situation.

4.1.2. Presentation of a range of methods used in the structural engineering practice

In Fig. 56, a curve (which is for instance a capacity curve, as described in Section 3.1.2) is presented that indicates domains of application of different methods of analysis, M1 to M6, which are detailed in this Section.

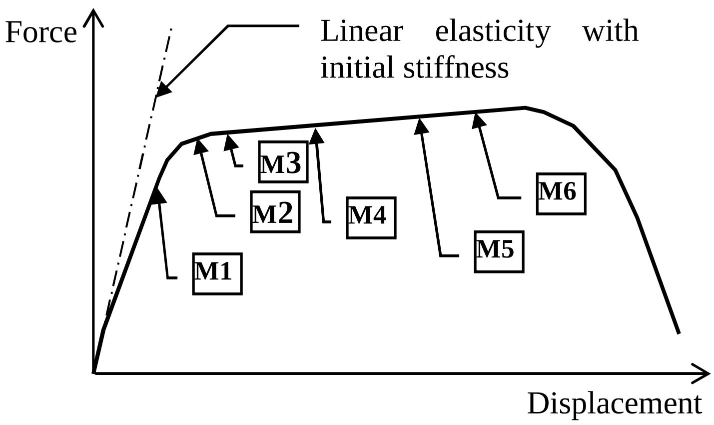


FIG. 56. Domains of application of methods presented in this section.

4.1.2.1. Method M1: linear analysis

Description

This procedure is based on a linear analysis of the structure and verifications at limit states. It entails four steps:

- Modelling assuming a linear elastic behaviour of the structure. The stiffness of uncracked concrete is used in this modelling. The linear soil–structure interaction is taken into account, including its influence on the modal damping. ‘Stick’ or finite element models may be used. Frequently, two models are made: the global model for performing the dynamic response of the building and the local model for performing the structural analysis.
- Analysis stage 1: The dynamic response analysis is linear, in most cases modal with use of an elastic response spectrum. In a few cases (for instance to estimate a more realistic magnitude of the uplift of a building or other unilateral behaviour), time history analyses are performed with limited geometric nonlinearities. It has also become rather common practice to perform linear analyses in the frequency domain to better assess the effects of soil–structure interaction. However, finally, the material behaviour of the structures is assumed to remain linearly elastic.
- Analysis stage 2: The calculation of strains and stresses in all structural elements — often termed structural analysis — is also based on linear assumptions. It may be performed with the same model as the dynamic response analysis, or with a more detailed one. It may be dynamic or static analysis.

- The verifications are usually the same as those performed for more common types of structures, considering limit states. Basically, the verifications of capacity and stability are performed at the ultimate limit state (ULS), considering accidental situations associated with an SL2 earthquake. It should be noted that the verification at ULS is performed at the local level on the basis of limit strains, and implies some inelastic behaviour. Nonetheless, such behaviour remains limited to very concentrated areas where the internal forces are at a maximum over the structure and the ULS is reached, thereby not jeopardizing the global elastic behaviour of the structure. At serviceability limit states (SLS) associated with an SL-1 input, criteria are mostly based on limit stresses.

Comments

The spirit of the method is that the input motion generates forces of inertia. Generally, these forces are calculated by multiplying the mass of every floor by its computed maximum acceleration. This means that the seismic load estimated in this way is regarded as a primary load and consistently a static equilibrium has to be achieved under this load. Forces of inertia that are computed assuming an elastic behaviour are regarded as valid and have to be taken into account in the structural analysis. They are addressed as any other forces and combined with them.

The use of this procedure results in a rather simple, robust and safe method of design. However, there is a lack of consistency between the method of analysis and the acceptance criteria; for instance, for RC, the verification of a specific section at ULS implies cracking and yielding of steel and concrete, although the structural analysis remains elastic and is even based on a non-cracked stiffness. A consequence is that margins are neither precisely known, nor consistently understood throughout the building.

4.1.1.2. Method M2: linear analysis with redistribution

Description

This method may apply if a limited number of elements are under-designed according to M1. It may be applied to structures made of RC and masonry infill.

- The first step consists of the standard procedure M1, resulting in the conclusion that some elements are beyond their acceptable ULS. In a more refined approach, it is also possible to evaluate cracking in structural elements that do not exceed ULS and to account for it.

After that, an iterative process is conducted on the stage 2 analysis:

- For each of the overstressed elements, a ‘linear equivalent’ reduced stiffness is evaluated (it may correspond to a rather extensive cracking) and the local model is updated accordingly. The linear structural analysis is also updated, resulting in a redistribution of internal forces (shear forces, bending moments);
- The verifications are then conducted as in M1. It may happen that new overstressed structural elements appear, leading to a new updating of the local model and structural analysis.

Either the process converges, meaning that a static equilibrium has been achieved, or it does not, meaning that the structure cannot be qualified with this method.

Comments

(a) Comparison with the plastic hinge method

In spirit this method is similar to the plastic hinge method, which was developed for the analysis of steel frames and may be summarized as follows:

- The first step also consists of an elastic analysis leading to the conclusion that some elements are submitted to states beyond their ULS.

After that, an iterative process is conducted:

- The local model is updated, taking into account the fact that every element cannot be stressed beyond its ULS (plastic moment). The linear structural analysis is updated accordingly, resulting in a redistribution of internal forces;
- At the verification stage, it may happen that new overstressed structural elements appear, leading to a new updating of the local model and structural analysis.

Either the process converges, meaning that a static equilibrium has been achieved, or it does not, meaning that the structure cannot be qualified with the plastic hinge method.

(b) Comparison with respect to M1

It is rather frequent that M1 is directly applied on a model that accounts for the expected cracking under seismic input. This can be regarded as an implicit first M2 iteration.

Consistently with the above reference to the plastic hinge method, it is clear that in the spirit of the method the seismic load is still regarded as a primary load.

The method is still rather simple; however, it is necessary to evaluate the equivalent stiffness of the overstressed elements, which may present some difficulties.

The consistency between the method of analysis and the acceptance criteria is improved. The method leads to a better estimate of margins. We should not forget that the plastic hinge method was introduced in order to get more homogeneous margins in the design of steel frames.

(c) Comments on the final state

When the process converges, the final state complies with both the static equilibrium and the compatibility of strains. However, the constitutive relationships are rough; therefore, the designer has to verify the consistency of the assumed constitutive relationship with the final state as it results from the analysis.

Finally, this state of stress corresponds to a nearly elastic state, with some limited areas of reduced stiffness where the displacements are contained by those elements that remain elastic. Therefore, the yielded elements are submitted to limited strains, in the vicinity of their elastic domain, that they can usually support, even if they are somewhat brittle. Consequently, such an analysis should be considered as a limited post-elastic analysis where the actual available ductility is not considered.In this method, the seismic action (output of analysis stage 1) is not supposed to vary, which is generally the case when the number of weak elements is limited.

However, it is always possible, at each step of the iteration, to update the seismic actions in performing a dynamic or modal analysis with the secant stiffness. In such a case, the iterative loop also includes the stage 1 analysis.

4.1.2.3. Method M3: linear analysis with the use of reduction factors

Description

This method is developed in Safety Reports Series No. 28 [19]:

- The modelling is similar to M1, except that it should be based on a best estimate approach;
- The analysis is similar to M1;
- At the verification stage, internal forces are reduced in each element by the inelastic energy absorption factor Fμ, which is linked to the ductile capacity of the element under consideration. Possibly, this factor also includes some over-strength that may exist in some types of elements. The Fμ factor depends on the type of element and the type of internal action considered (e.g. bending moment or shear force). Concurrently, the displacements resulting from the elastic analysis are increased by a factor in the order of magnitude of the Fμ factors;
- In the case of an extensive use of Fμ factors, it is recommended to assess the reduced stiffness of the structural elements and to update the computation of the dynamic response.

Comments

In this method, as Fμ factors may be different from one element to the other, static equilibrium is not achieved, which means that the seismic load is not regarded as primary. On the other hand, compatibility of strains is achieved. However, the displacements that are computed assuming elastic behaviour are not regarded as valid (they are increased at the verification stage), which means that the seismic load is not regarded as secondary. The seismic load is neither a primary nor a secondary load; the background justification of the method is based on a dynamic equilibrium.

The method is rather simple and robust. However, if the computation of the dynamic response has to be updated, evaluating the equivalent stiffness of the overstressed elements may present some difficulties. There is good consistency between the method of analysis (best estimate, update of the modelling) and the expected post-elastic behaviour of the structure. The method refers to and relies on the feedback of experience (e.g. the displacement field is amplified). This method was widely used for the evaluation of existing nuclear power plants. Developments on the spirit of the method are available in Safety Reports Series No. 28 [19]. The comments below about the use of M4 in a nuclear context are also valid for M3.

4.1.2.4. Method M4: linear analysis with behaviour factors

Description

This method is a codified method that usually applies to the design of new conventional structures. It is not directly applicable if the associated detail design rules are not met. The main steps are as follows:

- The modelling is based on the assumption of a linear elastic behaviour of the structure; however, a reduced stiffness is generally used that accounts for cracking effects. The SSI effects are neglected;

- The dynamic analysis is similar to M1, except that the seismic input is divided by a q factor for the computation of forces (not for the computation of displacements);
- The structural analysis and the verification phases are similar to M1.

Comments

The behaviour factor method is based on the assumption of a global ductile behaviour of the structure. This implies a global dynamic response that does not exhibit (strong) irregularities and a rather homogeneous design of all structural elements, including homogeneous ductility.

Basically, in the spirit of the method, the seismic load is regarded as a secondary load (the displacements computed on the assumption of an elastic behaviour are considered as valid). The q factor was invented so as to accommodate conventional engineering practice that is tailored in order to deal with force controlled loads. A static equilibrium is not achieved under the seismic load under consideration, but only under a reduced load. The theoretical background justification of this is a dynamic equilibrium. Similarly to M1, the method is simple and robust, but there is a lack of consistency between the method of analysis and the expected post-elastic behaviour of the structural elements.

Comments on the use of the method in a nuclear context

The use of a behaviour factor for the assessment of nuclear buildings can be based on a detailed analysis of the various structural elements to assess their respective ductility μ. This analysis comprises, for instance, for concrete structures: the quantity of transverse reinforcement in plastic hinges, the resistance to shear forces, the overlap and the anchorage of R-bars, the buckling of R-bars submitted to compression, etc. With this mapping of μ factors, it is possible, when also considering the regularity of the structure, to assign a value of a q factor (the behaviour factor), applicable to the whole structure.

Except in the case that the detailing of the structure under consideration is in good compliance with code provisions for new structures, especially as concerns capacity design measures, for existing structures, it is not expected that behaviour factors can attain values similar to those tabulated in codes unless detailing is in compliance with code provisions for new structures (especially provisions related to capacity design measures). It is also intended that nuclear structures keep an additional margin against earthquake as compared to conventional buildings. Consequently, as a very rough estimate, well-designed existing nuclear structures can be attributed values of q of the order of half those given in codes for new structures.

4.1.2.5. Method M5: static nonlinear analysis (pushover)

Description

This method is presented in detail in Section 4.2. It is built on a description of the structural behaviour beyond the conventional limit state, until the failure mode is identified. This behaviour is synthesized in the form of a capacity curve, as presented in Section 3.1.2.

Depending on the specific procedure considered, the dynamic behaviour of the structure is more or less taken into account. All the procedures assume that the earthquake results in some damage to the structure and that a degraded stiffness (as compared to the initial slope of the pushover curve) should consequently be considered. For some procedures, this leads to the identification of a ‘performance point’ at the crossing between the capacity curve and the ADRS, as already mentioned in Section 3.1.3.5. The structure is regarded as acceptable if the ultimate point of the capacity curve is not exceeded.

Comments

As it is intrinsically based on a description of the post-elastic behaviour, this method is beyond considerations relating to load or stresses classification. The most sophisticated versions of the method capture the main features of both the post-elastic behaviour and the dynamic response of the structure. There is good consistency between the method of analysis and the structural capacity assessment. As the capacity curves end at a failure mode of the structure, the method provides a rather good estimate of the available margins in terms of PGA level.

A drawback of the method is that it necessitates a rather accurate description of the post-elastic behaviour, which may be difficult to obtain with reasonable confidence. In particular, the capacity curve should account for the damaging cyclic effects. This means that a ‘monotonic equivalent behaviour’ of the structural elements should be taken into account.

So far, this method was principally used outside the nuclear industry. The question of floor response spectra generation was, therefore, not discussed in detail.

Further developments on the applicability of this method in a nuclear context are presented in Section 4.2.3.

4.1.7.2. Method M6: dynamic (time history) nonlinear analysis

Description

The details of the method may differ. The outline is as follows:

- The structural modelling is based on a best estimate description of the post-elastic behaviour. Depending on the range of nonlinearity to be investigated, the modelling may be more or less sophisticated. For instance, the Japanese approach described in Section 2.2.4 only necessitates a simplified approach, as opposed to an analysis of ruin mechanism that would necessitate sophisticated modelling;
- In order to compute the dynamic response, a series of N input motions is selected so that the average response spectrum envelops the prescribed (site specific) response spectrum. A series on N computations of the response is then carried out;
- The parameters of interest are the strains (differential displacements, inter-storey drifts). For each of them, the mean value is derived and compared to a reference strain value (ultimate or acceptable strain).

Comments

This method generally requires both a considerable number of degrees of freedom and precise constitutive laws. However, attempts are made in the present document to push forward acceptable simplifications.

The outputs generated by this method are known to be sensitive to the selection of the input motion. The reason for that was clearly elicited in the discussions relating to the DBA in Section 3.1.3.5.

Like M5, this method is beyond considerations relating to load or stresses classification. Unlike M5, it requires cyclic constitutive relationships of the constitutive elements and the assessment of margins necessitates a repeated application of the procedure, increasing the intensity of the input motion.

There is no consensus so far on the calibration of the method in the nuclear industry (Which type and how many input motions? Which constitutive relationships and which verification criteria?).

However, the Japanese methodology is based on simplified nonlinear time history analyses and copes with these difficulties (see Section 2.2.4).

4.2. DBAs for buildings

4.2.1. Background of the DBA

4.2.1.1. The past of seismic evaluation: check of compliance with codes for new buildings

In conventional procedures for the detailed seismic evaluation of individual existing RC buildings, evaluation was always done at the member level as a capacity–demand comparison in terms of forces (member seismic internal forces for the demand and member resistances for the capacity). These forces were determined according to code provisions for the seismic design of new buildings (force based approach).

Evaluation of existing structures by checking compliance with a standard for the design of new ones is neither rational nor practical, as it is extremely unlikely that an old structure meets the very stringent requirements of modern codes for structural regularity, local ductility (member detailing) as well as global ductility (control of inelastic response through capacity design), continuity of the load path, etc. In this way, all old structures, with the possible exception of low-rise ones with heavy structural walls, may be found to be inadequate and in need of retrofitting. Moreover, to comply with a current code for new structures, practically every structural element will need to be upgraded to meet all the resistance and detailing requirements of this code, increasing the cost of retrofitting so much, that demolition or the 'do-nothing' alternative will be the most likely outcomes of the evaluation.

It should be pointed out at this point that, although they are well established in engineering practice, current codes for seismic design of new buildings are not transparent and the security margins they provide are neither known nor uniform throughout a given building or among different buildings. In that sense, they do not constitute a rational basis for the seismic evaluation of existing buildings.

4.2.1.2. The evolution towards differentiation of seismic evaluation procedures from seismic design of new buildings

In view of the difficulty of conventional approaches, recent developments are in the direction of adopting performance requirements and criteria for existing buildings, which differ from those implicit in current codes for new buildings. The basis for this pragmatic attitude is not the presumably shorter remaining service life of an existing building[22] but the recognition of the much higher total cost of seismic retrofitting (including the indirect cost of disruption of occupancy) in comparison to new construction.

The differentiation is effected mainly in two ways. First, by explicitly taking into account sources of earthquake resistance and energy dissipation in the existing structure, which are normally neglected in the design of new buildings, such as the positive effects of non-structural elements (e.g. masonry infills) and the redistribution and reduction of seismic demands due to nonlinearities in structural elements and in the foundation. Second, by explicitly allowing certain structural elements to develop large and permanent deformations, provided that their gravity–load bearing capacity is not impaired. The first point requires modelling and analysis at a higher level of sophistication than provided by current codes for the seismic design of new structures. The second implies that poor detailing and insufficient strength in many elements is not a

[22] On such a basis, a building could be evaluated as adequate for a future life of a few years, after the end of which the evaluation might be renewed for another period and so on.

problem, provided that global stability is assured by a few lateral-load-resisting elements, existing or to be implemented. Both points represent a significant change in the philosophy that prevailed in earthquake-resistant design of buildings for the past decades, namely: (1) the use of relatively simple, yet conservative, modelling and analysis, and (2) the requirement for universal proportioning and detailing of members for strength and ductility, regardless of whether this is essential for the global seismic performance. As the new trend becomes established through successful application in evaluation and retrofitting projects, it is starting to affect the codes for earthquake-resistant design of new structures as well. This represents a reversal of past tradition, in which procedures and codification efforts for existing structures followed and emulated those for new ones.

Design or evaluation procedures proposed in recent years as alternative to conventional force based design or evaluation aim at a more direct link between design or evaluation criteria and expected performance, within the framework of performance-based seismic design. This link is best achieved when displacements or derivatives thereof, rather than forces, are used as design variables.

A parallel may be identified between the evolution towards DBAs and the introduction in the past of plastic design methods for steel frames (employing the plastic hinge concept) that provided designs with more homogenous safety and better transparency of the post-elastic behaviour.

4.2.1.3. The rationale behind displacement based seismic design or evaluation

An earthquake does not represent a set of given lateral forces to be resisted by the structure, as normally considered in forced based seismic design or evaluation, but rather a demand for the accommodation of imposed dynamic displacements. It can be said that an earthquake produces the (inelastic) displacements and the structure creates the forces: for a given mass and stiffness of the structure, the inelastic displacements induced by a strong earthquake are roughly equal to those in an elastic structure — according to the well known ‘equal displacement’ rule — and the structure reacts with forces equal to its strength, independently of the details of the earthquake motion. Therefore, deformations and displacements, rather than forces, represent a much more rational basis for the seismic design or evaluation of structures. After all, structures do not collapse due to the earthquake lateral loads per se, but due to gravity loads acting through the lateral displacements caused by the earthquake (P-Δ effects).

In displacement based procedures for the seismic design of new structures or the evaluation of existing ones, seismic displacements (or in general deformations) are the primary response variables for the design or the evaluation. This means that design or acceptance criteria and comparisons of demands to capacity are expressed in terms of displacements (or deformations) instead of forces.

4.2.1.4. Displacement based design versus displacement based evaluation

Since they were first proposed in the early 1990s [80], displacement based concepts have found their way more into seismic evaluation of existing structures than into the design of new ones. For existing structures, the application of displacement based concepts is more straightforward because structural configuration, sizes of members and reinforcement are known and can be used as an input to either simple or advanced analysis procedures. On this basis, member inelastic displacement and deformation demands can be estimated throughout the structure and compared to the corresponding capacities. For new structures, procedures for direct proportioning of RC members on the basis of given deformation demands have not yet been fully developed and accepted. Therefore, in displacement based design, the problem of member proportioning is still reduced (in most procedures proposed so far) to conventional force based proportioning. Therefore, the evaluation of existing structures provides better ground than the design of new ones for the application of deformation- and displacement-based concepts. As a logical extension, a strengthening intervention is easier to design if it is considered as a means of reducing the global seismic displacement demands on the existing members, to a demand below the corresponding deformation capacity. In other words, detailing of old members does not necessarily need to be upgraded, provided the demands imposed on them are not beyond their ultimate deformation capacity and do no impair their resistance to gravity loads.

The state of the art of displacement based design (DBD) for new buildings is reflected in Annex I ('Tentative Guidelines for Performance-Based Seismic Engineering') — Part B ('Force–Displacement Approach') of the 1999 SEAOC Blue Book [81]. Annex I refers to a 'direct' DBD procedure and an 'equal-displacement-based' (EBD) one. The former, proposed and advocated by Priestley and co-workers, uses a substitute elastic structure to relate displacement demands to the effective period at peak response. The EBD procedure uses instead the equal displacement rule to relate peak displacements to the period of the cracked elastic structure.

Displacement based design procedures hold great promise for seismic design codification and practice, especially after feedback from their application to real cases is received and after the code-specified drift limits, global ductility factors and damping values are further refined and rationalized.

4.2.2. Codified DBAs for the seismic evaluation of buildings

4.2.2.1. Overview of guidelines based on DBAs

Recent seismic evaluation and strengthening guidelines for buildings which are clearly and explicitly displacement-based are:

- The European Norm (EN) of Part 3 of EC8 [82] on 'Assessment and Retrofitting of Buildings' published in 2006;
- The guidelines for 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' [83] of the New Zealand National Society for Earthquake Engineering, first drafted in 1996 for the Nea Zealand Building Industry Authority published in 2006;
- The ATC-40 'Seismic Evaluation and Retrofit of Concrete Buildings' [84] which was published in 1996 in the USA by the Applied Technology Council (ATC);
- Also in the USA, the series of documents that were developed for the Federal Emergency Management Agency (FEMA): The '1997 NEHRP23 Guidelines for the Seismic Rehabilitation of Buildings', also known as FEMA 273-274, developed by the ATC [85, 86]. This document evolved into a 'Prestandard for Seismic Rehabilitation', also known as FEMA 356 developed by the ASCE [87].

The main regulatory-type documents for seismic evaluation of existing RC building structures are:

- Eurocode 8, Part 3 in its current close-to-final draft as prEN1998-3 [82];
- The New Zealand 1996 and 2002 draft guidelines;

[23] NEHRP: National Earthquake Hazard Reduction Program.

- The most recent US documents, namely FEMA 273-274 and FEMA 356, along with the 'Handbook for Evaluation of Existing Buildings', also known as FEMA 310 [88]. Differences between ATC-40 and other US documents are also presented there.

The approach in EC8, Part 3 bears strong similarities with that in FEMA 273-274 and FEMA 356:

- It adopts the same four types of possible analysis procedures: linear-elastic static analysis, linear-elastic modal response spectrum analysis, nonlinear static analysis (pushover) and nonlinear dynamic (response time history) analysis;
- It bases estimation of seismic displacement and deformation demands on the equal displacement rule, rather than on the 'composite spectrum' technique of ATC-40;
- It bases compliance criteria for flexure- or shear-controlled elements on experimental data on RC members, condensed in the case of EC8 into convenient equations in terms of the geometric and mechanical characteristics of the member and its reinforcement — instead of the FEMA tabulated values which were eye-ball fitted to data.

The New Zealand procedure has not the generality and completeness of the European and American ones, but is simple, possibly convenient for hand calculations and closer to the experience and intuition of engineers familiar with flexural mechanics approaches for RC members.

As far as concrete buildings are concerned, the scope of all available codified procedures is limited to structural systems consisting of frames and, to a certain extent, slender isolated walls. This limitation is mainly posed by the compliance criteria employed and the guidelines given for modelling and analysis, which only cover prismatic concrete members, such as beams, columns, and slender walls and their joints. Therefore, they are not directly applicable to buildings consisting mainly of large, fairly squat and interconnected shear walls, like in nuclear buildings.

4.2.2.2. Pushover analysis and 'capacity curve'

A main feature of all displacement based seismic evaluation procedures that have been codified so far is that they allow an estimation of deformation demands on the basis of — among other things — a nonlinear static analysis procedure commonly known as pushover analysis. As a matter of fact, due to its appealing simplicity and intuitiveness, and the wide availability of the necessary computer programs, pushover analysis has become the analysis method of choice in the everyday seismic evaluation practice of existing buildings.

Unlike linear-elastic analysis, equivalent static or modal (response spectrum), which has long been the basis for codified seismic design of new structures, and nonlinear dynamic (response time history) analysis, which has been extensively used since the 1970s for research, code-calibration or other special purposes, pushover analysis was not a widely known or used method until the above-mentioned US documents [84–86] emerged in response to the urgent need for practical and cost efficient, yet reliable, seismic evaluation procedures.

Pushover analysis is a nonlinear static analysis carried out under constant gravity loads and monotonically increasing horizontal loads, applied at the location of the masses in the structural model to simulate the inertia forces induced by the earthquake. It produces an estimate of the expected plastic mechanism(s) and of the distribution of damage, as a function of the magnitude of the imposed lateral loads and of the associated horizontal displacements.

A key outcome of pushover analysis is the 'capacity curve', i.e. the relationship between the base shear force F_b and a representative lateral displacement of the structure d_n. Displacement is often taken at a certain node n of the structural model, termed the 'control node'. The control node is usually at the (centre of mass of the) roof level. Although it is physically appealing to express the capacity curve in terms of the base shear force and the roof displacement, a mathematically better choice, that relates very well to the definition of the seismic demand in terms of spectral quantities, is to represent the capacity curve in terms of the lateral force and displacement of an equivalent single degree of freedom (SDOF) system. The equivalent SDOF system, which is essential for the determination of seismic demand, is introduced below.

4.2.2.3. Equivalent SDOF system

The lateral forces F_i applied to masses m_i in the course of a pushover analysis are taken to remain proportional to a certain pattern of horizontal displacements Φ_i, normalized so that at the control node $\Phi_n = 1$:

$$F_i = \alpha m_i \Phi_i$$

The features of an equivalent SDOF system can be determined on the following bases:

- Its stiffness k* is such that k* = Fb/dn;
- If Φ is the shape of an eigenmode of period T (in principle, the mode with the higher participation factor, which is generally the first one), then the period T* of the SDOF system is equal to T. Of course, if Φ is an approximation of the considered eigenshape, T* is an approximate value of T.

This ground leads to determining an equivalent mass m*, an equivalent force F* and an equivalent displacement d*. In case of an elastic-perfectly plastic SDOF system, its description is finalized by determining the equivalent plastic yield d_y*.

It is clear that the pushover analysis and the capacity curve can only capture the expected plastic mechanism(s) and the distribution of damage when the lateral inertial forces (represented by F_i) during the response indeed reasonably follow the postulated pattern of horizontal displacements Φ_i. In this regard, for the sake of simplicity, the codified seismic evaluation procedures mentioned above adopt a heightwise linear pattern of Φ_i, independent of the horizontal coordinate in the direction transverse to that of the considered horizontal seismic action. The validity of this assumption should be carefully examined for every case study.

4.2.2.4. Determination of seismic demands in terms of 'target displacement' of the equivalent SDOF system

Unlike linear-elastic analysis, equivalent static or modal, and nonlinear dynamic (response time history) analyses, both of which readily yield the (maximum) value of the response quantities to a given earthquake (i.e. the seismic demands), pushover analysis per se only yields the capacity curve. The demand has to be estimated separately. This is normally done in terms of the maximum displacement induced by the earthquake either at the control node or to the equivalent SDOF system. The displacement demand on either one of the two is termed 'target displacement'. The pushover analysis has to extend at least up to the point on the capacity curve with a displacement equal to the target displacement. The inelastic deformations and forces in the structure from the pushover analysis at the time the target displacement is attained are taken as the demands due to the particular input motion at the local level.

Two (approximate) procedures have been proposed for estimating the target displacement. The first, adopted in EC8 and in FEMA 273-274 and 356, is based on the equal displacement rule, appropriately modified for short period structures. The second one, adopted in ATC-40 and in the New Zealand draft guidelines, is based on the concept of the 'substitute-structure' method introduced in Shibata and Sozen [89].

Modified equal displacement rule approach for target displacement

In the equal displacement rule approach, the target displacement of the equivalent SDOF system with period T* is determined directly from the 5%-damped elastic acceleration spectrum, $S_e(T)$ at period T*:

$$d_t^* = S_e(T^*)\left[\frac{T^*}{2\pi}\right]^2$$

In the modified equal displacement rule approach, if T* is less than the corner period T_C, between the constant acceleration and the constant pseudovelocity parts of the elastic spectrum, d^*_t is corrected on the basis of the q-μ-T relation proposed by Vidic et al. [90].

Substitute-structure approach for target displacement

In the substitute-structure approach of Shibata and Sozen [89], adopted by ATC-40 and the New Zealand draft guidelines, the target displacement is estimated from the elastic response spectrum entered at a period corresponding to the secant lateral stiffness at peak response[24]:

$$T_{sec}^* = T^*\sqrt{\mu_\delta}$$

In this formula, μ_δ is the ductility demand: $\mu_\delta = d^*_t/d^*_y$. According to Shibata and Sozen, the damping value to be selected is also a function of μ_δ:

$$\xi\,(\%) = 2 + 20\left(1 - 1/\sqrt{\mu_\delta}\right)$$

It implies that the response spectrum should be available for a series of damping values. In the New Zealand guidelines, the elastic response spectrum for $\xi \neq 5\%$ is taken to be equal to the 5% spectrum multiplied by $\sqrt{7/(\xi+2)}$.

4.2.2.5. Graphical presentation of capacity and demand in pushover analysis

In ATC-40, Freeman's 'capacity spectrum method' [91] is employed for the graphical presentation of capacity and demand in pushover analysis. In this approach, the capacity of the structure under lateral force and the seismic demand are both presented through separate force–displacement diagrams, and visually compared by presenting both diagrams on the same plot. In this respect, the term 'capacity spectrum' may be considered as a misnomer, as neither demand nor capacity is plotted as a function of period. The term may have derived from the presentation of seismic demand, as determined on the basis of the substitute-structure approach, in the form of a plot of spectral acceleration versus spectral displacement of an SDOF oscillator (the so called ADRS format).

As, according to the substitute-structure approach, the period of the equivalent SDOF system is determined by the secant stiffness to the point of maximum response, the intersection of the capacity curve derived from the pushover analysis and the ADRS curve determines the seismic demand (provided that the ADRS curve incorporates the reduction of spectral values with

[24] We consider here the elastic-perfectly plastic case. The approach can be easily generalized to a bilinear SDOF system with a non-zero hardening.

damping ratio). This intersection, called the ‘performance point’ in ATC-40 is exemplified in Fig. 29.

The way in which the seismic demand (performance point) is determined has become a matter of considerable controversy. The substitute-structure adopted in ATC-40 has been questioned as lacking a physical basis and as producing inaccurate or biased results, compared to those of nonlinear time history analysis (see e.g. Refs [92–96]). This was the main reason for the adoption of the modified equal displacement rule approach in FEMA 273-274 and 356.

4.2.3. The DBA in the context of nuclear industry structures and buildings

4.2.3.1. Structural features of nuclear industry structures and buildings

The type of structural element most commonly present in nuclear buildings is the RC wall. The walls are rarely isolated. They are present in two principle directions and are connected at each crossing. The walls are also connected to the floors, which are generally made of RC slabs, with or without beams, with or without precast elements.

Numerous openings may be encountered in both walls and floors, thereby preventing a smooth distribution of stresses.

Other types of structures may also exist in nuclear buildings, but less frequently: frames, or columns and beams associated with walls, infilled frames and steel structures. A mixture of the different types can also be observed.

Due to radioprotection, some of these elements, mainly the walls, are very thick (1–2 m). The location of these thick elements is not driven by structural considerations. Therefore, they often induce strong irregularities (both in plane and elevation) of the structure.

In conclusion, the structures of a nuclear building are usually geometrically complex, highly hyperstatic, stiff, strong and irregular, with large holes and dynamically coupled with massive equipment.

4.2.3.2. Structural complexity and stiffness

Most nuclear structures are composed of walls intersecting between each other and with the floors. The high hyperstaticity, together with a generally strong irregularity, both in plane and in elevation, make the distribution of seismic forces among the different bracing systems (walls and floors) complicated in most cases. There are often transfers of forces through the floors and the perpendicular walls at various levels, to follow the variations of stiffness, and there may be strong discontinuities in forces or bending moment diagrams in a wall from the top to the foundation, making the displacement based method much more difficult to apply than in the case of an isolated wall.

This structural complexity can be captured properly with a complete 3-D model, where all the bracing elements are appropriately modelled, for instance with plate or shell elements. However, the use of such a model becomes very problematic when nonlinear behaviour has to be considered, because of the size of such models, and also because the appropriate constitutive relationships are still rather complex.

In the case of isolated walls, a multi-layered beam model may apply, provided that the shear strains are conveniently taken into account. This type of model can appropriately take into account uniaxial nonlinear material behaviours and also the extent of cracking. However, actual nuclear structures are much more complex. They appear as hyperstatic boxed structures for which such models can only be applied with difficulty. It should be envisaged to use local biaxial constitutive laws taking into account the influence of cracking, which may lead to very complex models, very difficult to apply in practical terms in an engineering process. Moreover, the presence of numerous openings increases the difficulty.

Apart from their complexity, it should be pointed out that nuclear structure walls are often long with usually low height:length ratios. This feature, associated to internal hyperstaticity makes nuclear structures generally very stiff. It has to be pointed out that DBAs have not yet been exposed to comparison to time history analyses for such very stiff structures.

4.2.3.3. Dynamic behaviour and higher mode effect

In principle, pushover analysis can be applied in geometrically complex structures, such as those of nuclear buildings, provided that the pattern of horizontal displacements Φ_i, that controls the pattern of imposed horizontal forces is taken to be proportional to the eigenmode with the largest participation factor (or participating mass) in the considered horizontal direction (implying that if this mode is not purely translational, the Φ_i and the lateral forces F_i will have horizontal components transverse to that of the considered seismic action). Nonetheless, such an analysis only captures the effects of a single mode, which may prove insufficient for a complex structure.

The 'modal pushover analysis' procedure proposed by Chopra and Goel [97, 98], for both symmetric and unsymmetric buildings, seems capable of capturing the effects of higher modes and may be appropriate for geometrically complex structures in which more than one mode is important in the considered horizontal direction. Therefore, its application is recommended in order to capture all modes with a total participating mass at least equal to 90% of the actual one of the building.

In modal pushover analysis, a pushover analysis is fully carried out separately for each mode considered of interest. Modal results for local deformations are then combined through the SRSS (square root of sum of squares) or the CQC (complete quadratic combination) rule, also accounting for the modal participation factors. Element forces are determined not by combining modal results through the SRSS or CQC rules, but from the so-computed total local deformations, via the constitutive relation of the nonlinear elements used in the pushover analysis.

However, as this method has only been successfully applied to simple and fairly regular structures, its reliability for nuclear buildings should be checked through comparisons with results of nonlinear time history analyses. Actually, the pushover analysis supposes a given shape of the system of external forces representing the system action. This firstly supposes that this shape fairly represents the participation of the various modes, secondly that the modifications on stiffness which will appear during the yielding of the various elements will not affect this shape. In the complex structures that are in question in nuclear buildings, the differences in stiffness of the various elements (including the floors) may induce large transfers of forces when the yielding occurs. This may result in unforeseeable[25] torsional movements and in magnification of higher mode effects.

4.2.3.4. Soil Structure Interaction (SSI)

The pushover type of analysis has not so far been applied to include SSI effects. There are many questions to be resolved there:

- The force pattern to be applied, which may have to be quite different from the 'uniform' and the 'triangular' force patterns commonly applied to regular buildings. The problem is

[25] Unforeseeable in the initial linear analysis.

that, unlike what happens in structures fixed at the base where nonlinearity develops after an extensive fully elastic range of the response and does not affect modal results, SSI effects are normally nonlinear from the very beginning due to soil nonlinearity, and after a certain point also due to the unilateral nature of the contact that causes uplift, etc. So, modal results including only the linear SSI effects may not be representative of the response, as governed from early on by the nonlinear SSI effects;

- The rules that have been developed for the determination of the target displacement from the response spectrum, elastic or inelastic, may not be appropriate for cases in which SSI effects are significant and govern the response. These rules have been developed from studies of SDOF systems with cyclic force–deformation models that involve considerable hysteresis. Those SSI effects that are governed by the interface between the soil and the foundation, such as uplift or sliding, involve much less hysteretic energy dissipation than the models mentioned above. Therefore, rules for the determination of the target displacement may be unconservative and special ones may have to be developed for cases in which SSI effects are significant. On the other hand, SSI will produce radiation damping, which is also different from the hysteretic damping involved in the SDOF studies mentioned above;
- Even in the case of linear SSI, most of the top displacement of the structure may be due to the rotation of the base of the structure due to soil compliance. This raises additional questions on how to perform the pushover analysis.

In conclusion, significant research may have to be done to establish the procedure by which pushover analysis should be performed to include SSI.

4.2.3.5. Criteria

Although these types of structures are not the most common in nuclear buildings, the basic DBA can be applied to steel or concrete frames, to isolated walls or steel braced structures, with the same limitations that occur for the method in general. It can also be envisaged to apply it to masonry infilled frames.

However, it should not be forgotten that, in principle, acceptance criteria for nuclear structures are more severe than for non-nuclear structures. This means that the ‘nuclear margin’ should be encompassed in the considered input motion or that conventional criteria should be amended before application in the nuclear context.

A specific tool of the nuclear industry is probabilistic risk assessment. In this type of approach, a realistic margin of the seismic capacity of structures should be evaluated to the extent possible. It seems that DBAs are inherently more appropriate for this type of evaluation than more conventional methods. For instance, in order to obtain a realistic evaluation of a given structure capacity, it is reasonable to increase the limit strain in R-bars to a value higher than 1%.

However, as this value also controls the bond failure, a reasonable limitation should apply, for instance 3%, which remains far below the real ultimate strain of that type of steel. For concrete, the usual limit of 0.35% may be increased to take into account the confinement of concrete. This type of construction is similar to that used for usual building.

Regarding acceptance criteria, the first difficulty to be resolved by the engineering community relates to shear walls. A representative deformation should be selected and acceptance criteria should be posed on it. In this regard, rather than local strain, it seems advisable to promote global deformation, for instance measured by the inter-storey drift as in the Japanese approach presented in Section 2.2.4, as representative of the damage in shear walls.

4.3. Other options for the evolution of current nuclear power plant engineering practices

4.3.1. Introduction

Tracks for the development of DBAs were discussed in the previous section. Other possible developments are now considered. The presentation starts with full scope nonlinear time history analysis. Currently, nonlinear time history analysis is not regarded as convenient for engineering purposes.

However, due to the fast evolution of computing capabilities and the development of modelling tools, it is possible that in a medium term perspective it will become a popular tool.

Time history analyses are generally envisaged to only be associated with sophisticated constitutive relationships and other sophisticated modelling items. It has to be pointed out that benefits can also be gained from simplified time history analyses, as is elaborated below. In this spirit, and not only for application to time history analysis, tracks for simplification of analysis are also discussed. The advantages and possibilities of the linearization technique are particularly examined. When appropriate, the corresponding needs for research and development are also presented.

4.3.2. Full scope time history analysis

4.3.2.1. Outlines of full scope time history analysis

A possible ideal analysis for a nuclear building is a fully nonlinear analysis performed in 3-D and in the time domain (response time history analysis), including nonlinear SSI. Such an analysis should be performed under three simultaneous translational components of seismic action. All pieces of equipment or systems thereof that have mass and flexibility sufficiently large that their dynamic response cannot be considered independently and uncoupled from that of the structural system should be included in the dynamic model. For those pieces of equipment which have a relatively small mass, the analysis should provide the time history of the response at the points of support to the structure, to be used as input motion for the evaluation of such equipment (for floor spectra).

Input motion

In general, it will be necessary to take into account only the three translational components of the seismic action. At least three triplets of translational components (or pairs of accelerograms, if the vertical component is disregarded) should be used in the evaluation. The triplets (or pairs) should be selected from recorded events with magnitudes, source distance and rupture mechanisms consistent with those that determine the review level seismic action, if identified. When the required number of triplets (or pairs) of appropriate recorded ground motions is not available, appropriate simulated accelerograms may replace the missing recorded motions. The duration and other features of the simulated accelerograms should be consistent with the magnitude and the other relevant features of the seismic event.

There is no consensus so far on the number of input motions that should be considered, the acceptance criteria on them and the way the analysis outputs should be processed. As it is expected that the nonlinear time history method will become more and more popular, a standardization effort should be engaged on this matter in the near future.

Modelling

The constitutive relationship to be used should be consistent with the discretization of the structure, the expected stresses and strains, and the considered acceptance criteria.

Attention should be given to the connectivity between different types of elements that may be applied to model different regions and materials of the structure, in particular regarding the nodal degrees of freedom: at the interface between a region modelled with 2-D finite elements and one that includes beam/column elements, nodal rotations, which are the most important degrees of freedom in beam/column elements, normally have no stiffness associated with them on the side of the 2-D finite elements. So, unless special modelling is used to connect these different types of elements, the beam/column elements will effectively be pinned at the nodes of the interface.

The cyclic constitutive laws to be used will normally reflect energy dissipation through hysteresis, by taking into account the difference of the response in unloading and reloading from that in monotonic (virgin) loading. Any energy dissipation which is not reflected by the hysteretic laws used for materials and elements should be accounted for through other means, such as viscous damping.

Acceptance criteria

Acceptance criteria (or failure limits) may be expressed in terms of deformation measures employed by the constitutive laws used:

- Local strains or overall shear deformation (drift) for concrete walls;
- Rotations — chord or plastic — for prismatic members or pipes in flexure;
- Axial deformations for bracings;
- Overall shear deformation for infill panels, etc.

The specific limits to be used should be related to acceptable damage levels and will certainly be far from the ultimate deformation (i.e. the one corresponding to substantial reduction in resistance). Therefore, attaining or even slightly exceeding these limits will not put into question the reliability of an analysis that may neglect the slight drop in local resistance implied by these limits.

Available acceptance criteria in civil engineering are founded on an engineering approach of structural element resistance (for instance acceptable shear force for a wall). There is no direct connection between this approach and outputs of nonlinear finite element time history analyses that principally consist of local strains. Consequently, there is no consensus on the way such outputs should be processed in terms of acceptance criteria. An effort should be made in this direction so that nonlinear time history analysis is regarded as an engineering tool. In this regard, it is interesting to observe that in the Japanese approach presented in Section 2.2.4, acceptance criteria are not expressed in terms of local strains, but in terms of a drift between successive floors.

Simplified time history analyses

Section 4.3.3 is dedicated to modelling simplification. It is pointed out here that time history analyses are necessarily associated with sophisticated nonlinear modelling. On the contrary, nonlinear analyses can fruitfully be carried out on simplified modelling. A simple model is not synonymous with a poor model; a simple model may result from an intelligent engineering approach so that this simple model is the most appropriate for modelling the structure and behaviour under consideration. This includes the use of macro-elements and adequate constitutive relationships, such as those discussed in Section 4.3.3.

It might also be advisable to take advantage of an a priori knowledge of nonlinearity oncentrated in pre-identified areas, such as a possible basement uplift.

4.3.2.2. Some specific guidance on modelling

Concrete walls, which are by far the most common type of structural element in nuclear structures, should normally be discretized with 2-D finite elements. This is the approach used by most participants of the benchmark study in the IAEA CRP, although the mesh refinements chosen in most cases are far beyond what would be practically feasible and necessary in a nonlinear time history analysis of a nuclear structure in its entirety. Membrane elements are generally sufficient, to the extent that the out of plane response and behaviour of the walls is of little practical importance.

For the concrete, constitutive relations may be expressed in the instantaneous principal stress directions, regardless of the occurrence of cracking (rotating crack approach). This simple approach normally provides acceptable accuracy. Concrete cracking may be considered as smeared. As concrete walls are rather heavily reinforced, the practical importance of the concrete tensile strength is very limited and can be disregarded. However, the experimentally confirmed dependence of concrete strength (and with it of the concrete stress–strain law in compression) on tensile strains in the orthogonal direction should be accounted for.

Wall reinforcement at both faces of the wall and possibly at intermediate planes, may be modelled as smeared. Heavy concentrations of wall reinforcement (e.g. at the edge of a wall or along the intersection of two walls) may be modelled through truss elements perfectly bound to the concrete finite elements. An elastic-perfectly plastic constitutive law may be used for both types of reinforcement.

Prismatic members (columns, beams, bracings or even pipes) may be modelled as nonlinear beam/column elements or nonlinear truss/strut elements, in general with degrees of freedom only at the ends. Then their constitutive laws will normally relate the element forces and deformations at the end nodes of these elements (e.g. chord rotations or plastic rotations to end moments taking into account the effect of axial force, axial deformations to axial force and possibly to moment, etc.). In concrete beam/column elements, fibre models can be used, or even simple phenomenological multi-linear hysteretic laws (such as the simplified-Takeda type). In bracings, the axial force–deformation relationship should include the post-buckling behaviour under cyclic loading.

Infill masonry panels may be modelled through no-tension diagonal struts with a constitutive relation derived from the effective shear force versus shear deformation relation of the panel. For simplicity, struts with symmetric force–deformation relation in both tension and compression may be used along both diagonals, provided that each one has half of the strength and stiffness properties of the full infill panel.

The nonlinear behaviour of the soil — including its interface with the structure — should in principle be included through finite element modelling of the soil media. The extent of modelling of the soil below and beyond the foundation of the structure depends on the particular conditions. At any rate, appropriate boundary conditions should be provided at the bottom of the soil layer to represent the bedrock or underlying stiffer soil, as well as laterally, to reflect the infinite lateral extent of the layer. Given that, in the ideal situation, the analysis will be in three dimensions, 3-D nonlinear finite elements will have to be used for the soil. Alternatively, recourse may have to be made to an appropriate number of nonlinear soil springs at the interface of the soil and the foundation, representing the nonlinear behaviour both of the soil and of the interface.

4.3.3. Modelling simplifications

4.3.3.1. Macro-element approach

A macro-element is a finite element that reflects, with an appropriate accuracy, the behaviour of a structural element with a very limited number of degrees of freedom (as compared to what it would be if the structural element were regarded as a continuum). Beam elements are more or less refined macro-elements, depending on the underlying theoretical description of a beam (Bernoulli theory, Timoshenko theory, etc.). Examples of popular macro-elements developed over the past decade are fibre elements. The models proposed by the participants to the benchmark are very diverse. Some of them are classical refined finite element models, while others are examples of models using macro-elements suitable for the description of a cantilever wall.

For vertical cantilever walls, such as the CAMUS wall, a simple fibre element could be successfully used, with just a few fibres over the section (either just one at each end of the section, or an additional one for the intermediate vertical reinforcement) and integration stations at the end of nodes. If the nonlinear constitutive relationships of constituent fibres are judiciously chosen (and associated to appropriate assumptions about kinematics and shear deformation), this type of macro-element reproduces the cyclic behaviour of cantilever walls well. It is more representative than stick models, provided the length of each macro-element is limited to a single storey or to the width of the wall, whichever is less.

A macro-element approach can also be envisaged for the modelling of (rather regular) shear wall buildings. In such buildings, a panel can be defined as part of a wall that is comprised between two floors and between two transverse walls. Such panels may be described by their length, height and thickness, and their behaviour can be synthesized in a macro-element approach. For the panels with holes (doors and other penetrations), a catalogue could be set up that provides the mechanical features of such panels as compared to those of the corresponding virgin panels.

The macro-element approach has proved to be very efficient in many circumstances for conventional buildings. Developments that would address the features of nuclear buildings are encouraged; their accuracy and reliability should be verified through comparison of their results to those of detailed and refined finite element models.

4.3.3.2. Separate analysis in the two horizontal directions

In some cases, it may be sufficient to perform an independent nonlinear analysis in each of the two main horizontal directions of the building, neglecting the horizontal component of the seismic action in the transverse direction. In such an analysis, only the walls parallel to the considered horizontal direction will be included in the model, connected with the slabs at floor levels.

Depending on the presence of large openings in the floor slabs and on the in-plane stiffness of these slabs relative to the individual walls, floor slabs may be modelled as rigid diaphragms or as in-plane flexible ones.

Connection with the walls in the transverse horizontal direction is neglected. Nonetheless, the contribution of these walls to the flexural strength, stiffness and response of the walls which are parallel to the considered horizontal direction may be considered by including an appropriate width of them as ribs (stiffeners) or flanges of the walls parallel to the considered horizontal direction.

4.3.3.3. Constitutive relationship

Taking the example of the CAMUS wall, in conjunction with the use of fibre type elements, a rather simple description of the concrete constitutive relationship would be sufficient, including:

- A uniaxial stress–strain law in compression that follows one of the standard stress–strain relations proposed for monotonic loading;
- Unloading with the initial elastic modulus until the zero-stress axis and following the zero-stress axis thereafter up to zero-strain and beyond into the tensile strain region;
- Reloading along the zero-stress axis up to the maximum permanent compressive strain that has been reached before continuing with the initial elastic modulus towards the monotonic loading curve and following that latter curve after the previously attained maximum compressive strain has been reached (this implies that one should keep track of the maximum compressive strain attained before, as a memory variable);
- Dependence of the stress–strain law in compression on the simultaneous stress or strain in the orthogonal direction.

This type of constitutive modelling is much simpler than some constitutive relationships employed in the 2-D finite elements used by participants for the benchmark exercise.

Nonetheless, it is considered as a good balance between simplicity and accuracy for nonlinear time history analyses of a nuclear structure in its entirety.

If pushover analysis is used, instead of time history (dynamic) analysis, then only the monotonic part of the constitutive relations is important. These relations do not need to include hysteretic relationships.

4.3.4. Equivalent linear analysis

4.3.4.1. Lessons learned from geotechnical engineering

In general, soils exhibit nonlinear behaviour for very low strains, much lower than those strains generated by input motions considered in the design of nuclear power plants. Therefore, geotechnical engineers cannot avoid dealing with these nonlinear effects. As early as the 1970s, the geotechnical scientific community set up an equivalent linear method in order to enable geotechnical engineers to cope with this difficulty in a practical manner.

Of course, this approach is not appropriate for dealing with large nonlinear effects; however, it appeared to be a powerful tool for earthquake engineering purposes and became very popular. It is worth remembering the outlines of this approach that can be regarded as an example of a practice that has not stuck to the elastic behaviour assumption and has found a way to evolve so as to take into account small nonlinearity effects.

Basically, the method is an iterative one. For every type of soil, the nonlinear effects are summarized by the shear modulus decrease (the $G–\gamma$ curve) and the damping factor increase (the $\eta–\gamma$ curve) versus the cyclic shear strain. For each layer of the soil profile, the seismically induced shear strain is computed. On this basis, the shear modulus and damping factor of each layer are updated and a new computation of the response is carried out (practical rules were established that account for the fact that the seismic response is not a harmonic one). The procedure converges in a few iterations.

The development of a similar approach for buildings can be envisaged. It means that, for each structural element, a relationship similar to the $G–\gamma$ curve should be available in the form of an equivalent elastic behaviour associated to the seismically induced strain. Associating such a development with the concurrent development of macro-elements seems reasonable.

4.3.4.2. Outlines of equivalent linear analyses

An equivalent linear analysis approach is shown conceptually in Fig. 57. It may be feasible to develop and calibrate an iterative analysis procedure consisting of loops of linear analysis for the earthquake action. This analysis may be linear static, modal response spectrum analysis, or even linear time history analysis. The effects of gravity loads should always be superimposed on those due to the earthquake action. In each loop of analysis, local elastic properties (mainly the elastic modulus) may be revised to be consistent with the level of local stress and strain predicted in the previous loop due to the combined effects of gravity loads and the earthquake action.

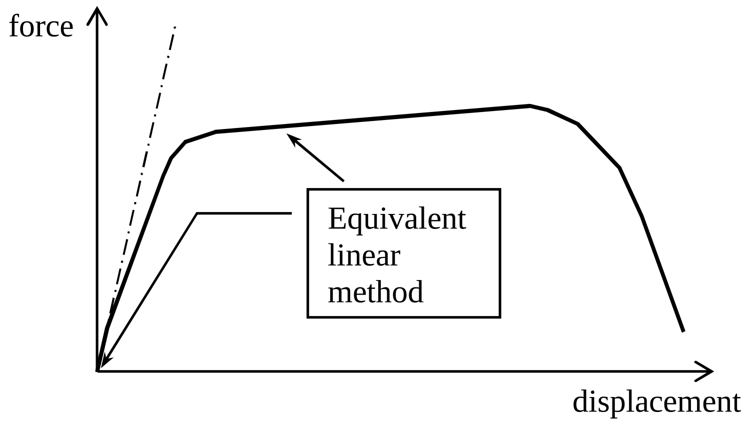


FIG. 57. Domain of application of the equivalent linear method.

The main challenge is to implement such an approach within the framework of computer codes for linear elastic analysis, without building a nonlinear constitutive relation in the code itself. In other words, ideally the approach could be implemented with a module, which is outside the main engine of the computer code.

The purpose of such a module would be to post-process the results of each loop of linear analysis in order to derive revised elastic properties and feed them to the linear analysis computer code as material input data for the next loop of analysis. The same post- and pre-processing module can include convergence criteria at the local and/or global level, to determine when the iterative analysis should stop and the final output be produced.

4.3.4.3. Proposal of a new method

A lesson learned from the benchmark exercise is that the main drawback of classical nuclear power plant engineering practice is the frequent lack of consistency between the modelling of structural elements (non-cracked elastic behaviour) and the associated verification criteria (ULS criteria). Therefore, an improved engineering practice, even a simplified practice such as an equivalent linear analysis, should first resolve this issue. This is the case in the method proposed in this paragraph, which can be considered as an evolution of Method M2, in which not only the stiffness of overloaded elements is updated but potentially the stiffness of any element.

The first step of the proposed method consists of an M1 analysis; thereafter it is iterative:

- An equivalent elastic stiffness (secant stiffness) is derived structural element by structural element, according to the state of strain (and cracking) reached;

- On this basis, the global structural modelling is updated and a new response is computed;
- The updated states of strain (and cracking) are derived in each structural element.

The iterative procedure stops when the state of strain (and cracking) is consistent with the secant stiffness in every element.

This method is intended to operate in the frame of conventional acceptance criteria, in particular a limited yielding of R-bars may be considered (e.g. not exceeding 1%, possibly limited to a lower value). It is expected that for most nuclear buildings, the vast majority of the structural elements will stay at a very low stress level, below the crack yield. For these buildings, the global behaviour should be very close to that estimated by Method M1. In the case of more extended cracking (e.g. the CAMUS mock-up), the method is deemed to account for this effect in a more realistic manner than M1. Depending on the case under consideration, the dynamic response could (or should) be updated. For this purpose, it may be advisable to use a unique model for both the static and dynamic analyses. In this regard, coupling the equivalent linear analysis with macro-element modelling may be suitable.

The main difficulty in implementing the method is the relationship between the state of strain (and cracking) in a given structural element and its equivalent linear stiffness. This point should be carefully examined. In the case of beams, it is possible to base the approach on an estimate of the crack pattern derived from the tensile limit stress of concrete. In the case of shear walls, the directions of cracking should be estimated (depending on whether shear or tensile effects dominate) which is a more complicated issue. In addition, the fact that the cracks are alternatively open and closed should be considered in the estimate of the equivalent linear stiffness. As the scope of the method is limited to those small nonlinear effects that are permitted in the frame of the conventional criteria, it is expected that the difficulties raised could be rather easily resolved, as was previously the case when implementing Method M2.

In the simplest case of a truss element that models a steel component in uniaxial tension (for a bracing or a concentration of reinforcement in a concrete wall, etc.), there is a single elastic property — the elastic modulus. The revised value of that modulus might be taken as equal to the initial and standard value multiplied by the ratio of the stress (or force) at yielding to the peak value predicted from the most recent cycle. This is equivalent to taking an elasto-plastic stress–strain law and using a secant modulus for the maximum strain. Alternatively, a secant modulus for a root mean square (RMS) value or for a fixed fraction of the peak plastic strain (e.g. something between 50 and 75% of the plastic part of the peak strain) might provide improved convergence. The convergence criterion might be the difference between the values of peak strain predicted for that particular element in successive cycles of the analysis. A similar approach is possible for a 2-D membrane finite element that models wall mesh reinforcement in two orthogonal directions, while generalization to beam elements and membrane elements would necessitate deeper investigations.

It is expected that the implementation of this method should contribute to resolving several of the issues raised by the nuclear power plant engineering community and exemplified by the benchmark exercise. It should lead to:

- A better knowledge of the structure behaviour in the range of small nonlinearity;
- A realistic value of the acceptable PGA, whatever the type of seismic input motion;
- More realistic floor response spectra.

Member States are encouraged to implement and test this method. Later on, the experience on its application could be shared so as to eventually make proposals for a detailed description of it.

More generally, the path of linear equivalent methods is considered to be promising and Member States are encouraged to regard it as a matter for research and developments in the coming years.

4.3.5. Floor response spectra generation

Outputs of the benchmark exercise have provided evidence that small nonlinearity may play a crucial role in the computation of floor response spectra. As already mentioned in the conclusions of the benchmark, an evolution of nuclear industry practices in this regard is desirable and feasible. In parallel, research that could be carried out in view of this evolution should address other items relating to floor response spectra generation.

A first item to be discussed is the monotonic impact of the ground motion PGA on the equipment. As the structural response is nonlinear with significant impacts on the frequency content of the floor response, it cannot be excluded that intermediate input motion has a more damaging effect on equipment, and in particular on electrical equipment than the design (or the highest) PGA envisaged in the safety assessment of the nuclear installation.

Another item is related to the direct transfer methods. These methods are regarded as inappropriate because they are founded on the assumption of a linear elastic structural response, while structures exhibit nonlinear behaviour for low level input motions. However, if linearization techniques (or equivalent linear analyses) are possible, the performance of the direct transfer method should be assessed when associated to such linear equivalent models.

5. CONCLUSIONS AND PROPOSALS FOR EVOLUTION OF NUCLEAR POWER PLANT STRUCTURAL ENGINEERING PRACTICE

5.1. Conclusions of the benchmark exercise

5.1.1. On the safety significance of near-field input motions

(1) The root cause of the ‘significant issue’ raised by the low–medium magnitude near-field input motions is not their damaging capacity (there is a consensus that this is very low in spite of their possible high PGAs), but rather the fact that the engineering community used the response spectrum as an indicator of the damaging capacity of these type of input motions. This indicator significantly overestimates the actual damaging capacity of this type of input motion;

(2) The poor capability of this indicator is linked to the fact that seismic input motions are conventionally regarded as force controlled loads (or primary loads in mechanical engineering terminology). However, it is well known that high frequency input motions (with respect to the structure frequency) act principally as displacement controlled loads (or secondary loads in mechanical engineering terminology) [9]. Consequently, this overestimate resulted from ignoring the favourable combination of:

- The high frequency content of this type of input motion;
- The ductile capacity of structures.

It should be noted here that the conventional approach was designed in the 1970s to deal with input motions rich in low frequency (e.g. NRC spectrum) and act as force controlled loads.

The conventional approach was not designed to address those high frequency input motions that appeared later and act as displacement controlled loads; it simply does not work with this type of input motion for which it was not designed;

(3) This high frequency content was first identified as generated by NFEs, but the NFE origin is not really pertinent to this analysis. What is pertinent is the frequency content, irrespective of the so called far- or near-field origin;

(4) The conventional nuclear approach should be amended, at least, when dealing with this type of input motion, especially when evaluating existing facilities, to avoid unduly overestimating their damaging capacity, such as illustrated by the CRP outputs;

(5) It is expected that a reasonable evolution of nuclear power plant engineering practices (also desirable for other reasons) will eliminate this artificial issue.

5.1.2. On alternative engineering approaches to the response spectrum method

5.1.2.1. DBAs

(6) DBAs have proved their effectiveness using the simple case of the CAMUS wall. They lead to a reasonable estimate of acceptable PGAs for different types of input motions. However, a research and development effort is necessary for the resolution of their current limitations, principally relating to:

- The complexity of nuclear structures;
- The predominant shear effect;
- The lack of methodology to account for SSI.

(7) Apart from these limitations, another drawback of DBAs is that, as well as the conventional response spectrum method, they are inherently not appropriate for floor response spectrum generation;

(8) These methods were developed for (low frequency) conventional buildings. The applicability to stiff structures such as nuclear structures needs to be benchmarked against time history analyses;

(9) Recent developments of DBAs might provide an appropriate scientific background (linearization concept) for developing simplified nonlinear models appropriate for nuclear power plant structure modelling (B. Iwan and C. Comartin presentations at the Ispra RCM).

5.1.2.2. Nonlinear time history analyses

(10) A major conclusion on this subject, which is practically the opposite of what was expected when the CRP was launched is that at least, using the simple case of the CAMUS experiment, dispersion of the time history outputs was not greater than dispersion of response spectrum method outputs, which reveals a real maturity of nonlinear time history analyses;

(11) Time history analysis appears to be the most robust method for estimating displacements, accelerations, forces and moments. This method is also the most robust for estimating the acceptable PGA (a PGA value that leads the structure to the conventional limit state) associated with a given spectral shape;

(12) Properly implemented time history analysis is the only method for reasonably computing realistic floor response spectra.

5.1.3. On challenges to nuclear power plant structural engineering practice

5.1.3.1. Lack of consistency of the conventional nuclear power plant structural engineering approach

(13) There is a lack of consistency in the classical nuclear power plant engineering approach due to the concurrent following practices and/or requirements:

- Structural responses are calculated on an (equivalent) linear behaviour assumption;
- Acceptance criteria stipulate that forces and moments should not exceed those corresponding to the conventional limit state.

However, the second statement does not imply that the structural response is actually linear. On the contrary, significant nonlinear effects appear for low PGAs (significantly lower than those corresponding to the conventional limit state or leading to a plastic yield in R-bars, as illustrated in Fig. 58). Therefore, any concrete structure, even if designed according to nuclear standards, should be recognized as exhibiting nonlinear behaviour under seismic input motion;

(14) To a certain extent, the RC response is similar to the soil response; that is, small nonlinear effects appear for low level input motions. Geotechnical engineers and scientists have developed engineering practices (linearization) that account for this phenomenon and are currently used in the nuclear context. This is not yet the case for engineers dealing with concrete structures.

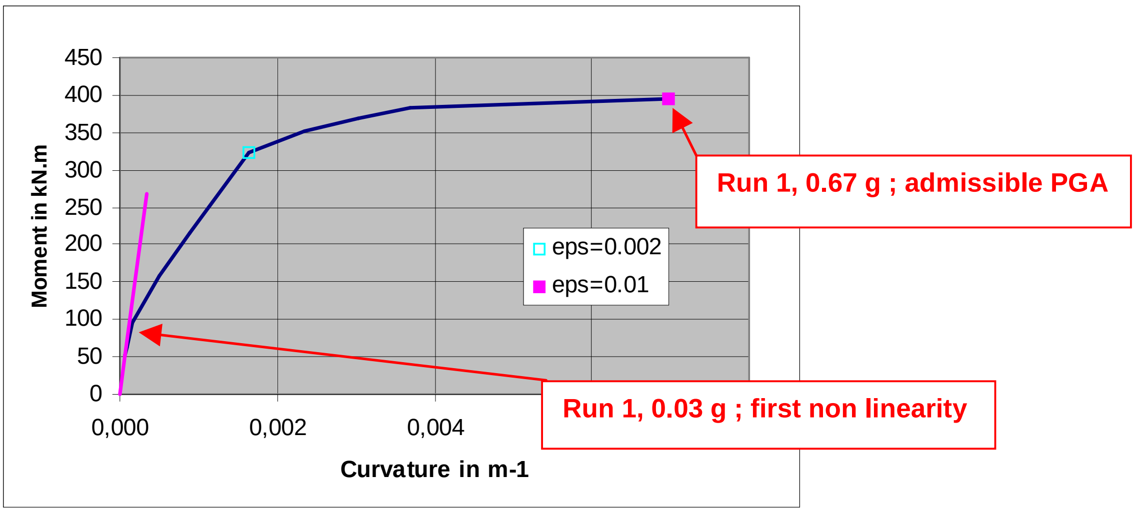


FIG. 58. Nonlinearity effects appear at very low level input motions; example of CAMUS run 1.

5.1.3.2. Floor response spectra generation issue

(15) Reasonably realistic floor response spectra cannot be computed without accounting for small nonlinearity effects. Depending on the circumstances, neglecting these effects may lead either to undue margins or to a lack of margins in the generated floor response spectra. Therefore, an evolution of nuclear power plant engineering practice is highly desirable in this regard.

5.2. Proposals for the evolution of nuclear power plant structural engineering practice

5.2.1. Generic proposals

5.2.1.1. Necessary realistic modelling of dynamic behaviour

(1) As a conclusion of the CRP, in order to adequately calculate dynamic response, all aspects of input and models need to be capable of representing phenomena observed or expected (nonlinear behaviour of structures, boundary conditions, including nonlinear geometric effects such as uplift). This is especially important when analysing an individual event;

(2) Acceptance criteria of structures and components should allow inelastic deformations compatible with the required performance and corresponding performance criteria. These criteria could be based on a probabilistic approach;

(3) It is recommended that the nuclear industry pursue its evolution towards dynamic modelling techniques that match structural behaviour. In particular, it is recommended that small nonlinearities be considered in the models: the computed response, strains and stresses should be consistent with the model. In this regard, the state of practice has significantly evolved in the last decade. The USA [15], France [16] and other countries require consideration of nonlinear behaviour, such as that induced by a cracked section for concrete structures. Approximate rules are provided if detailed stiffness evaluations are not performed;

(4) It is, however, strongly recommended that the first step of dynamic structural analysis is a linear analysis. Linear analysis could be equivalent linear, modelling small nonlinearities in an approximate way. Linear analysis should include calculation of eigenfrequencies and mode shapes. Nonlinear time history analyses could be a second step (e.g. current Japanese practice);

(5) To the extent possible, models should remain as simple as possible. The CRP outputs give evidence that sophisticated models do not guarantee a better quality of those analysis outputs that are of interest for structural designers;

(6) In application of these principles, it is recommended that the nuclear industry work towards a more systematic and codified use of simple nonlinear structural analysis, such as linearization techniques, both for design of new facilities and evaluation of existing ones. This evolution should be supported by appropriate research and development efforts (see Section 5.2.2).

5.2.1.2. Design versus post-event analyses

(7) The benchmark exercise and many other cases demonstrate difficulty in matching the calculated and measured response of structures. Even for a simple structure, uncertainties exist in the evaluation of eigenfrequencies and the structural response. Sensitivity studies should be carried out in order to better understand the effect or impact of uncertainties in input motions and model parameters on the structural response;

(8) Design procedures should continue to take into account uncertainties in ground motion, SSI, and SSC dynamic response characteristics. Seismic response analyses for design are intended to be conservative but not upper bound. Additional conservatisms are introduced in SSC allowable capacities (code, HCLPF or failure). This practice should continue.

5.2.2. Accompanying R&D effort

5.2.2.1. Linearization

(9) It is strongly recommended that linearization techniques be explored in the field of structural analysis. This research and development effort should at least address the following items:

- Categorizing structural elements (fewer than 10 categories are recommended);
- Developing a force–displacement relationship (or the equivalent of the G–□ curves in geotechnical engineering) for each category26;
- Possibly, developing super elements that represent portions of a building (e.g. one super element per level), and deriving super element properties;
- Developing an equivalent strain that controls the equivalent stiffness of the structural (super) elements under consideration;
- Developing a procedure that determines the equivalent linear strain-dependent properties (stiffness and damping).

5.2.2.2. DBAs

(10) An appropriate ‘nuclear DBA’ should be developed to address deficiencies in the applicability of existing DBAs to nuclear power plant structures, particularly addressing the following items:

- Structures with frequencies higher than those of conventional buildings (greater than 2 Hz versus less than 2 Hz);
- The complexity of nuclear structures (multiple modes contributing to response);
- The predominant shear deformation effect for nuclear power plant structures versus the predominant bending moment effect for conventional structures;
- Non-negligible SSI effects for nuclear power plant buildings versus negligible for conventional buildings.

(11) Existing DBA results should be benchmarked against nonlinear time history analysis on structures representative of nuclear power plant buildings. If developed as suggested above, the ‘nuclear DBA’ should also be validated against nonlinear time history analysis.

5.2.2.3. Time history methods

(12) As it is expected that time history analysis will play an increasing role in the future, normative documents should be developed that provide a formal framework for this type of analysis, with the objective of achieving a consensus in the nuclear industry.

5.2.2.4. Experimental R&D

(13) It is expected that the experimental effort based on a specimen test on shaking tables is going to be pursued. In order to optimize the benefit of such experimentation for nuclear

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[26] Recent developments of DBAs might provide an appropriate scientific background for deriving simplified nonlinear models: ‘linearization’ concept (B. Iwan and C. Comartin presentations at the Ispra RCM).

power plant engineering, it is recommended that the following facets of the experimentation programme be carefully examined:

- Design of the specimen: A candidate mock-up should be designed, including detailing the design, so that it is representative of nuclear power plant design (shear walls, thicker and more strongly reinforced than for conventional buildings)[27], in particular torsional effect should be represented in a reasonably realistic manner;
- Conduct: The experimental programme should be dedicated to small nonlinearity effects and their impact on the floor response spectra generation issue. It should start with very low level input motions so that the elastic features are documented. The first runs should be regarded as artificial ageing. A (short) series of specimens should be envisaged in order to check the effect of cumulative damage;
- Input motions: The CAMUS feedback of experience draws attention to the fact that controlling the input motion on the shaking table is a key point. To the extent possible, very high frequency input motion, such as recently recorded, should be considered in the first runs of the experimental programme;
- Monitoring: Following the CAMUS example, displacements should be directly measured and global deformations should be monitored, including to the extent possible global shear deformation between two floors. The possible variability of response spectra on a given floor should be examined;
- Accompanying analyses: An accompanying analysis effort should be developed, focusing on linearization techniques and their use in nonlinear time history analysis. The performance of existing DBAs on shear wall structures should be examined, as a first step towards a 'nuclear DBA'.

5.2.3. Specific recommendation on strong motion scaling factor

(14) It is expected that in the future more and more high frequency input motions will be recorded, resulting in higher and higher PGA values, meaningless in terms of input motion damaging capacity. The scientific community has already identified more relevant damaging capacity indicators. It is, therefore, strongly recommended that:

- A more relevant, and simple, indicator be selected and adopted by the structural engineering community as a scaling factor of recorded strong motions, such as PGV or CAV;
- A significant research and development effort be carried out to concur on engineering practices incorporating this new scaling factor (e.g. Design PGV).

[27] Some difficulties might be encountered in conjunction with the scaling rules, shaking table capacities and other constraints.

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ABBREVIATIONS

BOF	Benchmark Output Format
COV	coefficient of variation
DBA	displacement based approach
ISRS	in-structure response spectrum
NFE	near-field earthquake
RC	reinforced concrete
SSE	safe shutdown earthquake
SW	shear wall

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