SEISMIC ANALYSIS
OF LIQUID METAL
FAST BREEDER REACTORS

REPORT PREPARED BY
R.J. GIBERT, CEA, FRANCE,
AND
A. MARTELLI, ENEA, ITALY

A TECHNICAL DOCUMENT ISSUED BY THE
INTERNATIONAL ATOMIC ENERGY AGENCY, VIENNA, 1989
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FOREWORD

In the last two decades the awareness of the potential hazard to the nuclear plants from earthquakes, has dramatically increased. This was accompanied by a development of earthquake engineering methods directed at nuclear industry and particularly at PWRs.

Most of the early nuclear plants were not specifically designed against earthquakes, although subsequent post-analysis showed a significant seismic capabilities. This was also the case for the prototype fast reactors. The seismic loading become a design requirement for the second generation of large demonstration plants designed in 1970's. The seismic methodology prevalent at the time was often found to be inadequate to cope with some of the fast reactor design features and therefore a development of new methods was initiated.

The understanding of the physical phenomena, validation of the numerical methods and the design implications have now reached a degree of maturity. This promoted the IAEA to organize a specialists' meeting on Aseismic Design of LMFBR's held in Bologna in 1987.

Following this meeting, IAEA commissioned this technical document with the aim to provide state of the art report on the subject. The document covers present knowledge and experience accumulated in all countries involved in fast reactor design.

The report is written in two parts. The theoretical background to the seismic methods and their application were written by Dr. R.J. Gibert of CEA, France, and the design considerations and the experimental validation were written by Dr. A. Martelli of ENEA, Italy. The document was edited by Dr. M. Dostal of NNC, UK.

It is hoped that this document will be of some assistance to design companies and R & D organizations in all countries at different stages of development of fast breeders technology.
EDITORIAL NOTE

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1. INTRODUCTION

The design of LMFBR reactor-block and other components might be greatly affected by design and safety requirements for the fast reactor plant to withstand earthquakes. In fact, the presence of rather flexible structures and non-negligible clearances, which are necessary to minimize the effects of thermal conditions, may lead to considerable amplifications of the seismic motion of structures and components. Furthermore, the use of thin wall shells could cause buckling of the large vessels.

In some countries, in which large earthquakes are expected, restraints have been applied to the main vessel and/or the core, to make these structures as stiff as possible, and thus to avoid large seismic amplifications. In other countries and especially where earthquakes are less important, flexible structures and unrestrained vessels and cores have been adopted.

A detailed seismic analysis of vessels and reactor-block internals appeared necessary to the LMFBR designers of most countries: such analysis required a development of new specialized numerical techniques. In parallel, advanced and expensive experiments including on-site testing, were considered necessary to validate the numerical methods.

The role of seismic isolation concept is becoming increasingly important in all countries involved in fast reactor design. Its influence extends to all aspects of aseismic design from specification of ground motion, methodologies, component design and qualification to licensing. If the development and application of the seismic isolation is successful, it will relegate the importance of earthquake loadings in fast reactor design to a secondary effect.
2. SCOPE

This report is a general survey of the recent methods to predict the seismic structural behaviour of LMFBRs. It shall put into evidence the impact of seismic analysis on the design of the different structures of the reactor.

The report is addressed to specialists and institutions of governmental organizations in industrialized and developing countries responsible for the design and operation of LMFBRs. The information presented should enable specialists in the R & D institutions and industries likely to be involved, to establish the correct course of the design and operation of LMFBRs. Also, the safety aspect of seismic risk are emphasized in the report.

Note: This work takes into consideration the outlines of two Specialists' Meetings: "On-site Experimental Verification of the Seismic Behaviour of Nuclear Reactor Structures and Components" and "LMFBR Reactor Block Antiseismic Design and Verification" (both in Bologna, Italy, 1987).
3. GENERAL METHODS USED IN SEISMIC ANALYSIS

3.1 Introduction - free field motion definition

The seismic analysis of LMFBR are conducted according to general methods which are described here. We shall not discuss the regulatory principles which have to be used to define the seismic free field motion associated with a given site. Simply this seismic motion data consists of a set of oscillator response spectra (ORS) the definition of which is the following:

\[
\frac{\varepsilon}{\omega = 2\pi f} \quad \text{Ground acceleration } \gamma(t)
\]

Fig. 3-1
1 DOF Oscillator Scheme

Given a 1-DOF oscillator of resonance frequency \( f \) and damping ratio \( \varepsilon \), its motion \( x(t) \) relatively to the ground displacement is obtained by resolving the equation of motion:

\[
\ddot{x} + 2\omega \dot{x} + \omega^2 x = -\gamma(t) \tag{3-1}
\]

The maximum of \( x(t) \) during the seismic excitation is a point of the response spectra \( S(\omega, \varepsilon) \) which are plotted versus \( f = \omega / 2\pi \) for different values of \( \varepsilon \).

Response spectra can also be plotted in pseudo-velocity \( S_Y(\omega, \varepsilon) = \omega^2 S(\omega, \varepsilon) \) ordinates. It is noted that the response \( S(\omega, \varepsilon) \) in comparison to a real \( \gamma(t) \) signal is a very fluctuating function.
The regulatory response spectra are much simpler curves which have the meaning of envelopes or mean values as shown in fig. 3-2.

In probabilistic terms we shall say that the seismic motion is a random process which is characterized by certain mean values. Regulatory ORS are such mean values.

3.2 Structural response calculation

The classical principle of seismic analysis consists of calculating the maximum values of the response in term of displacements, strains, stresses, forces, etc. from the ORS data and from the dynamic characteristics of a structure. From such results a classical static loading design analysis is performed using specific codes and standards.
Sometimes the transient aspect of seismic loading may be taken into account by particular corrections as we shall see later. However this type of analysis implicitly assumes that the structural mode of failure, which has to be avoided is a monotonic one and not an oligocyclic one.

Let us now describe the main ways to calculate the maximum structural response.

3.2.1 Modal method

This method can be applied only if a linear behaviour of the structure can be assumed. The first $N$ natural modes $X_n$ are calculated with fixed conditions at the level of imposed seismic motion connections (P) by means of finite element computer codes. The dynamic equations of the structure $[x(t) = \text{relative to ground motion}]$ are

$$M\ddot{x} + C\dot{x} + Kx = -MU\gamma(t)$$

where $x = 0$ at (P). \hspace{1cm} (3-2)

and $U$ is the unit translation vector in the seismic direction.

Equation (3-2) projected on $X_n$ ($\vec{r}$) basis leads to a set of modal oscillator equations:

$$\ddot{a}_n + 2\epsilon_n \omega_n \dot{a}_n + \omega_n^2 a_n = -q_n/m_n \gamma(t) \forall n (1,N)$$

where $m_n = (X_n, MX_n$) = generalized mass
$\omega_n$ = natural frequencies
$\epsilon_n$ = modal damping factor
$q_n = (X_n, MU) = \text{participation factor}$

The maximum values of $a_n(t)$ are directly obtained from ORS:

$$(a_n)_{\text{max}} = q_n/m_n \omega_n^2 S_\gamma(\omega_n, \epsilon_n)$$ \hspace{1cm} (3-4)

The next step is to derive the maximum value of the response at a point $\vec{r}$ of the structure ($x(\vec{r},t)$). The classical method for that assumes a good statistical independence of $a_n(t)$ functions and that the statistical laws governing the maximum value don't significantly differ.
when a 1 DOF system or a multi DOF system is considered. Then \( x_{\text{max}} \) could be obtained by using the "quadratic combination" rule:

\[
X_{\text{max}}(t) = \sum_{n} X_{n}^{2}(t)(a_{n})^{2}
\]

(3-5)

The equation 3-5 is to be applied for each DOF. It is important to precise the truncated modal basis effect. In fact to correctly apply (3-5), modal basis \( X_{n} \) must be truncated according to:

\[
\omega_{n} \in [\omega_{0}, \omega_{c}] \text{ for } n \in [1, N]
\]

Where \( \omega_{c} \) being a cut-off frequency beyond which no significant seismic energy is found (\( \omega_{c} \approx 20 \text{ to } 30 \text{ Hz} \)).

In practice, neglected modes are taken into account in (3-5) by adding a pseudo-mode deduced from a static calculation of the structure and the accurate formula for \( x_{\text{max}} \) is then:

\[
x_{\text{max}}^{2} = \sum_{n=1}^{N} \left( \frac{q_{n}}{m_{n} \omega_{n}^{2}} S_{y}(\omega_{n}, \varepsilon_{n}) \frac{X_{n}}{X_{n}} \right)^{2} + \gamma_{\text{max}}^{2} \left[ x_{s} - \sum_{n=1}^{N} \frac{q_{n}}{m_{n} \omega_{n} X_{n}} \right]^{2}
\]

(3-6)

where \( \gamma_{\text{max}} \) is the maximum of ground acceleration and \( x_{s} \) is the static response of the structure given by: \( Kx_{s} = -MU \) with \( x_{s} = 0 \) at (P)

**Note**: the ground motion has 3 components which may be independently taken into account. Therefore, if the seismic analysis for each component leads to \( (x_{\text{max}})^{x}, (x_{\text{max}})^{y}, \) and \( (x_{\text{max}})^{z} \) we have:

\[
x_{\text{max}}^{2} = (x_{\text{max}})^{2} + (x_{\text{max}})^{2} + (x_{\text{max}})^{2}
\]

(3-7)

3.2.2 Time history analysis

In the case of complex structures with many closed modes in the seismic frequency range, or in the case of a structural non-linear behaviour, it may be necessary to directly integrate equations (3-2) time step by time step. This method encounters two difficulties:
- \( \gamma(t) \) accelerograms must be derived from the regulatory ORS data. Several technics are available to generate a synthetic accelerogram the mean ORS of which fits a given ORS with a given precision (see fig. 3-3).

- \( \gamma(t) \) and the corresponding response \( x(t) \) and its maximum \( x_{\text{max}} \) are in fact realisations of random processes. The interesting result is the mean value of \( x_{\text{max}} \) and the RMS value of its fluctuations. To derive this information a lot of time integrations are theoretically needed to attain a good statistical precision. In reality one or few calculations are performed.

(Source : ref. 5)

Fig. 3-3: Determination of Synthetic Accelerograms Fitted on a Set Regulatory Spectra
3.2.3 Improved methods - stochastic seismic analysis

As shown, time history analysis could be expensive if a good statistical precision is required. Therefore in the case of linear structures, modal method is preferred although it has several drawbacks:

- The quadratic combination law is not valid for close mode configurations.

- The knowledge of the maxima of response is not always sufficient. For example, in a current power plant seismic analysis, reactor building response is first calculated. Then from these results component responses are determined. For that, the knowledge of the ORS associated with the motion of their connection points on the building (floor spectra) is needed.

Improvements and a generalisation of the method for floor spectra calculation have been proposed. They are based on a more explicit probabilistic description of seismic signals.

3.2.3.1 Improvement of the quadratic combination law

When two modes have very close resonance frequencies, modal contributions \( a_i(t) \) and \( a_j(t) \) cannot be assumed independent from each other. Several authors, e.g. ROSENBLUETH, suggested to take into account this statistical dependence by considering cross terms in formula (3-5). These terms can be characterized by the following parameter:

\[
\alpha_{n,m}^2 = \frac{\omega_n - \omega_m}{\frac{\eta_n}{\omega_n} + \frac{\eta_m}{\omega_m}}
\]  \hspace{1cm} (3-8)

where \( \eta_n \) is an equivalent damping ratio including the transitory aspect of seismic excitation \((\omega, T)\). \( \eta_n \) can be expressed as:

\[
\eta_n = \epsilon_n \frac{1 - e^{-2\epsilon\omega_n T_n}}{1 - e^{-\epsilon\omega_n T_n}^2}
\]  \hspace{1cm} (3-9)
Finally the maximum of the response is given by ROSENBLUETH formula:

\[
x_{\text{max}}^2 = \sum_{n} C_n^2 x_n^2 + \sum_{n \neq m} C_n C_m \frac{(x_n^\text{max})(x_m^\text{max})}{1 + \alpha_{nm}^2}
\]  

(3-10)

where \((x_n^\text{max}) = S_{r}(\omega_n, \epsilon_n)\) is obtained from O.S.R. and \(C_n\) is a modal participation factor.

\[
C_n = \frac{q_n}{\omega_n^2} x_n \text{ (for displacement variables)}
\]

Taking for example a 2 DOF oscillator, we can verified that (3-10) works well if both resonance frequencies are close each other: \(\alpha_{12} \ll 1\) (see fig. 3-4 where the ratio between the exact value \(x_{\text{max}}^\text{exact}\) obtained from numerical simulations and (3-10) estimation \(x_{\text{max}}^R\) is plotted versus \(\alpha_{12}\)).

(Source : ref. 4)

Fig. 3-4 : Error made by using ROSENBLUETH formula for a bimodal oscillator.
But if \( \alpha_{12} \) is > 1 we observe a systematic underestimation which is relatively independent from \( \alpha_{12} \) and increases when \( \epsilon \) decreases. In fact (3-10) is obtained from the variance formula for a linear combination of random signals

\[
\sigma^2 = \sum_{n} C_{n}^2 \sigma_n^2 + \sum_{n \neq m} C_{n} C_{m} \frac{\sigma_n \sigma_m}{1 + \alpha_{nm}^2}
\]

and is extended to mean maximum. This extension is acceptable only in the case where the peak factor \( \mu = \frac{x_{\text{max}}}{\sigma} \) of the combined signal has the same order of magnitude as the peak factors of each component \( \mu_n = \frac{x_{\text{max}}}{\sigma_n} \).

For a 2-DOF oscillator with separated natural frequencies (\( \alpha_{12} > 1 \)) this is not verified (see ref. 4, 12, 13 and 15) and the response can be underestimated for lightly damped systems by 20%. Some authors proposed to introduce a correction term in (3-10) which is always positive and only works if \( \alpha_{nm} > 1 \). Then (3-10) becomes:

\[
x_{\text{max}}^2 = \sum_{n} C_{n}^2 x_{\text{max}}^2 + \sum_{n \neq m} C_{n} C_{m} \left( \frac{x_{\text{max}}}{\sigma_n} \right) \left( \frac{x_{\text{max}}}{\sigma_m} \right) \frac{1}{1 + \alpha_{nm}^2} H_{nm}
\]

where \( H_{nm} \) is theoretically a function of \( \epsilon_n \), \( \epsilon_m \), of the numbers of half-cycles \( N_n \) and \( N_m \) of both oscillators during \( T \), and finally of the ratio \( \lambda_{nm} = \frac{C_n x_{\text{max}}}{C_m x_{\text{max}}} \), which represents the relative level of each resonance in the response spectrum. For example an acceptable correction is made using for \( H \) the expression (if \( \epsilon \approx \epsilon_j \approx \epsilon \)):

\[
H(\epsilon, \lambda, \nu) = H_1(\lambda) H_2(\epsilon, \nu)
\]

where \( \nu = N_n / N_m \)

Function \( H_1 \) and \( H_2 \) are given in a graphical form in figure 3-5.
Note that for oscillations having very different resonance frequencies e.g. a seismically isolated building with a low isolation frequency \( f_1 \) and structural resonance frequency \( f_2 \gg f_1 \), \( H \) tends to 1.

![Graph](source: ref. 4)

Fig. 3-5: Abacus for the Determination of the Pic Factor Effect Correction

Therefore \( x_{\text{max}}^2 \approx \left( \frac{C_1}{x_{\text{1}}_{\text{max}}} + \frac{C_2}{x_{\text{2}}_{\text{max}}} \right)^2 \)

(in the time interval corresponding to the maximum reached by the low frequency oscillation, a large number of cycles is performed by the other).
3.2.3.2 Direct calculation of floor spectra

Floor spectra can be obtained directly without performing a time history calculation. In this approach the seismic excitation is simulated by a "separable" random process:

$$\gamma(t) = a(t) F(t)$$  \hspace{1cm} (3-13)

where $F(t)$ is a stationary random process characterized by its Power Spectral Density (PSD) $S_f(f)$, and $a(t)$ which is a deterministic envelope representing the transient aspect of seismic excitation.

a) The first step of the method is to obtain $S_f(\omega)$ and $a(t)$ from ORS data. The $S_f(\omega)$ is calculated from the pseudo-velocity ORS at zero damping ratio $S_{PV}(\omega, 0)$

$$S_{PV}^2(\omega, 0) = \frac{1}{2} \int_0^1 A^2(u) \, du$$  \hspace{1cm} (3-14)

where a standard form $A(u)$, for example a half sinus, is chosen for the envelope. Another relationship between seismic duration $T$ and $S_f(\omega)$ is needed. Several possibilities are available using the other ORS and specially their evolution with damping ratio which is characteristic of $T$.

b) The second step corresponds to the calculation of spectral characteristics of response. It can be demonstrated that the PSD of the response at a point $r$ of the structure, for a time $t_o$ after the beginning of excitation is given by:

$$S_R(r, \omega, t_o) = |H_a(r, \omega, t_o)|^2 S_f(\omega)$$

with $H_a(r, \omega, t_o) = \int_0^t G(r, \tau) a(t_o - \tau) e^{-i\omega\tau} \, d\tau$ \hspace{1cm} (3-15)

Where $G(r, \tau)$ is the Green function of the structure which can be represented by a sum of modal contributions. Then $S_R$ is given by a quadratic combination of modal contributions which can be resolved without any difficulties. From $S_R(\vec{r}, \omega, t_o)$ a mean spectrum $S_R(\vec{r}, \omega)$ and an envelope $a_R(\vec{r}, t_o)$ can be deduced. For example $a_R$
$(\vec{r}, t_o)$ is identified with the RMS value $\sigma_R (\vec{r}, t_o)$ obtained by integrating $S_R (\vec{r}, \omega, t_o)$ in frequency domain. A suitable value for $S_R (\vec{r}, \omega)$ is $S_R (\vec{r}, \omega, t_{max})$, where $t_{max}$ is the time when $\sigma_R (\vec{r}, t_o)$ is maximum.

Fig. 3–6: comparison between time history (solid line) and direct method (dashed line) for computing floor-spectra.
c) The last step of the method is an application of the formula of step a) to derive the floor ORS at $T$ from $S_R(\omega)$ and $a_R(\tau, t_0)$. In order to illustrate the validity of this method, we consider 10 time history realisations of a seismic motion adjusted on the regulatory guide ORS (see fig. 3.3).

The fig. 3-6 shown a comparison of the floor ORS for a multi-degree of freedom structure obtained by:

- average of the 10 spectra calculated by a time integration of the equations (solid line),
- direct method described above (dashed line).

3.3 The different steps of a seismic analysis

The seismic analysis of a complex system such as LMFBR plant, cannot be conducted in a single calculation. Several steps are needed:

- the first step consists of determining the global motion of the foundation which in general is not the same as the free-field motion.

- the second step is a calculation of reactor building in order to verify its behaviour and also to determine the floor motions.

- the third step is the analysis of the main equipments, as reactor block, steam generators, etc.

- the fourth step consists of calculating the response of the secondary equipments (e.g. secondary piping, reactor block components i.e. as pumps and IHX) using the results of the preceding step.

The above procedure is based on a solution of three types of analytical problems:

soil - structure interaction
system - subsystem interaction
multi-supported structure calculation

In the following the assumption and associated methods currently used are examined.
3.3.1 Soil-foundation interaction

In the description of the seismic analysis methods in paragraph 3.2, we implicitly assumed that the free-field soil motion is imposed to the building at its foundation level. This hypothesis is accurate only if soil is very stiff i.e. hard rocks. Generally the presence of foundation locally modifies soil motion.

In a first instance it must be decided where to apply the free field motion. In the case of a large installation, a phase evolution and correlation lost along the foundation should be considered. The second problem is to know how soil modifies the response characteristics of the structure.

3.3.1.1 General formulation

To answer these two questions, we must return to the problem of soil and building with its foundation considered as a medium excited by the real localized seismic source (S).

As soil has a linear behaviour (excepted near source and foundation), we can define a virtual surface (Σ) which limits the soil medium, in a vicinity of the plant. Then the problem can be solved in 2 steps:

- 1st step : the response \( x \) on (Σ) of the far soil medium to (S) and to a given force field \( f \) acting on (Σ) is:
  \[
  x = Af + C \tag{3-16}
  \]
  where \( A \) is a boundary impedance operator which can be solved by using for example boundary element computer codes (problem a) and \( C \) can be obtained
by assuming that the force field \( f_1 \) and the displacement field \( x_1 \) on \((\Sigma)\) of the free-field problem (fig. 3-9) are connected by (3-16) relation. Therefore:

\[
C = x_1 - Af_1 \tag{3-17}
\]

where \( x_1 \) and \( f_1 \) can be derived from the free-field surface ground motion which is the standard seismic data (problem \( a_1 \)).

\[
X = A[f - f_1] + x_1 \tag{3-18}
\]

This procedure is rather complicated. In practice the following simplifications are usually made:

3.3.1.2 "Shaking table" method

Often seismic motion is assumed to be vertically transmitted from an horizontal bed-rock. Therefore problem \( a_1 \) is mono-dimensional : \( x_1 \) and \( f_1 \) can be easily deduced from free surface motion \( x_S \). This method is used in FLUSH Computer Code (ref. 22). It works if phase variation may be neglected on the foundation surface.
3.3.1.3 "Soil impedance" method

Moreover, if soil behaviour is linear near foundation, it can be shown that step a, a₁ and b lead to the calculation of the "kinematic interaction" $x_2$ which represents the motion of foundation relatively to the bed-rock motion $x_1$, if building mass is not taken into account.

\[
\begin{align*}
M(x_1 + x_2) & = + \\
M_{\text{inst.}}(\ddot{x}_1 + \ddot{x}_2)
\end{align*}
\]

Fig. 3-11: Soil Impedance Method

Note that $x_1 + x_2 = x_s$ for superficial foundations.

Then the response $x_3$ of the building ("dynamic interaction") relatively to $x_1 + x_2$ can be obtained by solving a classical structural problem according to 3-2 methods:

Building and its foundation are connected to a rigid base by a set of springs $k_i$ and dampers $c_i$ which represent soil effects in the low frequency domain. If foundation is rigid $k_i$ and $c_i$ are the coefficients corresponding to translation and rotation motions. The system is excited by the inertial force field $-M(x_1 + x_2)$, $M$ being the mass operator of building.
Example of impedance coefficients for an homogeneous soil and a circular foundation (radius R):

If \( \lambda = \omega R/c_s \) (\( c_s \) is shear wave velocity and \( \omega \) is frequency)

\[
k = k_{\text{stat}} (k + i\lambda c)
\]

where 
- \( k_v = 4 GR/(1-v) \) is vertical stiffness 
- \( k_h = 8 GR/(2-v) \) is horizontal stiffness 
- \( k_r = 8 GR^3/(1-v) \) is rotation stiffness 
- \( k_{rh} = GR^2/(1-2v) \) is coupled horizontal translation-rotation stiffness, \( G \) is shear modulus and \( v \) is poisson coefficient).

The plot of \( k \) and \( c \) versus \( \lambda \) are given by fig. 3-13.

(Source ref. 2)

Fig. 3-13 : dimensionless "in phase and out-of phase by 90°" coefficients for a rigid circular foundation on a semi-infinite homogeneous soil, versus dimensionless frequency \( \lambda \).

3.3.2 System-subsystem interaction

We saw that the seismic analysis of a complex assembly of structures was divided into several steps. For a given step, the modelled structure is excited by imposing the motion of the support structure, calculated in the preceding step, at the connection points.
To take into account the existence of the supported structure, its mass can be added in the support structure modelisation. This method is approximate and can lead to non-negligible errors, if a resonance of the supported structure is close to a resonance of the support structure. A quantitative criterium can be deduced from the analysis of 2-DOF system (fig. 3-14). The three characteristic parameters are:

- the mass ratio $\lambda = \frac{m}{M}$
- the damping coefficient $\varepsilon$
- the relative difference of resonance frequencies:

$$\alpha = 2 \left(\frac{f_2 - f_1}{f_2 + f_1}\right) \ll 1$$

Fig. 3-14: 2 DDL Oscillator Scheme

- If $\lambda = \alpha$ or $\varepsilon$, supported structure can be neglected in support structure modelisation.

- If $\lambda \gg \alpha$ and $\varepsilon$, supported structure strongly modifies the connection motion and therefore has to be include into support structure modelisation. An important overestimation of supported structure motion is made if this rule is not applied.

3.3.3 Multisupported structure calculation

Some structures as piping systems are fixed at several levels on the main components or building floors which typically have different motions. In this case it is not possible to define the seismic excitation only by an inertial force in the relative displacement equation. In fact absolute displacement $x(t)$ must be devided into 2 terms:

- one describes the effect of the differential displacements imposed at connection points. It can be expressed by using the matrix $U$ of the static solutions corresponding to an imposed unit motion at each connection
- the second describes the inertial effects. Therefore:

$$x(t) = UX_L(t) + X_R(t)$$ (3-19)

$x_R(t)$ verifies:

$$\ddot{X}_R + C\dot{X}_R + KX_R = -MUX_L$$ (3-20)
where $x_L(t)$ are the seismic imposed motions at the connection and $K$ is the stiffness matrix of the structure fixed at each connection. Equations (3-19) and (3-20) can be easily solved if a time-history approach has been chosen for the calculation of the support structure.

If the modal method has been used, the knowledge of the floor spectra of each connection point (according to paragraph 3.2.3.2) is not sufficient. In fact informations about the correlation between each connection motion is needed. Theoretically this could be obtained by an extrapolation of the method of paragraph 3.2.3.2. Practically, we use the "envelope spectrum" method:

- the maximum of $x_R(t)$ is classically calculated by imposing the same motion at each connection characterised by the envelope of all the floor spectra.

- the quasi-static part $x_s$ is estimated by a static calculation where the maxima of differential displacements of connections are imposed to the structure. $x_R^{\text{max}}$ and $x_s$ are added to obtain the maximum of the response. It can be noted that this method is not conservative in all cases.

3.3.4 Comments on the non-linear aspects of seismic analysis

The methods described in last paragraphs are based on a linear structural behaviour. However seismic analysis often needs a non-linear approach:

- to avoid an over-conservative design (in the case of ductile structures for example),

- to design safety margins (in all cases)

Theoretically, the modal method cannot be applied and time-history analysis is the only correct approach, although to expensive. Simpler approximate methods are proposed. They are generally based on the concept of "equivalent linear system", the characteristics of which are determined from time history calculations or directly using stochastic considerations. We shall see specific applications of such techniques in
the chapter devoted to LMFBR core analysis. Let us describe here a simplified method which is often used for ductile concrete or metallic building analysis.

"Inelastic spectra" method:

Let us consider an elastic-plastic 1 DOF oscillator, the dynamic equation of which is:

\[ M\ddot{x} + R(x) = M\gamma(t) \]  

(3-21)

\[ R(x) \] being the non-linear stiffness force shown at fig. 3-15.

\[ \begin{array}{c}
\text{Re} \\
\text{Rupture}
\end{array} \]

\[ \begin{array}{c}
\text{e} \\
\text{x_o}
\end{array} \]

\[ \begin{array}{c}
\text{x_e} \\
\text{e}
\end{array} \]

Fig. 3-15: Non Linear Stiffness Law

The oscillator is excited by a set of seismic acceleration \( \gamma(t) \) with increasing levels \( \gamma_{\text{max}} \). Given \( \Gamma_{\text{max}} \) is the level for which the critical relative displacement \( x_{c} \) is reached. Let us do the same analysis but using now the linear oscillator:

\[ M\ddot{x} + \frac{Re}{x_e} x = M\gamma(t) \]  

(3-22)

and the classical method which consists of applying statically the maximum of load calculated by solving (3-22). Therefore the critical level \( \Gamma_{e_{\text{max}}} \) is obtained if the relative displacement \( x_{e} \) is reached. Fig. 3-16 a and b show the evolution of \( R = \Gamma_{\text{max}} / (\Gamma_{e_{\text{max}}}) \) versus the resonance frequency of the oscillator.

\[ f = \frac{1}{2\pi} \sqrt{\frac{Re}{Mx_e}} \]
Two types of seismic signals are used (Taft and San Francisco) and different ductility factors \( \mu = \frac{x_c}{x_e} \) are tested. Fig. 3-16 shows that \( R \) tends to \( \mu \) in low frequency domain and to 1 in high frequency domain. In the intermediate frequency domain which corresponds to the maximum seismic energy zone, \( 1 < R < \mu \). \( S_y(f, \epsilon) / R(f) \) represents the "inelastic spectra", which can be used in a classical linear analysis to take into account the ductility effects.

It is noted that \( R(f) \) can be roughly estimated by \( \sqrt{2 \mu - 1} \) in the intermediate frequency zone and then the inelastic spectra can be produced by using a method illustrated by fig. 3-17.

Fig. 3-16 : Evolution of the ratio \( R \) between Inelastic and Elastic limit seismic levels versus frequency for different values of ductility factor \( \mu \) and for two different ground motions.
In the case of multi-DOF structures inelastic spectra method must be carefully used. In fact it is not theoretically justified and can lead to non-conservative estimations.

(Source: ref. 2)

Fig. 3-17: inelastic response spectrum construction
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4. SPECIFIC METHODS USED FOR LMFBR SEISMIC ANALYSIS

The following design features typical of LMFBRs necessitates specific methods of seismic analysis:

- The importance of the fluid-structure interaction phenomena concerning the reactor block especially for the pool-type design concept.

- The importance of shock phenomena between the core fuel assemblies under horizontal seismic excitation.

- Thin wall secondary sodium piping.

- Vertical sodium pump with sodium fed hydrostatic bearings.

- Long and flexible control rod shutdown systems.

4.1 Reactor block Structures

As we have seen in the Technical Report devoted to Flow Induced Vibration (ref. 1, Chapter III), the LMFBR reactor block consists of many structures of complex geometry coupled by fluid system (see Figure 4-1). In the above report we have defined the physical principles of fluid-structure interaction and described the numerical methods which are used i.e. Finite Element - Boundary Element. The methodology is not repeated here and the reader not familiar with these aspects of structural dynamics, is kindly referred to this technical report. In the following chapters the main issues of the seismic analysis of such fluid coupled systems is examined.

4.1.1 Axisymmetric fluid-structure interaction

The basic difference between a seismic and a flow induced vibration modelling comes from the fact that only first circumferential (azimuthal) modes \((n = 0 \text{ and } n = 1)\) are excited by seismic motion. We have seen that perturbated flows excite preferentially the lowest resonance frequency modes with high \(n\) Fourier components and generally correspond to out-of-phase motion between two adjacent shells separated by a fluid. Often such modes are
relatively localized: a typical feature corresponds to the motion of two concentric shells in the upper part of the reactor block. It can be assumed that shells are clamped on the core support structures (see fig. 4-1) and that hot and cold collector sodium volumes have a small influence compared to the fluid sheet effect.

The "seismic" modes are rather different consisting of higher frequency modes (2 to 8 Hz compared with 0.1 to 0.5 Hz). Also the seismic motion excites the global modes which need a complete modelisation of
reactor block structures including concrete vault and roof slab (fig. 4-1). All the sodium volumes have to be represented. This aspect complicates the seismic modelisation in comparison with the fluid induced vibration, although seismic calculation only needs $n = 1$ modes.

Fig. 4-2 shows an example of the mesh of a Superphénix-1 reactor block and fig. 4-3 gives a table of the first horizontal seismic modal characteristics (frequencies, generalized and modal masses). It is noted that the low frequency modes corresponding to an out-of-phase motion of adjacent shells have a large generalized mass (associated with the sheet effect) but a low modal mass (which characterizes the seismic excitation projected on the mode). On the contrary, the higher frequency modes corresponding to a global motion of the structures are much more excited by the seismic motion (see both modal shapes on fig.4-4). In order to illustrate the specific difficulties of seismic modelisation let us examin the following problem:

![Fig. 4-2: Mesh](image)
<table>
<thead>
<tr>
<th>Ni</th>
<th>Frequency</th>
<th>Generalized mass (T)</th>
<th>Participation factor</th>
<th>Seismic mass (T)</th>
<th>Mode description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.57</td>
<td>33418.80</td>
<td>-1149.98</td>
<td>39.60</td>
<td>Out of phase</td>
</tr>
<tr>
<td>2</td>
<td>0.82</td>
<td>4373.32</td>
<td>49.78</td>
<td>0.60</td>
<td>Shell modes</td>
</tr>
<tr>
<td>3</td>
<td>1.90</td>
<td>83.40</td>
<td>113.91</td>
<td>155.60</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>2.10</td>
<td>85.12</td>
<td>122.43</td>
<td>170.10</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>2.27</td>
<td>394.79</td>
<td>282.14</td>
<td>201.10</td>
<td>Core modes</td>
</tr>
<tr>
<td>6</td>
<td>2.30</td>
<td>191.16</td>
<td>120.13</td>
<td>75.50</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>2.59</td>
<td>82.26</td>
<td>114.53</td>
<td>159.50</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>3.04</td>
<td>9652.62</td>
<td>-7.69</td>
<td>0.01</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>3.16</td>
<td>5006.90</td>
<td>691.69</td>
<td>95.56</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>3.85</td>
<td>1830.28</td>
<td>850.15</td>
<td>394.90</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>4.48</td>
<td>2356.54</td>
<td>2584.48</td>
<td>2834.47</td>
<td>Global pendulum</td>
</tr>
<tr>
<td>12</td>
<td>5.13</td>
<td>2388.56</td>
<td>2799.18</td>
<td>3280.40</td>
<td>modes</td>
</tr>
<tr>
<td>13</td>
<td>5.39</td>
<td>3700.51</td>
<td>-1585.03</td>
<td>679.00</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>6.80</td>
<td>532.88</td>
<td>-226.52</td>
<td>96.30</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>7.13</td>
<td>77.72</td>
<td>169.62</td>
<td>370.20</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>7.21</td>
<td>36.39</td>
<td>104.87</td>
<td>302.20</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>7.27</td>
<td>38.73</td>
<td>-197.21</td>
<td>1004.23</td>
<td>Global internal</td>
</tr>
<tr>
<td>18</td>
<td>7.97</td>
<td>2438.92</td>
<td>1573.82</td>
<td>1015.60</td>
<td>+ reactor vault</td>
</tr>
<tr>
<td>19</td>
<td>8.39</td>
<td>1754.21</td>
<td>-2068.45</td>
<td>2439.00</td>
<td>modes</td>
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<tr>
<td>20</td>
<td>8.96</td>
<td>897.21</td>
<td>397.47</td>
<td>171.10</td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>9.21</td>
<td>622.93</td>
<td>75.01</td>
<td>9.03</td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>9.71</td>
<td>419.57</td>
<td>60.02</td>
<td>8.60</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 4-3: Table of the axisymmetric natural modes of pool type LMFBR internals
The shell system of a pool-type LMFBR has a complex geometry but a global modelisation is required for the seismic analysis. Therefore, some simplification is necessary. For example, two short shells separated by a fluid sheet could be represented by only one equivalent shell as shows fig. 4-5. The obvious assumption is to consider that the motions of both shells are in phase, because the out of phase motion is not excited by seismic sources. This leads to an equivalent stiffness equal to the sum of the stiffnesses of each shell and an equivalent mass equal to the sum of the shells masses and mass of the fluid sheet.
Fig. 4-5: Definition of an equivalent shell to represent a system of two short shells separated by a fluid sheet. (mass \( m \) and stiffness \( M_0^2 \) represent the whole fluid shell system near one of its resonance frequency \( f_0 = \omega_0/2\pi \)).

This assumption is generally acceptable, but it can be shown that, if there is a coincidence between the frequency \( f_0 \) of a seismic global mode of the structure and the frequency \( f_1 \) of a local mode of the short shells including an out of phase motion, the simplification cannot predict the important fluctuations of pressure arising in the fluid sheet. This pressure field can induce a buckling failure of the shell as shall be discussed later.

This can be expressed quantitatively \( f_1 - f_0 / f_0 < \varepsilon \) where \( \varepsilon = \) equivalent damping ratio

\[
R = \frac{p_{\text{max exact}}}{p_{\text{max simplified}}} = \frac{\mu h}{\varepsilon}
\]

\( \mu = \text{equivalent mass of fluid sheet} \)
\( \text{generalized mass of global mode} \)  \( (4-1) \)

\( h = \text{characteristic height of global structure} \)
\( \text{fluid sheet height} \)

In practice the numerical calculation is not accurate enough to conclude whether the resonance frequencies are coincidental or not. For pool type LMFBR internals, the chance of coincidental frequencies is not negligible due to the large number of seismic modes.
For example a modelisation which does not include the thermal baffle, (see fig. 4-6) leads to an underestimate by a factor of 3, of the maximum pressure acting on the upper part of the shells and hence to an underestimate of the buckling loads.

Fig. 4-6: Detailed View of the Upper Part of the Baffles

4.1.2 3-dimensional effects

The axisymmetric analysis provides an insight into the basic behaviour of reactor block internals. However, the real geometry is not exactly axisymmetric. This is due to the existence of:

- components as pumps and heat exchangers
- fluid communications between collector
- geometrical imperfections of the shells

These non-axisymmetric effects have been studied concerning the F.I.V problems (IAEA report ref. 1 chapter III). Especially we have shown the effect on the modal characteristics. The considerations developed in ref. 1 can be applied to the seismic analysis. A particular
aspect of the 3-D effect is that \( n^0 \) or \( o \) modes can be directly excited by the seismic motion. Let us recall some aspects of the 3-D modelisation.

4.1.2.1 **Description using a substructuration technique**

The whole structure is divided into independent structures which are defined by natural modes corresponding to free or fixed conditions at their connection nodes. The motion of the complete assembly is projected on this modal basis. The neglected high order modes are important for the behaviour of the structure near connections. This effect is taken into account by using a set of link solutions (ref. 1). The system formed by modal and link equations, the variables of which are modal contributions and link variables, is solved by classical method. The seismic modelisation is more complex than the FIV one. Especially the fluid of the hot collector must be correctly modelized (in the FIV model an added mass correction was sufficient).

Fig. 4-7 shows a meridian view of pool-type internals of Superphénix reactor block. Solid lines correspond to the axisymmetric substructure \( n^0 \) 1 (shells and fluid sheets), dashed lines correspond to substructures 2 to 4 (2 IHX and 1 pump including upstands and fluid sheets, a 90% sector is represented as shown in fig. 4-8). Fig. 4-9 shows the assembled system and fig. 4-10 shows the mesh of the hot collector sodium volume.

The seismic analysis of such a system is rather expensive, because of the great number of modes (160 in the range 0.10 Hz). A direct time integration method can be chosen but the necessity of having sufficient statistics on the results also leads to large calculations.
1 - Main vessel
2 - Heat exchanger
3 - Crossings
4 - Core
5 - Roof slab
6 - Primary pump
7 - Crossings
8 - Toroidal redan shroud
9 - Conical redan shroud
10 - Baffle Bl
11 - Safety vessel
12 - Concrete hot ring

(Source : ref. 9)

Fig. 4-7 - Super Phénix internal structures

19°-71° heat exchangers
45° pump

Fig. 4-8 : Super Phénix geometry : plans of symmetry.
Fig. 4-9: General view of redans, shrouds, crossings and components (quarter of structure) = mesh

(Source: ref. 9)
Fig. 4-10: hot collector sodium volume
(quarter of structure) = mesh

Fig. 4-11 a: Shape of the natural mode at 4.3 Hz:
meridian section by the heat exchanger located at 19:

Fig. 4-11 b: Shape of the natural mode at 5.0 Hz:
meridian section by the primary pump.

Fig. 4-11 two meridian view of 2 modes.
If the modal method is chosen, a good modal recombination can be done by the use of Rosenblueth formula. Fig. 4-12 show typical differences between 2-D and 3-D calculations on the maxima of the fluctuating pressure gap acting of certain shells. Note that this gap is directly responsible for the buckling risk which will be studied in Section 4.1.4.

(Source : ref. 9)

Fig. 4-12 a : Pressure differences on the toroidal redan due to the horizontal earthquake : evolution with the azimut (0 < θ < 90°)

Fig. 4-12 b : Comparison between 2D and 3D calculation : maxima of pressure differences on the weir shell dues to the horizontal earthquake (maximum of the 2D calculation is normalized to 1)

4.1.2.2 Fluid communication effect

In the report devoted to F.I.V we have noted the existence of flow passages which connect the different fluid volumes of the reactor block (see fig. 4-13). They are generally composed by holes which are too small to be represented by finite elements. They modify the fluctuating pressure field on the shell walls of both connected fluid volumes. This effect can be simply represented by an impedance \( I = \Delta p/q \) (with \( \Delta p \) = fluctuating pressure gap at the ends of the hole, \( q \) = mass flow rate through the hole).
It is deduced from the local resolution of Laplace equation which can be made independently of the global calculation of the fluid structure system. This resolution provides diagrams which allow a passage equivalent length $l$ ($l = \frac{\omega l}{s}$, $s =$ passage cross-section) to be determined as a function of the different geometrical parameters of holes. Then $l$ can be used to define the characteristics of a special finite element which is introduced in the global mesh of the system to model a connection between both fluid volumes.

Fig. 4-15 shows an example of $l$ diagram for an annular hole. It is noted, that for a set of small axisymmetrically distributed holes an equivalent annular can be defined. The presence of fluid passages modifies the fluid-structure interaction and therefore natural frequencies and mode shapes of fluid-structure system.
Fig. 4-15: Equivalent length $l$ compared with the size $e$ of the annular hole obtained in $n = 1$ mode for an annular cavity.

Fig. 4-16 a: Maximum of pressure differences on the Bl baffle and the weir due to a horizontal earthquake

Fig. 4-16 b: Maximum of radial displacements ratio on the Bl baffle and the weir due to a horizontal earthquake
Finally, the motion of shells due to seismic excitation and especially the fluctuating pressure fields can be strongly perturbated as shown in fig. 4-16, in the case of an horizontal excitation. Fig. 4-16 shows the variations of the fluctuating pressure difference acting on the Bl and weir baffles and shows the effect of introducing flow passages for estimating the buckling loads.

4.1.2.3 Shell geometrical imperfections

In the report devoted to FIV we described in some details how natural frequencies and mode shapes were modified by shell imperfections. Especially a \( \cos n_0 \theta \) shape defect can induce coupled modes \( n = 0, n = n_0 \). Such modes are excited by the \( n = 0 \) vertical seismic motion. In the same way for an horizontal excitation coupled modes \( n = 1, n = n_0 + 1 \) are excited. This phenomenon strongly modifies seismic fluctuating pressure field and shell motions. Taking into account shell defects is fundamental for seismic buckling analysis. We shall make a detailed presentation of it in the section 4.1.4.

In a seismic analysis sloshing effects must be taken into account, however, two difficulties arise:

a) In fact, in the traditional seismic range (1 to 10 Hz); sloshing is located near free surface and does not influence fluid-structure modes, for which a calculation using a \( p = 0 \) boundary condition on (\( \xi_1 \)) is sufficient. In fact it is recommended that this condition is used, because if condition (4-2) is used, many unimportant modes will influence the calculation.

In low frequency domain (0.1 to 0.5 Hz) pressure fluctuations due to sloshing may be important if seismic energy is high enough in this range. Therefore a special study has to be made: in practice only the first sloshing mode is calculated. Unfortunately classical regulatory seismic spectra are not very clearly defined in this frequency zone and may be unrealistic. As will be discussed in section 4.1.5 on seismic isolation devices, a more realistic description of seismic phenomena is required in the low frequency range. For example, using the classical regulatory spectra a height of about 0.5 m is obtained for free surface waves (corresponding to a pressure of \( 4 \times 10^3 \) Pa), for a large pool type LMFBR.
b) The waves are high enough to eventually create impacts on the roof slab. Experiments show that impact pressure is much higher than what can be predicted by a linear theory. Moreover, an impact wave propagates in fluid and may induce non negligible effects on shells. Fig. 4-18 shows the relationship between impact velocity of fluid and impact pressure (the linear theory predicts $p_{lin} = \rho g / \omega U$). The ratio $p_{measured}/p_{lin}$ could give an estimation of the amplification effect due to impact. However only a non-linear analysis using explicit time integration can give a correct level of the impact pressure.

4.1.3. Sloshing effects

Sloshing effects have also been studied in FIV technical report. The importance of sloshing on vibrational behaviour of shell-fluid sheet systems has been shown. For the small displacements, the linear hypothesis may be applied. It leads to the following boundary condition on the mean free surface ($\xi_1$) for the fluctuating pressure $p$:

$$\frac{\partial p}{\partial z} = -\frac{p}{g} \quad (4-2)$$

($g$ = gravitational acceleration)

We saw, that the first sloshing modes observed in LMFBR fluid cavities are located in a very low frequency range, where the flexibility of shells may be neglected. In the simple case of a cylindrical cavity (height $H$, radius $R$), the $n = 1$ sloshing frequencies are given by:

$$\omega_{1m} = \sqrt{\frac{k}{R} a_{1m} \text{th} \frac{a_{1m} H}{R}} \quad (4-3)$$

($a_{1m} = n^m$ zero of $J'_1(x)$)
The associated mode shapes are:

\[ P_{nm}(r, \theta, z) = J_1 \left( \frac{a_m r}{R} \right) \left\{ \cos \theta \right\} \left( \frac{a_m z}{R} \right) \text{ch} \left( \frac{a_m z}{R} \right) \]  

(4-4)

For the classical dimensions of a pool type LMFBR fluid volume \((H = 10m, R = 10m)\) we obtain \(f_{11} = 0.1\) Hz.

Fig. 4-18: Experimental Correlation Between Impact Velocity and Impact Pressure on a Wall During Sloshing

4.1.4 Buckling analysis

4.1.4.1 Background

The thin internal shells are submitted to permanent differential pressure due to the gravity effect of the fluid. This loading can induce a buckling failure. Moreover during an earthquake the motion of the structures can induce a fluctuating pressure field in the fluid. This
pressure field can also induce buckling. An accurate analysis of these effects is not practical. It would be complicated and have to take into account the following characteristics:

- 3D fluid-structure interaction including the effect of shell imperfections,
- Non-linear behaviour: geometrical and material,
- Seismic analysis (it is noted that only a time history analysis could be used with the above features)

In practice simplified methods are used based on experimental results.

The physical aspects of this problem are illustrated by the "ring model" which is a simple structure having a linear behaviour. Let us consider a thin ring connected to a 'fluid sheet'. If the ring is perfect and if loads and displacements are small, the modal equations are uncoupled:

\[ u_n(t) \cos n \theta \]  
\[ \nu_n(t) \sin n \theta \]

(radial)  
(azimuthal)

If we take into account geometrical non-linearities and defects, modal equations are coupled. For example let us express the radial defects as \( \delta_0 \cos n \theta \), the stiffness terms of modes \( n = 0 \) and \( n_0 \) \( (n = 0 \) represent seismic vertical modes; the results of the ring model can be extended to \( n = 1 \) horizontal seismic modes). Neglecting mode \( 2n_0 \) terms, we have for the radial displacements:

\[
\begin{pmatrix}
2\pi Eh/D & k_{0n} (u_{n0}) \\
\end{pmatrix} \times \begin{pmatrix}
u_{n0} \\
\end{pmatrix} = \begin{pmatrix}
u_{n0} \\
\end{pmatrix}
\]

(4-5)

with:

\[ k_{0n} (u_{n0}) = (n_0 - 1/n_0) \frac{2}{D} (\delta_0 + u_{n0}) \]

\[ k_n (u_{n0}) = (n_0^2 - 1)^2 \frac{h^2}{3D^2} + 3 (n_0 - 1/n_0)^4 \frac{\delta_0 + u_{n0}}{D^2} \]

\((h = \text{ring thickness}, D = \text{ring diameter})\)
Note that these terms are obtained from the linear terms written in FIV report (paragraph 4.2.3), replacing \( \delta_0 \) by \( \delta_0 + u_n \).

As discussed in this report, the mass matrix also includes coupled and non-linear terms due to "fluid sheet defects" caused by shell defects. Before concluding how the ring model can explain the complex behaviour observed in experimental studies, let us discuss the most important terms of (4-5) which describe the different physical aspects of the buckling problem.

4.1.4.2 Classical static buckling

Let us assume no defect and static loading \( n = o \) then

\[
 k_n u_n = \left(\frac{n}{n_o} - \frac{1}{n_o}\right)^2 \frac{2}{D} u_n u_{n_o} 
\]

represents the classical Euler buckling term of mode \( n_o \) equation (stiffness is negative if \( P_{ext} - P_{int} > 0 \Rightarrow u_o < 0 \)). The resonance frequency \( f \) of each mode decreases with loading. The mode for which the critical loading (\( f = 0 \)) is the lowest is often called the buckling mode. For more complex structures, the behaviour can sensibly change, from one case to another:

For example, hemispherical shells have high order modes which are more sensitive to \( n = o \) loading than low order modes. Therefore if load is increased, high order mode resonance frequencies are reduced and could become lower than the frequencies of the low order modes. Thus buckling appears on high order modes and also the time characteristics of the instability induced by a little overflow of the critical value, is very small (much smaller than seismic time characteristics). For this reason, seismic and static buckling have the same critical value.

However plasticity plays an important role, as shown in the Table below.

For cylindrical shells buckling modes are not very different from the lowest resonance frequency modes (without loading), \( n = 10 \) to \( 12 \) for a typical LMFBR internal shell. The time characteristic of the instability has the same order of magnitude as the seismic loading and therefore a static buckling analysis cannot be applied. The problem is therefore more complicated than for hemispherical shells although the material behaviour remains quasi-elastic.
### Shell n° 1 2

<table>
<thead>
<tr>
<th></th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Elastic buckling</strong></td>
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<td></td>
</tr>
<tr>
<td>pressure</td>
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<td>0.74 MPa</td>
</tr>
<tr>
<td><strong>Plastic buckling</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>pressure</td>
<td>0.45 MPa</td>
<td>0.47 MPa</td>
</tr>
<tr>
<td>Plastic buckling</td>
<td></td>
<td></td>
</tr>
<tr>
<td>pressure with defect</td>
<td>0.30 MPa</td>
<td>0.21 MPa</td>
</tr>
<tr>
<td>(δ/h = 0.3)*</td>
<td>(δ/h = 0.5)*</td>
<td></td>
</tr>
<tr>
<td><strong>Experimental</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>buckling</td>
<td>0.33 MPa</td>
<td>0.22 MPa</td>
</tr>
<tr>
<td>pressure</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* δ/h = ratio defect/thickness

### 4.1.4.3 Importance of defects

Shell defects have a significant influence on critical loading. Equations (4-5) put into evidence terms which are function of the defect δ₀ cos n₀ θ in the equation of the n₀ mode:

\[
2 \left( n₀ - \frac{1}{n₀} \right)^2 \frac{δ₀}{D} uₐ + 3 \left( n₀ - \frac{1}{n₀} \right)^4 \left( \frac{δ₀ + uₐ}{D} \right)^2 uₐ
\] (4-6)

These terms have to be compared to shell stiffness:

\[
\left( n₀^2 - 1 \right)^2 \frac{h^2}{3D^2}
\]

Therefore defects play a role if δ₀ ~ h. The preceding table illustrates this point in the case of a hemispherical shell. Generally defects tend to reduce critical loadings. This effect is more important if the main Fourier azimuthal component n₀ of the defect coincides with the Fourier number n of buckling mode (defined as in 4.1.4.2).
4.1.4.4 Dynamic buckling

Let us consider now a dynamically applied loading, which leads to an imposed $n = 0$ motion $u_0(t)$. The dynamic equation for mode $n_0$ can be written from (4-5) stiffness terms and, for simplicity, assuming that no defect is present, then:

$$m_n \dddot{u}_n + 2 \epsilon_n m \omega_n \ddot{u}_n + m \omega_n^2 (1 + \frac{u_0(t)}{u_E}) u_n = 0$$  \hspace{1cm} (4-7)

with: $u_E = \frac{2u_0^2}{3} \frac{h^2}{D} = \text{displacement corresponding to static Euler buckling (n = 0)}$

$$\omega_n^2 = -\frac{(n_0^2 - 1)^2}{3} \frac{h^3}{D^3} \frac{2\pi E}{m_n}$$

$m_n$ = generalized mass of an equivalent ring

$\epsilon_n$ = damping coefficient

For $u_0(t) = 2\pi u_E \cos \omega t$

which is a classical Mathieu equation, the stability domain is given by fig. 4-19. It shows that if the ratio $\Omega = \omega/\omega_n$ is close to certain value such as 2, 1, 2/3 etc., instabilities are observed for $\mu < 0.5$ (Euler).

Fig. 4-19: Instability diagram of a MATHIEU Oscillator

(Source : ref 3-1)
In the case of a seismic excitation \( \omega \) can represent the resonance frequency of a global mode of the structure ("seismic mode" \( n = 0 \) or \( n = 1 \)). Therefore the Mathieu model predicts a strong decrease of buckling load if there is a frequency coincidence between a seismic mode and the buckling mode. Such a coincidence is very probable for cylindrical shells (\( n = 0 \) or \( 1 \) and \( n = 12 \) for typical LMFBR shells). However such a behaviour is not experimentally verified: for cylinders, buckling critical loads are higher dynamically than statically. Let us present the results of a typical test:

A cylindrical shell (4-20) coupled to two fluid sheets is excited by an imposed ground motion (sine sweep in a frequency domain containing the main seismic mode and the buckling mode which are chosen to be close to each other). Tests are carried out at different levels of intensity.

Fig. 4-20: Scheme of a Dynamic Buckling Experimental Rig

The maximum of fluctuating pressure in fluid sheets is plotted versus maximum ground acceleration. When no permanent differential pressure acts on shell, no instability is detected. The experimental curve of fig. 4-21 shows a saturation effect which can be predict only using the full ring model. The main reason is that the full model takes into account the coupling effect between mode 0 and \( n_0 \) oscillators due to defects, which tends to separate the resonance frequencies and create coupling modes as shows fig. 4.21 where the evolution of the first resonance frequencies of a cylindrical shell with defect versus \( n \) is plotted (linear analysis). The solid curves of fig. 4-22 correspond to non-linear calculation for various values of damping coefficient \( \epsilon \). The dashed curves correspond to linear calculations.
Fig. 4-21: Effect of Shape Defects on the Resonance Frequencies of a Cylindrical Thin Shell

Fig. 4-22: Maximum Fluctuating Pressure Versus Ground Acceleration. Comparison Between Experiment and a Non Linear Numerical Model

(Source: ref. 14)
It is noted that the agreement between experimental and calculated curves is better if damping coefficients are high (an increase in damping is justified considering the large displacements of shell and fluid). The figure also shows that a prediction based on a linear estimation of fluctuating pressure field and a static buckling analysis using Euler critical loading is too conservative.

4.1.4.5 Conclusion

The accurate calculation of the seismic response and the buckling instability risk of internals is very complex. In practice only a linear seismic analysis including fluid-structure interaction is performed. From this calculation a "maximum" seismic pressure fields $P_d$ is defined which corresponds to the "worse" loading of shells reached during seismic excitation. This field combined with the permanent pressure $P_s$ effects is used to make a classical static buckling analysis (generally linear) which leads to critical value $P_{dc}$ and $P_{sc}$.

Finally the margin due to dynamic effects is estimated using diagrams shown in fig. 4-23 which are produced from experimental results or dynamic analysis results of the cases described before.

(Source: ref. 14)

Fig. 4-23 a  Fig. 4-23 b
Abacus for the Estimation of the Dynamic Buckling Effects
4.1.5 Isolation device effects

The benefits of isolation devices will be further discussed in section devoted to design aspects. Concerning the calculation methods we must note two difficulties, which stem from the fact that seismic isolation brings about low resonance frequencies, typically 0.3 to 1 Hz:

- The regulatory seismic spectra are adjusted in the classical frequency range (1 Hz - 30 Hz). Special spectra have to be defined in (0.3, 1 Hz) range and therefore a better knowledge of seismic energetic distribution in this range is needed. This point is particularly important in LMFBR analysis because of the existence of low frequency sloshing modes and of the sensitivity of shells to buckling.

- As we mentioned in paragraph 3.2.3, isolated systems have very separated resonance frequencies: low frequency swinging mode due to isolation device (typically less than 1 Hz) and medium frequency structural modes (typically > 3 Hz). For such systems quadratic combination fails and an improved formula has to be used.

It is also noted that isolation devices often have a non-linear behaviour (sliding for example) which must be taken into account in the calculations.

4.2 Core analysis

4.2.1 Non-linear behaviour of core structures

LMFBR core is composed of fuel, shielding and other types of subassemblies which essentially are cantilever hexagonal beams imperfectly clamped at their lower end on a diagrid and free at their upper end. Small gaps are maintained between them by pads usually located near the upper end. Core is divided into 3 zones:

- fuel elements zone
- fertile elements zone
- Lateral neutronic shield elements zone.
Fig. 4-24 shows such elements in the case of the RAPSODIE mock-up (Fig. 4-25) which is a 1/4 scaled schematic representation of Superphénix core. These 3 types of beam structure have different dynamic characteristics. Typically fuel elements have a first resonance frequency rather lower than shielding elements. For this reason, during a seismic excitation impact shocks occur between the adjacent subassemblies generally at the pad level where nominal gaps are almost zero.

(Source: ref. 20)

Fig. 4-24: Rapsodie mock-up
This non-linear behaviour must be taken into account in the seismic analysis to assess the impact forces on the pads and to evaluate the relative motion and the variations of core volume which has a direct effect on core reactivity. Once again the only available method to deal with such non-linearity is the step by step time-history analysis.

In order to reduce cost, the degrees of freedom of the problem and the time integration algorithms have to be optimized. In our case, the basic structures are relatively simple (beams), and the eventual contact points are well known and their number is limited. Moreover the frequency domain of interest is low. For all these reasons, a mathematical description using the first few modal contributions of each beams (substructures) is the best way forward. In order to be consistent with such representation, the shock conditions must include a "local flexibility" sometimes called "shock or impact flexibility" which represents the effect of the neglected modes (truncation effect). Such a non-linear system does not contain any natural frequency up to the cut-off frequency $f_c$ chosen for the modal basis. Therefore explicit time integration algorithms correctly work using a time step which has...
Fig. 4-26: Comparison Experiment – Calculation for a Fuel Assembly Displacement

Fig. 4-27: Comparison Experiment – Calculation for a Neutronic Shield Element Displacement
the same order of magnitude as $1/f_c$. This technique allows the performance of not too expensive a calculation although models which include all the S/A elements of a LMFBR core (~1000), lead to large computer times. Available computer codes based on these techniques are CORALIE (CEA & ENEA) and CLASH (Belgonucléaire).

Fig. 4-26 and 4-27 show a comparison between RAPSODIE mock-up results and CORALIE calculation. Tests were performed in air. The modelisation only concerns the central row of S/A elements. The figures show time history displacement responses and corresponding oscillator response spectra, for an OBE earthquake input.

4.2.2 Fluid-structure interaction

The whole core is immersed in sodium which creates an inertial coupling between all the elements of the core. The fluid volume is 3-dimensional and in zones where shocks occur fluid-structure interaction is completely non-linear. These two aspects of the fluid interaction may be treated separately based on the following hypothesis.

4.2.2.1 Local aspect

The non-linear geometrical effects essentially occur in the vicinity of two adjacent pads which impact each other. The presence of fluid modifies the law which connects the relative displacement, velocity, acceleration and force. To estimate this law, we have to solve the local transient flow problem in the gap between the two moving parts. Three components of force can be identified:

- inertial force
- viscous force
- force due to turbulent pressure drops.

If $x(t)$ is the relative distance between the pads, we generally have:

$$ f(t) = - m_f \frac{1}{2} \left( \frac{x}{x} + \Psi \left( \frac{x}{x} \right) \frac{x^2}{x^2} \right) $$

(4-8)

$m_f$ and 1 are characteristic added mass and length.
Depending on the parameters of the system, i.e.: impact velocity $V_0$, $m_f l$, equivalent mass of moving bodies $M$ and fluid dynamic viscosity $\nu$, the shock is dominated either by the viscosity force or by the turbulent pressure drop force. Let us consider the equivalent length $\tilde{l} = m_f l / M$ and express the Reynolds number as $R_e = V_0 \tilde{l} / \nu$.

if $R_e \sim 1$ "viscous shock" dominates
if $R_e \gg 1$ "turbulent shock" dominates

$$l = \frac{2}{3} \rho_f L^3 \frac{1}{M}$$

Fig. 4-28: Scheme of an Idealized Fluid-Structure Interaction Problem During a Shock

An example of a rigid rectangular body of mass $M$ which impacts a rigid wall is shown in fig. 4-28. The evolution versus time of the resulting displacement, velocity, acceleration and forces for two cases a) $R_e = 3$ and b) $R_e = 3 \times 10^3$) are given in figures 4-29 and 4-30 respectively. It is noted that no rebound of the rigid body occurs, its whole kinetic energy is absorbed by viscous effect in the first case and by turbulent jet effect in the second case.

This result is not directly applicable to LMFBR subassemblies which are flexible. In fact potential energy due to local deformities are recovered during the shock and only a part of the total energy is lost. A comparison of tests in air and in water, for otherwise the same conditions performed on the Rapsodie mock-up, shows a decrease of the level of the response due the damping effect of water (fig. 4-31).
a) Projectile displacement

Case a:
- Projectile mass: $M = 25 \text{kg}$
- Fluid density: $\rho = 10^3 \text{kg/m}^3$
- Impact surface length: $2L = 200 \text{mm}$
- Kinematic viscosity: $v = 10^{-4} \text{m}^2/\text{s}$
- Initial gap: $e = 50 \text{mm}$
- Initial velocity: $V_i = 10^{-3} \text{m/s}$

$\Delta t = 3.33 \text{mm} \quad R_e = 3.33$

b) Projectile velocity

c) Projectile acceleration

d) Total fluid force

e) Fluid inertial force

f) Pressure drop force

g) Viscosity force

(Source: ref. 3.1)

Fig. 4-29: Calculation Results in the Case of Predominant Viscous Effects
(Case a)

Projectile mass: \( M = 25 \text{ kg} \)
Fluid density: \( \rho = 10^3 \text{ kg/m}^3 \)
Impact surface length: \( 2L = 200 \text{ mm} \)
Kinematic viscosity: \( \nu = 10^{-4} \text{ m}^2/\text{s} \)
Initial gap: \( \varepsilon = 50 \text{ mm} \)
Initial velocity: \( V_0 = 1 \text{ m/s} \)

\[
T = 3.33 \text{ mm} \quad R_n = 3.33 \times 10^3
\]

Fig. 4-30: Calculation Results in the Case of Predominant Turbulent Effects
Fig. 4-31 a: Experimental Analysis of Water Effects in Rapsodie Mock-up

Fig. 4-31 b: Experimental Analysis of Water Effects in Rapsodie Mock-up
4.2.2.2 Global effect – homogenisation techniques

The results of non-linear calculations of LMFBR core shown that the motions of the fuel elements zone, neutron shield elements zone, etc., have global characteristic features, in spite of important local relative displacements between each element. The local fluid-structure interactions described in the previous section cannot represent the fluid effects associated with this type of global motion. The fluid coupling between the adjacent S/A and also between all other S/A and the surrounding structures (characteristic length or core diameter) would need to be considered. However, calculations including all the coupling terms are too expensive and generally not very fruitful.

In order to estimate the global fluid-structure effects, there is no need to distinguish each beam and the inhomogeneous medium consisting of beams and fluid can be represented by an equivalent homogeneous medium. The techniques used to derive the equivalent characteristics of this homogeneous medium consist of solving a set of elementary cell problems:

\[
\begin{align*}
\Delta X_j &= 0 \quad \text{in } y^* \\
\frac{\partial X_j}{\partial n} &= y_j \cdot n \quad \text{in } (\gamma) \\
X_j &\text{ periodic on the cell} \\
\int (y^*) X_j \ d \Omega &= 0
\end{align*}
\]  \hspace{1cm} (4-9)

Fig. 4-32: different type of cells

Where \(X_j\) represents the pressure field in the incompressible fluid domain of a unit cell \((y^*)\), induced by a unit translation of the beam in the \(j^{th}\) direction \((y_j)\). From \(X_j\) the coefficient \(\alpha_{ij}\) and \(\beta_{ij}\) of two tensors \(A\) and \(B\) can be deduced:

\[
\begin{align*}
\alpha_{ij} &= \int (y^*) \left( \delta_{ij} + \frac{\partial X_1}{\partial y_j} \right) \ d \Omega \\
\beta_{ij} &= \int (y^*) \ \frac{\partial X_1}{\partial y_j} \ d \Omega
\end{align*}
\]  \hspace{1cm} (4-10)

\[\delta = \text{Kronecker index}\]
Then the equations of the equivalent homogeneous medium consist of an equivalent compressible fluid equation (variable p) and an equivalent structural equation (variable \( x \)):

\[
\begin{align*}
\text{div} \left( A \nabla p \right) - \frac{1}{\gamma^*} \frac{\partial^2 p}{\partial t^2} + \frac{1}{c^2} \frac{\partial p}{\partial t} &= \rho_f \text{div} ( Bx ) \\
\left( \frac{m}{d^2} I + \rho_f B \right) \dot{x} + \frac{k}{d^2} I x &= - B \nabla p
\end{align*}
\] (4-11)

\( \gamma^* \) is the area of the cell, \( d \) its characteristic size.

\( m \) and \( k \) are the mass and stiffness of the beams.

\( I \) is the unit tensor.

Note: As system (4-9) shows, the basic hypothesis of the homogenisation method consists of the quasi-periodicity of motions from a cell to the adjacent one. This corresponds to a weak evolution of long distance effects. In particular two adjacent beams must have almost the same motion.

Therefore the homogeneous model cannot represent the local fluid effects. Moreover the behaviour of the system is assumed to be linear. However it is possible to represent both local and global effects by considering cells containing several tubes as shown in fig. 4-33. Then the elementary solution includes the differential motion between adjacent tubes. Moreover certain non-linearities can be introduced.

![Fig. 4-33: Different Types of Multi-Cell Elements](image)

The equivalent medium defined above can be connected to other structural shells or fluid volumes. In particular the use of the homogenisation technique is well suited to calculate the coupling between core and adjacent structures (cover plug etc). For example, the
first resonances of the RAPSODIE mock-up have been calculated in air and in water. The table below shows a comparison between analytical and experimental results:

<table>
<thead>
<tr>
<th>Tests</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>resonance modes</td>
<td></td>
</tr>
<tr>
<td>Freq. in air</td>
<td>Freq. in water</td>
</tr>
<tr>
<td>(Hz)</td>
<td>(Hz)</td>
</tr>
<tr>
<td>Fuel S/A</td>
<td></td>
</tr>
<tr>
<td>dominant</td>
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<td>6</td>
</tr>
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<td>7,9</td>
<td>6,5</td>
</tr>
<tr>
<td>Shielding S/A</td>
<td></td>
</tr>
<tr>
<td>dominant</td>
<td></td>
</tr>
<tr>
<td>20,7</td>
<td>16</td>
</tr>
<tr>
<td>20,4</td>
<td>16,5</td>
</tr>
</tbody>
</table>

Note: The mock-up consists of hexagonal assemblies surrounded by a rigid cylindrical vessel.

The fluctuating pressure acting on the vessel has been measured during a seismic excitation. Fig. 4-34 shows the axial evolution of the ratio of maximum fluctuating pressure/maximum displacement of neutron shields element along the element length.

Fig. 4-34: Mock-up Rapsodie - pressure field on the vessel: comparison test-calculation (100% OBE)
4.2.3 Equivalent linear model

As shown, it was important to take into account the non-linear behaviour of the core. However the core is not an isolated structure and has to be considered together with the whole reactor block. It is impractical to perform a non-linear calculation of the whole reactor block including the core, the method generally used consists of performing a linear reactor block analysis with an equivalent linear core model.

This equivalent model depends on the core non-linear characteristics, but also on the excitation characteristics, for this purpose (the seismic motion of the diagrid can be regarded as the seismic source for the core). From a preliminary reactor block calculation the diagrid spectrum is obtained. Then two ways are possible to derive the characteristics of the equivalent linear model:

a) Time history accelerograms are deduced from the diagrid spectrum and non-linear calculations are performed to obtain response spectra. The ratios between response and source spectra can be considered as equivalent transfert functions which are described by few linear harmonic oscillators.

Fig. 4-35 shows the example of Rapsodie mock-up. The pseudo-transfert function is represented by the ratio of the ORS core reaction on its base and the ORS of the lateral acceleration of this base for a given seismic horizontal excitation. Two cases are considered: a) low level excitation and b) high level excitation. For both cases curves obtained by non-linear computation are compared with curves obtained by equivalent linear model. It is noticed that for high level excitation case (great importance of shocks) more oscillators are needed to represent the pseudo-transfert function.

b) A stochastic model of the excitation can be deduced from the floor spectra (generally a stationary gaussian random process w(t) weighted by a deterministic envelope E(t)) as described in the section 3.2.3.2. The non-linear response X(t) of the system is formally given by:

\[ M\ddot{X} + C\dot{X} + KX + G(X, \dot{X}) = E(t)w(t) \quad (4-12) \]

where G expresses the non linear part of the system.
Fig. 4-35 a: Case a: Fitting of the Parameters of an Equivalent Linear Model for two Levels of Excitation

Fig. 4-35 b: Case b: Fitting of the Parameters of an Equivalent Linear Model for two Levels of Excitation
On the other hand, let us consider the equivalent linear system excited by the same source:

\[ M \ddot{X}_e + C \dot{X}_e + K X_e + C_{e} \dot{X}_e + K_{e} X_e = E(t) w(t) \]  \hspace{1cm} (4-13)

The error \( \varepsilon = X - X_e \) between \( X \) and \( X_e \) can be minimized by means of a minimum mean square technique:

\[ \frac{\partial \langle \varepsilon^T \varepsilon \rangle}{\partial K_e} = 0 \quad \text{and} \quad \frac{\partial \langle \varepsilon^T \varepsilon \rangle}{\partial C_e} = 0 \]  \hspace{1cm} (4-14)

\( K_e \) and \( C_e \) are obtained directly from the equation 4-14.

In some cases, iterations of the linear analysis of the entire block and the core non-linear calculations may be necessary due to a strong mechanical and fluid coupling between the core and vessel internals. This was for instance the case of PEC reactor (ref. 21).

The above technique has been applied to the 1-DOF shock oscillator (see fig. 4-36).

Fig. 4-36 : 1-DOF shock oscillator

Fig. 4-37 shows the plot of the RMS value of the displacement of the mass \( m \) versus gap \( e \). The figure compares the exact curves obtained by the resolution of the associated Fokker-Planck equation and the approximated curves obtained by using the present method.
4.3 Seismic analysis of major LMFBR components

4.3.1 Control rods and drive mechanisms

Control rods and drive mechanisms are very flexible systems which could have a large response due to seismic excitation. This excitation is applied at different levels of the reactor block e.g. rotating shield plate, guide tubes, control assembly, as shown in fig. 4-38 and the applied motions are different in the level and frequency content. Moreover shocks can occur, for example between dashpot or grip mechanism and guide tube.

The main objective is to verify, that during seismic event the rods can fall down in an acceptable time. Fig. 4-40 shows the increase of the drop time, due to an earthquake, observed on a Superphénix type control rod tested on the VESUBY facility of CEA (shown in fig. 4-39).
Concerning the computation methods used to solve this problem, it is necessary to take into account shocks and friction between the rod and the guide tube or subassembly. Also, some of the problems described in the core analysis section must be solved i.e. shocks and friction, local effect of fluid films etc.
Fig. 4-40: Effect of Seismic Excitation on the Control Rod Fall Down Duration

Fig. 4-41 a and b show a comparison between calculation and experiment concerning the relative displacement and stress history of a Superphenix control rod. Different excitations are applied at 3 points of the structure. In this case, the control rod is held in its withdrawn position.

(Source: ref. 27)

Fig. 4-41 a: control rod stress
The transient calculation during the rod fall is more difficult. The nodes of both fixed and moving structures, which may or may not be in contact, change and this variation is determined by the calculation of the drop which depends on the vertical friction forces, these in turn are functions of the impact forces. Therefore two coupled analyses must be performed.

4.3.2 Piping systems

Piping systems are very common items of the nuclear power plants. The peculiarity of LMFBR piping is the low thickness to diameter ratio. Typically, linear methods of piping seismic analysis are used. As for the control rods, the multi-support excitation and the non-linear behaviour of the supports is an important aspect. Non-linear time history calculation can also be made. This type of analysis is generally devoted to the main piping which play an important safety role.
4.3.3 Sodium pumps

The special features of the pump shafts and bearing systems have been described in the IAEA report on flow induced vibration. Especially the gyroscopic effects and the fluid-elastic effects due to the rotating flow around the shaft can significantly modify the seismic response (note that in the case of a seismic excitation the "retrograde" modes can be excited). Moreover the relative motion between the journal and the bearings must be determined, in order to verify the behaviour of the bearing components and to prevent a bearing seizure.

In fact it can be shown that during a seismic excitation, these relative motions are of the same order as the fluid film thickness (bearing clearance). Therefore, equivalent stiffness and damping factor of the bearing vary with time. Shocks and friction can also occur. Once again, non-linear time history calculation must be done to solve the Reynolds equation and taking into account unilateral contacts between the journal and the bearing. This type of analysis is very demanding on computer time and typically an equivalent mean bearing stiffness and damping ratio is determined and the absence of excessive relative motions is linearly verified. This type of analysis is validated by tests:

For Superphénix type primary pumps, tests of the hydrostatic bearing on a shaking table have been performed. Stator-rotor relative displacements and pressure fluctuations in the fluid film have been measured. Also dynamic tests on a real shaft performed in sodium on BRASIMONE facilities have furnished a full scale demonstration.

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/35/ I.G. FIGURINA, "Consideration of Seismic Loading in Nuclear Power Plant Piping Calculations".

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/40/ A. PREUMONT, A. PAY, A. DECAUWERS, "Seismic Analysis of a Large FBR Core, some SNR2 Preliminary Calculations".

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/44/ A. SAKURAI, Y. MASUKO, K ISHIHAMA, Y.W. CHANG, E. RODWELL, "EPRI/CRJEPIL Joint Program on Seismic Sloshing of LMR Reactors; Part I Experimental tests".


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D.C. MA, J. GVLILDYS, "Seismic Analysis of a 400-MWe Pool-Type Reactor".

C.Y. WANG, "Seismic Analysis of Liquid Metal Reactor Piping Systems".

K. BUSCH, K. PETERS, W. ROSENHAUER, "Seismic Design Principles of the German Fast Breeder Reactor SNR2".

P. BULAND, A. BERNARD, J.P. DEBAENE, "Seismic Calculation of Superphenix Control Rod and Drive Mechanism, Comparison with Test Results".


S. FUJIMOTO, H. YOGUCHI, H. HIRAYAMA, A. SAKURAI, Y. MASHIKO, "Experimental Study on Shear Key Restraint for Earthquake".

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5. IMPACT ON THE DESIGN AND NEEDS FOR TECHNOLOGY IMPROVEMENT

5.1 Problems of the fast reactor seismic design

Because fast reactor systems operate at virtually atmospheric pressure and must cope with high temperatures and a coolant which possesses excellent heat transfer properties, the fast reactor vessels, piping and other components tend to be very thin-walled and flexible. Since these items are made of stainless steel, there is an additional incentive to keep the component structural thickness low to save costs. As a result, these flexible structures have little inherent resistance to earthquake effects.

The presence of rather flexible structures and non-negligible clearances may lead to considerable amplification of the seismic motion. Furthermore, in the case of the pool-type reactors, the presence of large vessels with thin walls might entail dynamic buckling. These large vessels, which contain sodium, might also be subjected to sloshing waves during an earthquake.

5.2 Need for research and development

Obviously, design of fast reactors must be such as to prevent or limit the above mentioned phenomena to acceptable levels. The key point is how to balance the conflicting requirements for low thermal stresses and high earthquake resistance. For thermal reactors an adequate definition of the reference earthquakes (Safe-Shutdown Earthquake - SSE and Operational Basis Earthquake - OBE) and the correct use of existing design criteria and methods is generally sufficient to verify whether the structural and operational seismic safety requirements are satisfied. However, for the fast reactors, detailed numerical and experimental studies are also necessary to correctly analyse the complicated phenomena which are related to their specific features.

These studies are necessary for most of the major components, for which the standard methods are not directly applicable. In particular, the experimental analysis must be performed in order to validate the advanced numerical methods described in the previous chapter of this report. Also design specific experimental data are necessary to apply these methods to the design.
5.3 **Seismic design of the existing reactors**

In the recent years, great efforts have been devoted by all the countries involved in fast reactor design and construction to the development, improvement and validation of the above-mentioned methods. On the other hand, the lack of sufficient understanding of many of the physical phenomena affecting the seismic response of fast reactor structures, often forced the designers to adopt very conservative design solutions and to use very stringent design criteria and methods to guarantee seismic safety. To this attitude also contributed, in some countries, a lower experience of earthquake resistance of structures in general, or in other countries, the unsufficient knowledge of site seismicity.

In Japan, for instance, the experience of earthquake damage has led to a kind of conservatism, in a design of major structures, based on historically verified technology (ref. [1]). According to this experience, the present Japanese design philosophy leads to the construction of nuclear power plants on a firm rock site and to the adoption of the concept of rigid structures. This is also applied for fast reactor design.

Indeed, in spite of the fact that floor response spectra might be reduced to 1/3 - 1/5 on soft rock sites compared to firm rock sites (ref. [1]), much better knowledge of the response of buildings constructed on firm rock sites is presently available in Japan (experience has demonstrated stability and reliability of firm rock). Moreover, with regard to the concept of rigid structures, it is based on the assumption that little energy is lost by elastic deformation of the structure.

Thus, as far as the existing fast reactors are concerned, the results of the research and development studies which are being conducted were mainly used to verify design adequacy.

5.4 **Impact of the seismic conditions on designs and projects in the past**

The acceptance of very conservative design solutions and the use of too stringent design criteria and methods not only caused large additional costs with respect to a design optimized for a thermal behaviour, but also entailed difficult construction and maintenance problems, and affected the overall system reliability.
Furthermore, the unsufficient knowledge of seismic behaviour - and in some cases the seismic conditions - at the early design stage, together with changes of the seismic requirements as the knowledge improved, made it necessary to introduce significant modifications of the design of some fast reactors and/or to perform extensive re-analysis and re-testing of previously designed components. Some modifications had to be made even during construction. In all the cases, these modifications and reanalyses led to higher, additional costs and layout complications.

For instance, FFTF and CRBR experienced a rapid changes in seismic design technology in the U.S.A. (soil characterization, soil-structure interaction, component response, etc.) and as a result, costly changes were necessary. In 1972, the FFTF had to change building and equipment seismic design response spectra (ref. [2]). The pipe supports and snubbers initially supplied and installed in FFTF were found to be inadequate. Before FFTF startup, replacement snubbers and strengthened pipe supports of re-qualified design had to be installed at a cost of $20 million to the project.

Also the CRBR project has suffered plant cost increase due to changing seismic requirements in USA (refs. [2, 3]): after the project was well underway, it was deemed necessary for the plant earthquake level to be increased from 0.18 g to 0.25 g. Many re-analyses and re-designs, such as the turbine building, led to an increased cost of about $30 million.

As far as other countries are concerned, the example of the Italian PEC fast reactor is very significant (refs. [4, 5]). In 1982, after the main vessel had been constructed and was ready to be mounted in the reactor, more detailed core seismic analysis methods - which became possible at that time - revealed that it was necessary to modify the reactor-block design, in order to limit core seismic response to a level compatible with shutdown feasibility.

The advanced construction stage of the PEC reactor limited the possible modifications to a solution which adopts a core restraint ring combined with thick pads on all the core elements and an improved element spike design. Later on, detailed evaluation of the neutronic and thermal effects of the SSE in the presence of the core restraint ring, forced the
designers to further increase pad thickness. These design solutions made it necessary to perform very expensive re-analysis and re-testing on shake table. Tests and reanalyses were also needed to verify that fuel element handling remained feasible.

Especially the U.S. experience has shown that the cost of a seismic design in all types of nuclear plants has been high when the state-of-the art methods and overly conservative regulatory requirements are used (ref. [3]).

5.5 The key point of cost reduction for future plants

It is necessary to remember that one of the key points of LMFBR development is cost reduction in order to make fast reactors more competitive with LWRs. In this respect, it must be stressed that seismic design is a major cost factor for nuclear power plants in the countries which are characterized by non-negligible seismicity. For the reasons mentioned above, this is be especially true for the fast reactors.

For example, the cost of a seismic design and analyses of the past U.S. LMFBR projects was about 6% of the overall capital cost (refs. [2, 3]). However, seismic design changes and backfitting have caused delays in construction and licensing, and seriously increasing plant lead times and costs of the LWRs in the U.S.A. Future LMR plants in U.S.A. will be vulnerable, to at least the same extent as experienced by LWRs. The commercial viability of fast reactors in the U.S.A. therefore depends on the reduction in capital cost made possible by improvement in seismic design.

From the past experiences, some general lessons for improved seismic design include the following (ref. [2]):

- designs need to be insensitive to close tolerances, have some flexibility to cope with a possible changes in design requirements without major redesign;

- safety related components should be separated from other structures and components to eliminate their interaction;
- standard designs and generic qualification techniques should be used as far as possible;

- seismic methodology needs to be established and generally acceptable.

Although these conclusions are affected by the particular problems that nuclear power plants suffer in the U.S.A., they constitute more general design objectives for the future fast reactors.

5.6 Safety improvement and cost reduction through further R & D

As stressed at the IAEA/IWGFR Specialists Meeting on Fast Breeder Reactor Block Antisesimic Design and Verification (Bologna, October 12 - 15, 1987), a large amount of work has already been performed in the framework of seismic analysis of fast reactors (ref. [6]). Rigorous methods - which are rather similar in all the countries - were used for the seismic design and verification of the existing fast reactors.

However, further improvements are judged necessary for the future fast reactors, in order to achieve a better knowledge of the physical phenomena, and thus to allow more realistic, less conservative, criteria and methods to be adopted. The aim is a general improvement of the design, which may result not only in a decrease of plant costs, but also in a further increase of the already high overall reactor safety level.

Indeed, the use of unnecessarily too severe seismic criteria and methods, beside unnecessarily complicating plant design, might also make reactor behaviour worse in normal operating conditions and complicate plant safety demonstration with regard to accidents that may be much more frequent than the very improbable design earthquakes (ref. [6]).

For demonstration of seismic safety, it is also important that Probabilistic Risk Analysis (PRA) methods are developed and applied to the fast reactors. Indeed, PRA is becoming very important, particularly for the beyond-design-basis event (see, for instance, ref. [3]).
Further improvement of technology and analysis methods is also judged necessary by countries in which the seismic requirements have not so far greatly influenced, the fast reactor design. This is the case of the U.S.S.R., where fast reactors (for instance BN-800) have been built in low seismicity zones (refs. [21, 37]). But the future fast reactors in the U.S.S.R. might be located in higher seismicity sites.

The following sections give examples of how the seismic analyses have influenced the design of different structures of the existing fast reactors, and how the design of future plants could be rationalized.

The examples given will be mainly based on the presentations and discussions made at the IAEA/IWGFR Specialists Meeting on Fast Reactor Development of Bologna and on the already quoted papers of refs. [1, 2, 4]. The latter papers were presented at the International Symposium on LMFBR Development, which was organized by the Japanese Institute of Applied Energy and held in Tokyo on November 6-9, 1984. They provided useful information on the approaches followed in Japan, the U.S.A. and the European countries with regard to aseismic design. Finally, reference will also be made to papers presented at ENEA/ISMES/ENS Specialist Meeting on On-Site Experimental Verification of the Seismic Behaviour of Nuclear Reactor Structures and Components, held at the ENEA Brasimone Center on May 4-7, 1987.

5.7 Design earthquakes

5.7.1 General need for a rationalization of the seismic input

At the IAEA/IWGFR meeting of Bologna, the F.R.G. (ref. [7]) stressed that, for the fast reactors KNK II and SNR-300, earthquake protection had to be conceived in the early seventies when only comparatively rudimentary notions existed about seismic events and their possible implications for nuclear plants in Germany. Today, an extensive seismological data-base exists, which allows for classification and probability of earthquakes and characterization of the site. Most likely sites in the F.R.G. and also in many European countries are characterized by low seismicity, which has to be accounted for by realistic seismic protection measures.
Thus, the seismic design principles which are used in the F.R.G. for the SNR-2 project are restricted to German or other low seismicity zones with SSE intensities lower than VIII and return frequency lower than $10^{-4}$/year. The approach is based on best estimate of input data without further conservatism and no considerations of OBE. The studies which are being currently carried out in the F.R.G. to determine site dependent ground motion time-histories consistent with the assumed intensity values will contribute to the optimization of the seismic design of the SNR-2 project.

Also the seismic design of the Italian fast reactor PEC was affected by an insufficient knowledge of site seismic conditions. This led to the definition of very high SSE and OBE levels (0.3 g and 0.15 g, respectively), although the reactor site area was not considered as seismic by the Italian law for civil construction (ref. [4]). Although detailed seismological studies (ref. [8]) have recently shown that such high levels were based on a wrong interpretation of historical data and confirmed that the site area corresponds to moderate seismicity, the seismic qualification had to be performed for the high earthquake levels.

5.7.2 Need for more rational OBE criteria

With regard to the OBE requirement, it is noted it may be more stringent for seismic qualification than the SSE requirement if the OBE is defined according to the present U.S. codes (upset plant conditions and design limits, five OBE events, each equal to at least 50% SSE). A comparison of fast reactors seismic design in different European countries has shown that the design problems were somewhat different in countries which do not have the OBE requirement (e.g. the UK).

On the other hand, the OBE criteria are considered ambiguous even in the U.S.A. (ref. [2]). In fact, the U.S. N.R.C. has accepted OBE magnitudes less than 50% SSE for a considerable number of plants. Here, the OBE magnitudes ranged between 20% and 40% of the SSE level and they were usually tied to earthquake probabilistic return periods in the range of 100 to 400 years. The return periods for the SSE was from about $10^4$ to $10^6$ years. Return periods for 50% SSE range from about 300 to 1500 years. Ref. [2] stresses that many experts in the U.S.A. favor a 100 to 300 years recurrence interval for OBE: among other factors, this would
be more consistent with the code position regarding the OBE as an upset plant condition.

Also the number of OBEs required by the codes (e.g. five used for PEC seismic design) has been judged in the U.S.A. to be excessively conservative (for the present OBE level). The study, ref. [9], recommended that a minimum of two OBEs be assumed to occur during the plant life.

It is noted that, for the design of the Superphénix-1 reactor, while the ratio 1/2 was maintained between the two reference earthquakes, emergency plant conditions were associated with the OBE (ref. [4]). This entailed the use of only one OBE and also less stringent stress criteria were used to demonstrate that structural damage was limited to allow for a safe restart after some repairs (ref. [10]).

The use of the more realistic OBE values means that the OBE criteria is based on economic risk. Indeed, the improvement of seismic analysis methods has been such that the SSE verification methods are sufficiently reliable as to guarantee seismic safety of nuclear power plants. Therefore, there is less justification to use high OBE levels as a back-up for plant safety demonstration.

5.7.3 Need for improved acceleration data

As far as design earthquake definition is concerned, the comments made in ref. [1] are especially important for fast reactors. The first point to be stressed is that the vertical excitation is more important for the fast reactors, than for the LWRs. To this purpose, the definition of the vertical design earthquake including its phase relation to the horizontal motion is essential.

Then, it must be pointed out that correct sloshing analysis of liquid sodium in the main vessel requires the specification of the long-period components of the earthquake ground motion. Also regarding the seismic isolation, the lower frequency components of the ground motion play an important role.
In Japan, because of the concept of rigid structures, attention has been paid mainly to the high frequency components and the recorded strong-motion accelerographs are not sensitive enough in the low frequency range. Thus, according to ref. [1], research is needed to record ground motions which are appropriate to the design earthquakes for seismically isolated structures.

5.7.4 Importance of good knowledge of near-field earthquakes

It is noted that a reliable definition of design earthquakes not only requires a correct definition of the far-field earthquakes (on which design spectra are usually based), but also a good knowledge of potential near-field earthquakes. The latter, if not considered in the early design phase, may cause problems because their peaks might be at frequencies which had a low energy content in the far-field earthquakes (ref. [8]).

5.8 Effect of seismic analyses on the reactor building design

For the nuclear power plants, reactor buildings must be designed to withstand the earthquake and to minimize the seismic loads transmitted to the inner components.

5.8.1 Improvement of building structure design

Design of buildings must obviously lead to rigid and compact structures, such as to minimize amplification at the various floors with respect to the motion at the foundation. Technological development and better understanding of the effect of seismic event on the behaviour and safety of fast reactors have led to a considerable improvement in building designs. For instance, the solution proposed for CDFR ,ref. [4], (which is designed for a SSE of 0.25 g) shows that all the nuclear buildings – i.e. the steam generator, fuel handling and maintenance buildings – are structurally connected to the circular containment buildings, located in the center of the nuclear island(Fig. 5-1). The interlocked buildings, mounted on a rigid concrete raft, provide a high lateral stiffness and prevent a relative motion between the buildings.
In the mentioned CDFR design solution, the reactor vessel is mounted within a concrete vault below ground level. This reduces the overall building height by over 20 m and also significantly lowers the centre of gravity. In a plan view, the centre of shear rigidity is close to the centre of gravity as far as practically possible for reasons of eliminating torsional modes of vibration. The above features all contribute to very low seismic response in particular at the reactor roof support, which was considered of paramount importance.

In order to optimize building seismic response, the need for better damping and stiffness of reinforced data is recognised (see, for instance, ref. [1]).

5.8.2 Improvement of soil-structure interaction analysis

Besides adopting improved building designs, the evaluation of soil-structure interaction effects is essential not to overestimate the earthquake input to the reactor. To this aim, suitable experimental data have to be obtained and improved soil-structure interaction models based on these data have to be developed (see, for instance, ref. [1]). The effects of possible building-to-building coupling, such as those detected for PEC (refs. [5, 11]) must also be carefully analysed.
5.8.3 Usefulness of on-site tests and detailed analytical methods

To evaluate the floor response of the reactor building, taking into account soil-structure interaction, even on-site experimental tests and detailed (three-dimensional) numerical analyses, such as those performed for PEC (see Sect. 6.8.3) may be justified. These tests and analyses were judged necessary for PEC to check that first natural frequency of the reactor building had been correctly determined in the simplified design calculations, and to evaluate safety margins with respect to the floor-response spectra obtained by these calculations.

On-site tests on structural models of significant size, performed for instance in high seismicity areas, such as those of EPRI in Taiwan (refs. [12, 13]), may also provide important results. They allow for consideration of the effects of real earthquakes and inelastic material response, while on-site tests on the reactor itself are usually limited to rather low dynamic excitations.

5.8.4 Need for new design solutions and siting techniques

New design concepts or new siting techniques capable of reducing the seismic input transmitted to the foundations are considered to be promising (ref. [6]). Indeed, the application of improved analysis methods has shown that for high ground acceleration (SSE > 0.3 g, ref. [14]) the feasibility of the fast reactor design is questionable. Also the cost of aseismic design of this level of earthquakes could become disproportionately large if a conventional above ground layout of the plant is followed.

The most attractive from the above-mentioned design solutions and siting techniques are:

- embedment of the reactor building;
- seismic isolation;
- siting on soft rock sites;
- in-shore siting.
All these solutions (but especially seismic isolation) permit the design and construction of significantly lighter components and structures, which are less expensive and which are better able to resist thermal effects.

5.8.4.1 Embedment

Partial or complete embedment of the reactor vault offers a technologically feasible and economic solution, at least for moderate earthquakes (SSE values up to 0.25 g, according to the studies cited by ref. [14]). Also the studies performed in Japan (ref. [1]) showed that a significant cost reduction can be achieved through embedment: these studies indicated that 50% embedment leads to large reduction of floor-response spectra (Fig. 5-2).

![Diagram of floor response spectra and embedment options](image)

**Fig. 5-2: Example of floor response reduction due to partially embedded base mat. (Japan)**
However, reliable consideration of soil-structure interaction in the case of embedded reactors is often not possible by use of the current soil-structure interaction models (as stated in ref. [1], the sway-and-rocking model usually adopted in Japan cannot be applied to an embedded reactor).

5.8.4.2 Seismic isolation

Seismic isolation is considered as an alternative to plant embedment in the case of sites characterized by high seismic level (refs. [1-3, 14-15]). It decouples the structure and its contents from the destructive forces resulting from an earthquake. Thus, it extends the feasibility of fast reactors to zones of much higher seismicity. Other advantages are simplification of aseismic design, cost reduction, standard plant design and improved plant reliability.

The possibility of standard designs in the case of seismic isolation leads to a further cost reduction with respect to that related to the use of lighter structures and components. It also favours co-operation for common designs among countries which are characterized by different seismological conditions. This international co-operation is essential for the development of fast reactors.

It is noted that, in the case of isolated reactors, the emphasis is placed on ensuring satisfactory performance of the isolation system. The aseismic design and qualification of the rest of the plant is much simplified and is reduced to a secondary importance. On the other hand, for a non-isolated plant, the seismic qualification of each of the major components has its own peculiarities and has to be dealt with individually.

The horizontal isolation concept has already been used for some large buildings (for instance in the U.S.A., New Zealand, Greece, France and Yugoslavia) and for LWR plants at Koeberg and Cruas. The vertical isolation is considered by many to be much more difficult (see, for instance, ref. [2]).
Fig. 5-3a: Typical comparison of horizontal response spectra with and without aseismic bearing pads for reactor vessel support (USA)

Fig. 5-3b: Example of isolation system – Japan (vertically isolated floor for NSSS)
The potential use of various seismic isolation solutions for the fast reactors has been evaluated in various countries. A seismic isolation system has been proposed in France for the Superphinix-2 project (ref. [16]).

In the U.S.A. (ref. [2]) some very encouraging results, with regard to seismic response reduction and spectra shift, have already been obtained by analysing the effects of seismic isolation under nuclear island base mats and structures (Fig. 5-3). The results of the U.S. studies to date show a dramatic reduction not only in the input accelerations and forces to the reactor structures, but also in reduced loadings on the content of the structures (ref. [3]). Due to these results, the use of seismic isolation is being considered for innovative reactor plants such as PRISM.

In Japan, seismic isolation concepts are being developed for the future large-scale fast reactor plants, together with embedment concepts and new siting techniques (ref. [1]). The interest for seismic isolation is due to the fact that, in Japan, the design floor-response spectra are 5-10 times higher than in other countries. Three types of isolation are being studied (ref. [15]):

- combination of horizontally isolated building and vertically isolated floor for Nuclear Steam Supply System;
- three-dimensional seismic isolation of the reactor;
- isolation system of the main vessel.

Although seismic isolation appears technically feasible, it is not yet a fully proven technology, and some questions remain to be resolved from the safety and the licensing point of view. According to ref. [14], these are:

- performance of the system beyond design limit;
- effect of non-vertically propagating waves;
- variation of dynamic characteristics of the isolation and energy absorbing devices with other design parameters and time.

Furthermore, R&D is still needed for partial isolation and the vertical isolation, and development of engineering aspects related to construction, in-service inspection, maintenance, repair and replacement
As far as tests are concerned, it was stated at the IAEA/IWGFR Meeting of Bologna that testing the complete isolation systems rather than a single unit, which has dominated the experimental work so far, and using multi-directional excitation may be essential to demonstrate system reliability. These two issues are best answered by installation of seismic isolation devices to real structures in an environment of high seismicity (ref. [17]). On-site tests with forced excitation, such as those performed on the PEC reactor, might also be useful.

5.8.4.3 Siting on soft rock sites

The nuclear power plants in Japan, including fast reactors (JOYO and MONJU), are built on firm rock. The power station sites are mainly located in the areas of high electricity demand. However, these sites will be occupied by LWRs by the time when fast reactors are commercialized (refs. [1, 15]). Thus, the extension of the potential construction sites is judged essential for the commercialization of fast reactors in Japan.

Soft rock siting appears now feasible on the basis of the advancement in earthquake engineering. More recent experience showed integrity of buildings on a relatively soft rock site (diluvium layer) when the building is rigid enough (ref. [1]). Furthermore, soft rock siting has been found in Japan to be more advantageous than embedment (ref. [15] and Fig. 5-4), because it gives a lower seismic response.

5.8.4.4 In-shore siting

In the framework of the development of new siting techniques, in-shore siting is also being seriously considered in Japan (refs. [1, 15]). Although vertical motion is induced as in the case of the conventional structure, the studies performed in Japan have shown that inshore siting has strong seismic isolation characteristics in the horizontal directions. The drastic reduction of horizontal earthquake motion could result in a plant cost reduction and a safety margin increase.
Among in-shore siting techniques, that of a full-floating platform in a semi-closed basin was selected from the cost and safety point of view (ref. [15] and Fig. 5-5).

5.9 Effect of the seismic loading on the reactor vessel design

The seismic loads are often the largest of the primary loads and therefore determine the wall thickness of the structure based on one or more of the following criteria:—

- prevention of excessive displacements;
- plastic collapse;
- buckling.
Thus, seismic analyses have considerably influenced design of the vessels and baffles of the existing fast reactors.

5.9.1 Effects of seismic loads on the existing pool-type designs

The insufficient knowledge of buckling and fluid-structure interactions has often forced designers of pool-type reactors to adopt conservative values of the wall thicknesses, although these were in general not desirable for thermal shock low cycle fatigue resistance (ref. [2]).
For instance, in the Superphinix-1 design the seismic loads were finalized during the early phase of the construction, and they influenced the design of many components (ref. [10]). Some problems were detected due to the relatively high design earthquake levels (SSE = 0.2 g), the component sizes and fluid-structure interaction phenomena. These problems were solved by an increase of the stiffness of some structures with minimum modifications of the thermal stress.

The most significant design modification of the Superphénix-1 reactor-block concerned the core support structure, internal structures and the above-core structure. More precisely (see ref. [10] and Fig. 5-6):

- heavy reinforcements of the core-support structure were adopted to accommodate the horizontal seismic effect and to protect against a buckling failure;

![Fig.5-6: Superphénix reactor block](image-url)
stiffening rings were welded to the baffle B1 of Fig. 5-6 and the thickness of the weir cylindrical shell was increased at its upper part;

- rather thick thermal shield was used despite the adverse effect on thermal stresses.

The need of increasing wall thicknesses to accomodate seismic loads has also been found for other reactors: for instance, wall thickness of the cylindrical part of the primary vessel was increased in the CDFR project (ref. [4]).

5.9.2 Effects of seismic loads on the existing loop-type designs

The seismic problems of loop-type reactors are somewhat different from those of the pool-type plants. The size of the loop-type vessel is considerably smaller (therefore less sodium) and, typically, wall thickness is greater. For instance, the wall thickness of the main vessel of the Japanese MONJU reactor (280 MWe) is 50 mm (ref. [15]), and those of the inner and outer vessel walls of the Italian PEC reactor (120 MWe) are 30 mm and 25 mm, respectively (ref. [5]).

However, in the loop-type reactors, the presence of long suspended vessels might lead to large seismic amplifications of the core supporting structure and the primary piping connections. For instance, this was the case of PEC, which has a rather flexible vessel supporting structure in order to minimize the effects of a core disruptive accident (Fig. 5-7). Moreover, the large core mass relative to the reactor-block mass associated with the loop reactor causes strong vessel-core mechanical coupling, whose analysis is rather complicated.

In line with the concept of 'rigid' structures which has been extensively used in Japan and to the high level of design earthquakes (0.47 g SSE and 0.28 g OBE, ref. [15]), the primary vessel of MONJU which is suspended from the upper flange, has been provided with a lower support to restrain vibration of the vessel in a seismic event (Fig. 5-8). This solution moves reactor-block natural frequencies to the relatively high values which correspond to the spectrum flat zones (ref. [18]). Furthermore, a dipp plate was installed to suppress the sloshing of the sodium free-surface.
Fig. 5-7: Sketch of the PEC main vessel (without the test channel and the shutdown system)

Fig. 5-8: Schematic view of the reactor structure of Monju
The possibility of using a restraint vessel design similar to MONJU was also considered for the PEC reactor, in order to limit the core seismic response (ref. [4]). Although the main vessel restraint was not feasible for construction reasons, this solution was considered to be the most reliable to resolve the core seismic problems. Because restraining the PEC vessel was impractical, extensive on-site tests on the mounted vessel became necessary, to check adequacy of the vessel supporting stiffness values calculated in the design analysis (ref. [36]). This check was essential, due to the fact that first natural frequency of the PEC vessel, and thus, vessel rotation, are governed by such stiffness values.

5.9.3 R & D work needed to improve the design

It is generally accepted that pool-type reactor is the most promising concept for all future fast reactors. Indeed, a considerable R & D work has already been performed also in Japan, in the framework of the feasibility study of pool-type reactors (1000 MWe class) to search for a structural concept which better balances thermal stress reduction with structural integrity during earthquakes (ref. [1, 15]).

With the aim of rationalizing the reactor block design it is widely agreed that further R & D work on the following areas is needed (ref. [6]):-

- fluid-structure interaction;
- sloshing phenomena;
- buckling phenomena;
- seismic isolation.

5.9.3.1 Improvement of fluid-structure interaction analysis

Fluid-structure interaction effects are particularly significant for the pool-type reactors due to closely spaced components and the presence of perforated structures (fluid passages). Based on the results which have already been obtained, it is generally believed that the fluid-structure interaction may be positively used to reduce seismic response of components (refs. [1, 2, 15]).
On the other hand, the studies of ref. [16] also show that the effects of imperfections on the vessel response may be important and that some three-dimensional effects may be not negligible in the reactor-block response (although two-dimensional analysis generally provides conservative results).

5.9.3.2 Improvement of understanding of buckling phenomena

A better knowledge of buckling is judged essential to allow for the demonstration that thickness of the vessel walls and baffles can be reduced to lower values. Indeed, with regard to buckling due to pressure, the results of studies cited in ref. [16] show that margins exist between the dynamic and the static pressures leading to buckling. Furthermore, as far as shear buckling is concerned, the studies of ref. [14] showed that elastic bifurcation analysis was unrealistically conservative, while an elasto-plastic bifurcation or full three-dimensional non-linear incremental analysis gave satisfactory results.

Thus, present methods for buckling analysis appear conservative. However, the effects of the imperfections, and in some cases, plasticity, have to be carefully evaluated in the case of buckling due to pressure, because they may considerably affect this type of buckling phenomena.

On the other hand, the effect of initial imperfections on shear buckling is remarkably small (refs. [14, 15]). (Furthermore, the results of the Japanese studies cited in ref. [15] show that shear buckling modes are dominant even if various parameters such as dynamic loading and internal pressure were considered, and that the post-buckling behaviour on plastic shear buckling was relatively stable under repeated loads).

5.9.3.3 Improvement of sloshing analysis

With regard to sloshing, a particular attention to the problem is being paid in Japan. Special devices such as dipp plate used in MONJU have been developed and methods have been improved to demonstrate structural integrity. The presence of such devices (also demonstration that the sodium outlet nozzles are not exposed to the cover gas due to sloshing) was considered necessary in Japan because of the sloshing phenomena (ref. [1]).
Improvement of analysis methods is required for non-linear sloshing (to evaluate roof slab integrity) and especially for the vertical sloshing effects, which are less well understood. It is pointed out that sloshing effects may be important in the case of isolated reactor concepts, because of the low frequency values associated with the sloshing waves.

5.9.3.4 **Improvement of reactor-block vertical analysis**

Past experience of the large pool-type designs have shown the need to minimize the diagrid vertical excitation and differential motion between the core diagrid and the vessel plug. In fact, large vertical accelerations of the diagrid and/or large differential motions between the two structures may lead to considerable reactivity change.

5.9.4 **Development of vessel support concepts**

A number of different solutions of the vessel supporting system have been examined in Japan and many experiments have been performed for the various concepts shown in Fig. 5-9 (ref. [1]).

![Diagram of reactor vessel supports](image)

**Fig.5-9: Reactor vessel supports analysed in Japan**
For instance, the adequacy of shear keys (Fig. 5-10) as reactor vessel and guard vessel horizontal support has been demonstrated (ref. [15]). Shear keys restrain the vessel and thus, limit its seismic amplification, without leading to problems from the point of view of thermal stresses.

![Diagram of Main Vessel and Guard Vessel](image)

**Horizontal Section**

**Maximum Load Distributions of Shear Keys (Experiment and Analysis)**

Fig. 5-10: Experimental study on shear key restraint structure

Other concepts of horizontal support such as the fluid-filled gap support have been studied (ref. [1]). This type of support, however, is now judged to be more efficient for the internal components (see refs. [18, 19] and Sect. 5.13.2.2).

5.10 Impact of the seismic loads on the core design

5.10.1 Core horizontal response

The complicated core behaviour is mainly related to the earthquake horizontal components. This is due to the presence of different core element types, which are characterized by low, but different natural frequencies, and the immersion in the liquid sodium. The collisions
among the elements lead to non-linear behaviour while the immersion in fluid may lead to strong fluid coupling among the core elements or between the core and surrounding structures.

Furthermore, the core horizontal response is two-dimensional, even in the case of one-dimensional diagrid excitations. This is due to the hexagonal cross-section of the fuel and other core elements. Because of the two-dimensional effects may be rather important. They may considerably affect especially the core volume variations, and thus, earthquake neutronic and thermal effects.

5.10.1.1 Core seismic requirements

The main issues of the core seismic behaviour have been discussed in the section 4.2. From the safety point of view it must be demonstrated that the reactor can be safely shutdown during and after the SSE and that it can be later kept in a safe shut-down condition (decay heat removal must be guaranteed) indefinitely.

Thus, satisfactory aseismic design of the fast reactor core must ensure insertion of the control rods, prevent large reactivity increase (i.e. prevent melting of fuel pins in the SSE) and guarantee structural integrity of the subassemblies. The latter includes integrity of the S/A spike due to bending in the horizontal motion and integrity of the hexagonal wrappers, pin bundles and grids under the impact loadings.

5.10.1.2 Core design solutions

From the point of view of minimizing thermal stresses, the best core design solution, consists of restraining the S/As at the spikes only, thus leaving them free to expand. This is, the solution adopted for the Superphénix-1 core.

In the presence of non-negligible clearances between the element spike and the support tube, the free standing concept leads to low first natural frequencies of the core elements. This in turn could result in rather large horizontal displacements. The element displacements might be too large as to allow for demonstrating the structural integrity of the spikes, and insertion of the shutdown rods.
With regard to the demonstration of scram during an earthquake, it is important to note that, not all the reactor designs include seismic safety systems which trigger automatic shutdown (e.g. manual is used in the U.S.A., F.R.G. and U.K., see ref. [21]). The probability that scram triggering occurs during (and not after) the earthquake is high. In fact, in the case of large earthquakes, the induced neutronic and thermal effects are almost certain to cause the intervention of other safety systems such as reactivity or power monitoring systems which will initiate the shutdown very rapidly (within 1 s for PEC, see ref. [22]).

The necessity of a reliable safe shutdown system forced the Japanese to use a core barrel restraint for both JOYO and MONJU reactors (Fig. 5-11). The presence of this restraint system, together with the small gaps between the core elements even in cold conditions, limits the core element motion and increases core response frequency. The latter feature is consistent with the general concept of adopting the 'rigid' structures, used in Japan.

![Diagram](image)

**Fig.5-11:** Core restraint system of Monju: axial levels of possible interactions with the core elements – schematization for the numerical analysis
The core barrel restraint has also been developed for the CDFR reactor for static loads associated with swelling wrapper materials and seismic loads (ref [24], Chapter IV).

5.10.1.3 The PEC case

The case of the PEC reactor is a typical example of how the seismic loads may influence core design. As already mentioned in Sect. 5.4, a core-restraint ring and thick pads, attached to all the core elements at two axial levels, had to be adopted for PEC to allow for the demonstration of scram feasibility (Figs. 5-7 and 5-12). The gaps between the adjacent elements and between the core and the restraint ring are small but non-zero even at full power. Also, all the element spikes had to be redesigned. The reasons were again the severe design earthquakes, but also the use of a rather flexible vessel supporting structure, which leads to large amplification of the seismic motion at the core diagrid level (ref. [23]).

Fig.5-12: Sketch of the PEC fuel element

For the problems stated in Sect. 5.9.2, the vessel-restraint systems could not be used for the PEC reactor. The large vessel amplification resulted in large impact forces acting on the core elements and causing compaction of the fuel elements due to the collisions against the restraint ring.

The first modified PEC core design limited pad thickness to the minimum values which had been found necessary to guarantee scram feasibility. The reason was not to complicate demonstration of the feasibility of fuel element handling (the PEC transfer machine had been designed as to only allow for rather limited extraction/insertion loads).
However, the non-negligible gaps which were still present between the fuel elements at the contacting pads were later found to be excessive, because they permitted unacceptable core volume variations i.e. a high reactivity increase.

Thus, a further increase of pad thickness at both pad levels was necessary for the PEC fuel elements. On the contrary, thickness was decreased for the neutron shield elements, because it was found that gap increase at the core periphery decreases compaction (ref. [23]). The latter change helped handling, without increasing control rod displacements with respect to the first design modification.

A significant contribution to core volume variation was also provided by the fuel element wrapper deformation. Therefore, the final design also included large stiffening (by a factor 3) of the fuel elements at the upper pads. This also helped to demonstrate wrapper structural integrity under impact loads.

The above-mentioned modifications of the PEC core design made it necessary to recheck the fuel element handling feasibility and the margins in the transfer machine capabilities, by test and analysis. They also influenced element handling strategies. Furthermore, the introduction of the core-restraint ring and the element modifications made it necessary to reevaluate the whole reactor-block seismic analysis and to repeat many experimental tests. The effects of these modifications on cost is self evident.

5.10.1.4 Need for further R & D

It is noted that the lack of validated two-dimensional methods to calculate the core volume variations in the case of the restrained core forced the designers to evaluate the SSE neutronic and thermal effects by use of very conservative method. To determine maximum reactivity insertion, this model uses static radial (i.e. two-dimensional) compaction of the fuel elements at the two pad levels in the most outward bowed condition during the core life (refs. [23, 29]). The problems arisen for PEC thus might be sufficient alone to stress the need for further R & D aimed at providing very reliable and accurate core analysis methods, to allow for an optimized design without undue conservatism.
However, improvement of core analysis methods is also of great importance in other respects. For instance, hexcan integrity is a major requirement to ensure element cooling and limited displacements and core volume variations, and because collisions might also occur at axial levels where embrittlement by irradiation of the fuel element materials might be significant (ref. [24]). Furthermore, reliable evaluation of acceleration peaks (mainly due to shocks) might also be important for fuel pin bundle structural verification at OBE (ref. [22, 39]).

The improvement of the horizontal seismic analysis is needed for the restrained core, because its seismic behaviour has not yet been systematically investigated and is not well understood at present. Also the impact loads and the reactivity insertion - whose values may be significantly enhanced in restraining the core - are more difficult to evaluate with the necessary precision. Calculation of absolute displacements, used for the demonstration of element structural integrity and scram feasibility, presents no problem.

In the case of the naturally restrained core, the behaviour is better understood. Displacements are the dominant response parameters in this case. However, improvement of knowledge on fluid-coupling effects among elements as well as among the core and the other vessel internals is probably more important than in the restrained core, where - in some cases at least - the presence of the core barrel dominates the seismic response (see, for instance, ref. [25]).

Finally, structural and functional verification of the core elements also needs further experimental work to assess criteria which are applicable to the fast reactor specific features: this need has been stressed by ref. [39].

5.10.2 Core vertical response

With regard to the vertical core response, seismic safety requires that lifting of the core elements must be prevented. This limits diagrid acceleration peaks to the values which are compatible with the hydraulic hold-down features.
Furthermore, the vertical displacements must be limited in order to avoid large contribution to reactivity insertion due to control rod extraction during the earthquake motion. This requires a sufficiently high stiffness of the core supporting structure, and in general, limited diagrid-plug differential displacements.

The two above-mentioned problems are significant only for the large pool-type reactors. The relatively lower sizes of the loop reactor vessels entail low reactivity insertion values due to the vertical motion, because of the rigid diagrid and roof plug. In the SSE analysis of PEC, for instance, the maximum diagrid acceleration in the vertical direction was lower than 0.5 g and the contribution to reactivity insertion given by control rod extraction was about 1/40 of dollar (see refs. [22,29]).

The need for limited values of the core motion in the vertical direction may considerably affect design of the large pool-type reactors in the high seismicity countries. In Japan, for instance, the permissible relative displacement (15 mm) between the core and the upper internal structure was found to be the dominant factor in relation to the vertical movement (ref. [1]). Thus, this was one of key points on which the reference design of the future Japanese large scale plants was based.

Another item which might need some attention is the potential compaction of the fuel pellets due to vertical acceleration in some plant operating conditions, in which gaps are present between these pellets. The effect of such a compaction may contribute significantly to reactivity insertion. For example, in the case of PEC, Italian licensing asked for analysis of the effects of a simultaneous sudden compaction of 50% of the pins, added arithmetically to maximum reactivity insertions due to the radial core compaction and control rod extraction.

The request was due to the fact that probability of pellets' compaction could not be assessed at the time, and that no reliable models existed to describe the phenomenon. The PEC analysis demonstrated that no fuel melting occurs even in this very unrealistic case, because gaps between pellets are only present in some plant conditions, which offer wider margins with respect to the normal operating conditions at full power. However, the definition of more realistic approaches might be of interest for future plants.
To conclude, it is worth observing that most of the problems which might be related to the vertical response can be only solved by reducing seismic input to the core. Indeed, core element design itself cannot provide significant contributions to optimize core vertical response.

5.10.3 Development of core support structures and arrangements

To optimize the core design, various core support structures and arrangements have been analysed and proposed for the future large-scale plants. A particular effort has been made again in Japan, basing on a long series of experiments (refs. [1, 15]).

Core supports from the roof slab, side wall and bottom head (Fig. 5-13) have been analysed in the first feasibility studies, which confirmed the validity of design for roof-slab and led to the proposal of large scale pool-type LMFBR with upper and lateral core supporting arrangements (ref. [15]). In the following study phase, devoted to rationalization of aseismic design, the feasibility of bottom support arrangement was confirmed (Fig. 5-14).

![Core Support from Roof Slab, Side Wall, Bottom Head](image)

Fig.5-13a: Concept of support structures (Japan)

5.11 Impact of the seismic analyses on the shutdown system design

5.11.1 Main problems of the analysis

The seismic analysis of the shutdown systems which are used in the fast reactors is rather difficult due to a highly non-linear behaviour in the horizontal plane, which is caused by the collisions of the various inner parts and the core, as well as by the vertical motion of the
Reactor Structure (Core Support from Roof Slab)

Reactor Structure (Core Support from Side Wall)

Fig. 5-13b: Concept of support structures (Japan)
control rod absorber. Furthermore, the analysis is complicated by strong sodium effects leading to fluid-coupling and by the fact that different seismic motions are applied to the system at various axial levels.

The axial levels to be considered, correspond to the points where the system is connected to the reactor-block structures and to those where interactions with vessel internals may occur (for instance, the core, see Sect. 6.3.4 and Fig. 4-39 and 5-16). If scram takes place during earthquake, which is very likely, the system stiffness varies with time during the control rod drop.
Due to system highly non-linear behaviour the demonstration of scram feasibility requires testing, which has to be performed on full-scale mock-ups. Furthermore, the mock-ups must correctly describe the real system, at least the parts which affect the seismic response. This made it necessary, for instance, to use a real primary shutdown system in the seismic verification of Superphénix-1 and a quasi-prototype mock-up for PEC (refs. [26, 27]).
5.11.2 Limits of the experimental analysis

Even full scale tests do not provide a full qualification of the shutdown systems. The reason is that the number of seismic tests at the highest excitation levels (SSE) is limited, in order to minimize damage to the system (the high cost usually limits the tests to one test for each of the different system types which are present in the reactor). Indeed, the shutdown system must resist only one SSE, according to the seismic safety requirements.

On the other hand, the effects of many parameters should be examined to guarantee seismic safety. This would imply several SSE tests.
The parameters may be, for instance, the time at which scram is triggered and the initial axial position of the absorber (refs. [26, 27]). These affect the stiffness of the system, and in general the non-linear dynamic behaviour, during the earthquake.

Also time-histories which are applied to the system at the various axial locations are affected by uncertainties. In fact, they result from numerical analysis of the reactor-block and in particular the core. The uncertainties affecting phase difference might be particularly significant. Several SSE tests would be necessary for a detailed experimental evaluation of these effects.

Furthermore, the effects of the two-directional nature of the seismic excitations in the horizontal plane should be studied. The vertical component might also have some effects. In the tests carried out up to now, only one-directional, simultaneous, horizontal excitations were used at the different axial levels.

Finally, experiments cannot certainly account correctly for certain reactor conditions and parameters, such as temperature and friction coefficient in sodium. These items may have a significant effect on system non-linear behaviour, and especially insertion of the control rod absorbers.

5.11.3 Usefulness and limits of detailed numerical tools

Because of the reasons mentioned above, reliable numerical tools are also necessary to prove seismic safety of the conventional shutdown systems. These tools must be fully qualified by experiments, and should allow for the correct calculation of structural response and drop time.

At the time being, however, most of the computer codes which have been developed do not fully model the vertical movement (ref. [24]). This is further discussed in section 4.3.

But in spite of the possible improvements of computer codes, it is doubtful that it will be feasible, to avoid full scale expensive experimental qualification in support to the numerical studies, to demonstrate safety of the present shutdown system concepts (ref. [14]).
The complexity of the problem would obviously be much reduced if design concepts such as seismic isolation are adopted, or in the case that innovative shutdown system concepts, less affected by the seismic conditions, become available (such as the ASD system proposed for CDFR, see Sect. 5.11.4).

5.11.4 Perspectives of new design solutions

As mentioned, the use of seismic isolation systems, as proposed for the U.S. PRISM project (ref. [30]) may also solve the problems related to the demonstration of scram feasibility in an earthquake.

However, the use of alternative shutdown system designs could provide a solution. We remember that an emergency, articulated, shutdown system (SAC) already exists in Superphénix-1, beside the conventional primary system (ref. [26]). The seismic qualification of the SAC system showed that it is not sensitive to the seismic excitation (the drop time increases by 10% at SSE level).

Also the development of advanced systems, with inherent safety features, is of interest e.g., the Alternative Shutdown System (ASD) which was developed in the U.K. (ref. [31]) for the CDFR project. The main objective of the ASD is to provide completely divergent absorber rod and mechanism system for all shutdown conditions including an earthquake.

The ASD channel is fully contained within the core boundaries (Fig. 5-17). There is no structural coupling between the ASD channels, above-core structures and rotating shield. Thus, the seismic qualification of the ASD is certainly much easier than that of the conventional primary systems (tests have already been performed with encouraging results, see Sect. 6.3.4.5).

The ASD concept was discussed at the IAEA/IWGFR meeting of Bologna (ref. [24]). Although the concept was considered to be very attractive from the seismic point of view, concern was raised at the meeting about the lack of operational experience, in a prototype reactor, of this system.
5.12 Impact of the seismic loads on the piping design

5.12.1 Problems and possible design improvements

A major concern of the nuclear power plant industries has been the increased rigidity of the piping systems (resulting from seismic load requirements) and proliferation of pipe supports. It is generally
recognized that the use of current design codes leads to too many snubbers, which are costly to install, reduce access and make in-service inspection and maintenance more difficult and costly. Also the overall system availability due to relatively lower reliability of the snubbers (refs. [2, 3, 14]) is reduced.

The need of improving technology in the piping area – i.e. the need for more effective design methods and criteria is recognized. Studies aiming at improving piping technology include:

- the assessment of damping data which are less conservative than the values required by the codes;

- the definition of less onerous design criteria (e.g. strain criteria);

- the development of new or better methods of predicting the dynamic response of systems to inertial loads, which also use the inelastic properties of piping and associated components;

- the development of new concepts, such as expansion joints.

The technology improvements will allow the designers to justify a much reduced number of supports and snubbers and a higher flexibility of piping systems. The related design solutions will be more adequate especially for the fast reactor thin-walled piping.

5.12.2 Damping data

A damping is an important parameter for piping seismic analysis. The data obtained in various countries show that damping values specified by U.S. N.R.C. Regulatory Guide 1.61 are in general conservative (see, for instance, refs. [2, 3, 14]). Heavy insulation of large bore LMFBR piping leads to a further increase of damping.

The assessment of a complete data-set of the existing damping data might allow for a more realistic analysis of piping systems.
5.12.3 Use of strain limits

Piping systems, especially the more flexible small bore piping, have large seismic reserve strength margins, due to material non-linearities and energy absorption (refs. [2, 3, 7, 14]). But the present design methods, beside treating damping conservatively, use elastic analysis.

Experiments performed in the U.S.A. and the F.R.G. (refs. [2, 3]) show no piping collapse, leak or pressure loss failures for LWR piping systems at extreme loads up to 300-400% of the design code limit. This demonstrates a significant capacity beyond the elastic capacity. Although further tests, specific to the fast reactor piping, are still necessary to confirm these results in the case of the thin-walled piping, the use of strain limits instead of the presently adopted stress-based criteria is suggested (refs. [2, 3, 7]).

This proposal has been discussed at the IAEA/IWGFR meeting of Bologna (ref. [32]). The general conclusion was that the adoption of strain limits looks justified and might enhance the overall safety level of fast reactor piping. However, the need of a careful Quality Assurance on piping components and welds was also pointed out at the meeting, to prevent dangerous effects of flaws, scratches, etc., if the design relies on ductility.

5.12.4 Improvement of analytical methods

Piping is subjected to different seismic excitations at the different restraints and supports. The extensive use of time-history methods, although rather expensive, and the use of improved multisupport excitation methods, may help to reduce overconservatism. This is especially true if the advantageous effect of the inelastic piping behaviour and non-linear support behaviour is to be realized (present response spectra method is based on linear models and damping values consistent with elastic response and small motions).

Other improvements include the simplified methods under development in the F.R.G. (ref. [33]) and the definition of reliable methods for the evaluation of non-linear support effects (on this item, simplified
linearized stiffness models have already been defined, for instance, in Japan, see ref. [15]).

5.12.5 Development of expansion joints

Development of expansion joints is considered as an important item for the optimization of the piping design of the future fast reactor piping design in the U.S.A. and Japan. In the U.S.A. gimbaled expansion joints are being developed for the PRISM project. These joints are necessary for piping connecting the seismically isolated reactor-block to the non-isolated parts of the reactor plant.

In Japan, great effort is devoted to piping bellows expansion joints (Fig. 5-18). They are judged to be a very promising solution, which enables volumes to be significantly reduced (ref. [18]).

![Diagram of a primary heat transport system with bellows expansion joints](image)

Fig. 5-18: Primary heat transport system with bellows expansion joints

5.12.6 Need for further tests

Further tests are needed to better characterize the piping restraint elements and to demonstrate adequacy of new concepts such as expansion joints. Furthermore, on-site tests may provide very useful information on actual behaviour of piping systems in the real restraint conditions. Finally, fragility tests, such as those carried out for the
LWRs, may also help to confirm the large seismic reserve strength margins above the present design basis requirements, and thus, to fully take advantage of the apparent overconservatism of the present design methods.

5.13 Impact of seismic analyses on the design of other components and equipments

5.13.1 Effects on the existing designs and projects

In addition to the design modifications discussed in the previous sections the seismic loads influenced the design of the following components (ref. [10] and [4]):

- Support of the Superphénix-1 intermediate heat exchanger (IHX), to reduce thermal stresses and increase flexibility;

- A complete redesign of the Superphénix-1 storage drum which supports the fuel elements in the fuel storage tank was necessary in order to increase its seismic resistance and stiffness.

Furthermore, with regard to the CDFR project, the relatively large SSE loads used in the design made it necessary (according to ref. [4]):

- Provision of a bottom support of CDFR IHX to reduce seismic stresses.

- To change the support of the primary pump from encastre to pinned support accomplished either by spherical joint or diaphram. Also the bottom end of the pump undergone design changes involving lateral support.

- To decrease sodium bearing clearances.

- To include support spokes perpendicular to the main girder beams in the design of the component handling polar beam machine.
5.13.2 Need for further work

5.13.2.1 Improvements for pump dynamic analysis

A reliable operation of the primary and secondary pumps is fundamental to economic and safe operation of a fast reactor plant. Most of the prototype reactors experienced vibration problems and failures of the sodium pumps. The various loading conditions and phenomena which could result in unacceptable oscillations need to be understood and resolved for the full commercial fast reactor.

From the safety point of view, it is essential to demonstrate that a common failure such as seizure of sodium bearing cannot occur at any operating speed as a result of SSE.

To this aim, it is necessary to develop and validate numerical methods for rotor dynamics under various operating conditions and also transient loadings resulting from an earthquake. Dynamic and seismic tests of pumps in sodium (besides tests in water) might be useful for contributing to the validation the numerical methods. Such tests have already been performed on the Superphénix-1 primary pump shaft at the ENEA Center of Brasimone (ref. [34]).

5.13.2.2 Development of systems reducing component seismic response

Development of new systems which are capable of reducing the seismic response of components may also be of interest. This is especially the case of so called "fluid-filled gap support system", which is being developed in Japan (ref. [19]).

This is a hydrodynamic support system, which consists of inner cylinder and outer restraint cylinder, with a small gap filled with fluid (Fig. 5-19). In the Japanese pool-type LMFBRs this system is considered to be efficient as a seismic support system for vessel internal components, such as pump, IHXs, core, etc., which are suspended from the deck, because this support system has no mechanical connections with internal components and accommodates the thermal deformations without causing thermal stresses. The problem is to suppress axial flows in the annular gap. It was proven that this can be obtained using seal rings.
Fig. 5-19: Vibration characteristics of the fluid filled gap support system
Obviously, reliable fluid-structure interaction methods are essential to evaluate the seismic response of this kind of systems.

5.14 Usefulness of advanced seismic monitoring systems

The use of advanced seismic monitoring systems can substantially contribute to the reactor safety.

In all the fast reactors seismic monitoring is provided by means of suitable safety and/or alarm systems. A threshold level is fixed, at OBE or lower level. In the case of earthquakes exceeding this threshold, the reactor is either shut down or alarm is raised. As already mentioned, fast reactor shutdown is automatic, triggered by the safety system, in France, Italy, Japan and the U.S.S.R., while it is manual, triggered by the operator, the U.S.A., the F.R.G. and the U.K.

For instance, in the case of PEC, the safety system consists of three triaxial seismic monitors (located at the ground surface in the free-field), which are triggered if a threshold value of the ground acceleration is exceeded (refs. [23, 35]). The signals from the seismic instruments are processed by the safety system (with a 2/3 logic) and the shutdown is triggered.

As far as the threshold level is concerned, values being considerably lower than those corresponding to the OBE are used for all the fast reactors in the European Community countries. This leads to a further enhancement of safety. More precisely, the threshold level corresponds to 1/4 SSE for PEC and SNR-300, to 1/8 SSE for Superphinix-1 and to the "Operational Shutdown Earthquake" (OSE, corresponding to 0.05 g peak acceleration) for CDFR (ref. [4]).

For PEC a very advanced seismic monitoring system has been foreseen, besides the automatic safety system (ref. [35]). The main reason for using this system is that the safety system cannot provide any information with regard to earthquake frequency content, due to the fact that peak acceleration is the parameter used in that system to quantify the seismic event.
### Instrumentation of the PEC seismic monitoring system

**Fig. 5-20:** Instrumentation of the PEC seismic monitoring system

The PEC seismic monitoring system, consists of 20 triaxial accelerometers, located both in the free-field and on the plant structures (Fig. 5-20). The positions on the structure have been defined taking advantage of the detailed numerical and on-site experimental studies (see Section 6.8.3.1).
Measured data are digitized and recorded. From the recorded time-histories the floor response spectra are calculated and compared with the design values. The elaborations and comparisons are performed in "short time" (i.e. quasi-real time) for two triaxial measuring positions, thus allowing the operator to immediately get a more complete information on the seismic event.

The complete set of data recorded by the seismic monitoring system also allows the actual dynamic response of the plant to be determined and compared to the design values. On the basis of this comparison, the necessary safety analysis would be possible after a large earthquake, to verify whether the design limits of the plant have been respected: in the positive case, reactor restart is possible.

The development of such seismic monitoring systems might be useful for all the nuclear reactor types. But especially for the fast reactor where seismic loading plays more important role in design and safety.

REFERENCES


6. EXPERIMENTAL ANALYSIS

6.1. Introductory remarks

It has been mentioned that extensive experimental studies are necessary for the demonstration of seismic safety and reliability of the fast reactors. While for the thermal reactors tests are usually limited to the equipment seismic qualification, in the case of fast reactors complicated tests are also needed to validate the new numerical methods developed for the specific features of this plant type.

The main items for which development and validation of methods has been necessary, are the main vessel structures, the core, shutdown systems, piping and pumps. Furthermore, improvements in the nuclear island analysis was also considered important. The latter improvements take advantage from the on-site testing.

This chapter deals with the items of interest in the experimental seismic analysis of the fast reactor structures, components and systems. To this aim, some information is given on the experimental devices used for seismic analysis, mock-ups concepts, instrumentation, excitation types, data treatment and seismic qualification of equipment.

To illustrate this, examples of experiments performed on some major fast reactor components and systems are reported based on the papers presented at the IAEA/IWGFR Specialist Meeting on Fast Breeder Reactor Block Antiseismic Design and Verification held in Bologna, Italy, in October 1987. A more complete review of the recent tests is contained in the IAEA Report IWGFR-65, which contains the meeting proceedings.

Finally, this chapter contains some comments on on-site experiments, based on the work discussed at the ENEA/ISMES/ENS International Specialist Meeting on On-Site Experimental Verification of the Seismic Behaviour of Nuclear Reactor Structures and Components, held in Bologna and Brasimone in May 1987.
6.2 Seismic test procedures and experimental devices

6.2.1 Types of test procedures

Three types of test procedures are used in the framework of the seismic experimental activities (ref. [1]):

- Tests to provide understanding of structural behaviour. These tests are carried out to identify the parameters which characterize the structural behaviour, in order to allow suitable computer codes to be calibrated and validated.

- Qualification tests. These aim at verifying and thus, guaranteeing, that the structure under study remains functional within design limits, when subjected to the design dynamic loads which are foreseen during its life in the reactor.

- "Dynamic characterization" tests. This is a new experimental field, which has the final purpose of structural monitoring during time.

6.2.2 Excitation techniques

The different types of test procedures often use different excitation techniques (ref. [1]). These are:

- shaking tables;
- concentrated excitations;
- excitation through the foundations.

A description of the main seismic test facilities used in the countries involved in the European co-operation agreements for fast reactor development were published in ref. [1] as an example, the information concerning the French and the Italian facilities is given in Figs. 6-1 and 6-2.

Each of the excitation techniques is characterized by a well defined application field, which depends on test features and structure sizes. The most significant aspects of the three techniques are reported below.
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transports ...

MOYENS :

- 4 tables vibrantes dont une de 100 t biaxiale
- mur de réaction (H = 4 m)
- fosse d'essai (H = 15 m)

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<th>AZALEE</th>
<th>VESUVE (existent)</th>
<th>TOURNESOL (existent)</th>
<th>MIMOSA (existent)</th>
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<td>12</td>
<td>12</td>
<td>27</td>
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Fig.6-1: Facilities existing or under construction at CEA/DEMT, CEN Saclay
### Shaking Tables

<table>
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<tr>
<th>Laboratory</th>
<th>Master</th>
<th>Sister</th>
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<th>Micratable</th>
<th>&quot;Discono vibraite eletrico-hydraulic&quot;</th>
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<td>0 - 120</td>
<td>0 - 60</td>
<td>1 - 100</td>
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<td>X Z simultaneous</td>
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<td>81.9 - 57.5</td>
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<td>Frequency range (Hz)</td>
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<td>0 - 60</td>
</tr>
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<td>Excitation type</td>
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<td>Sinusoidal Random Transient</td>
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</table>

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Fig.6-2: Examples of shaking tables in Italy
6.2.2.1 Tests on shaking table

This type of equipment makes it possible to reproduce prescribed motions at the base of the structure under test. In some cases, this is made simultaneously in several directions. For instance, the 30 ton MASTER shaking table, which is the most recent and largest table of ISMES in Italy, allows for simultaneous excitation of all the six degrees of freedom (the three translations and the three rotations). Furthermore, the new 100 ton shaking table at CEA in France (AZALEE), will enable the two horizontal directions to be simultaneously excited (ref. [1]).

This technique has the following limitations: the sizes of the structure which can be tested, the fact that shaking table provides a rigid motion at the base of the structure and the fact that, in the actual reactor conditions, seismic excitation is transmitted to other components not only across the base, but also through other restraint points.

With regard to the limits of the size of the structure, it is pointed out that the Japanese philosophy emphasizes large-scale shaking table tests, such as those now being performed on the TADOTSU 1000 ton table (ref. [2]). These large-scale tests are also judged important from public relation point of view to show that nuclear plants are safe even in a high seismicity country like Japan. On the contrary, the approach used in other countries is to validate sophisticated numerical tools based on the results of shaking table tests performed on scaled models.

6.2.2.2 Tests with concentrated excitation

This technique is based on the simultaneous utilization of several actuators, which are generally applied to the structure at the restraint points. Such actuators allow the prescribed time-histories motion to be reproduced at different restraint points. By this technique, many of the disadvantages of shaking tables can be solved. On the other hand this technique is related to the maximum power available, which frequently limits the experiments to the elastic regime.
6.2.2.3 On-site test techniques

It was agreed at the mentioned ENEA/ISMES/ENS Specialist Meeting, that on-site tests constitute useful complementary techniques with respect to shaking table experiments or to the laboratory dynamic experiments (ref. [2]).

In fact, laboratory tests generally allow much higher excitation levels to be attained at reasonable costs, but the appraisal of the effects of some important parameters (boundary conditions of components, soil-structure interaction, aging, etc.) is not always possible. On the contrary, on-site experiments, while usually limited to very low excitation levels (if they are performed on real reactor structures), enable the actual structure conditions to be tested. Four on-site excitation methods are identified:

- vibrations generated by explosives (in bore - hole);
- mechanical shakers;
- hydraulic actuators at the reactor foundations;
- real earthquakes on actual reactors or scaled models.

Blasting in bore-hole enables important information on soil-structure interaction and building – to – building coupling to be obtained. However, the correct use of this technique is not always easy, and significant improvements are still necessary. Some problems are related to the fact that the dynamic excitation which can be obtained is usually of very short duration compared to an earthquake and have rather high frequency content.

The advantages of mechanical shakers, including eccentric mass vibrators, are useful to component testing, because they allow for a correct description of the boundary conditions. Some practical problems may occur in using this technique for tests in a reactor. Indeed, there might be space limits for the installation of a large vibrator in a suitable position, especially if the reactor is already in operation or when construction is in an advanced stage. Furthermore, interference with mounting and other activities inside the reactor could make testing difficult during the construction. Finally, access difficulties to certain parts of the reactor exist when it is already in operation, due to excessive doses.
Concerning the application of real earthquakes, tests on scaled models in high seismicity areas, such as the EPRI tests in Taiwan (refs. [3, 4]), are of a particular interest, not only due to the nature of excitation source, but also because of the very severe levels applied. Data obtained from the recordings of seismic monitoring systems on actual reactors, although taking advantage of the realistic environment, are usually limited to rather low excitations and unfrequent events.

A very promising excitation technique is the use of hydraulic actuators at the reactor foundations (ref. [2]). This technique was used for the first time in the framework of the PEC analysis (see ref. [5] and Section 6.8.3.1).

This type of excitation can provide rather large dynamic forces and allows the reactor dynamic response to be analysed in detail at any time including when the reactor is already in the operation phase. Furthermore, the use of this technique requires devices which may be easily installed in the reactor by adopting suitable and cheap design solutions provided the installation was conceived at the design stage.

6.3 Scale models and examples of mock-up tests

6.3.1 Full-scale mock-ups

The purpose of the seismic experiments on fast reactor structures is often both to validate numerical techniques and to obtain reliable data for design calculations. The seismic behaviour of some major safety-related components and systems is often highly non-linear and therefore it is necessary to perform tests using full scale mock-ups, in order to obtain reliable design data and/or to provide the experimental contribution necessary for the structure seismic qualification.

Even for the full scale tests, some parameters such as temperature conditions and some effects related to the fluid nature (for instance, friction) cannot be reproduced. Typically, tests are performed in water at room temperature. Water represents the liquid sodium rather well, because the densities of the two fluids are very similar (see Sect. 6.3.2.4).
Two typical examples of fast reactor structures for which full-scale tests are necessary are the core and the shutdown system. For other components such as sodium pumps, full-scale tests are needed for seismic qualification (see Sect. 5.13.2.1). Examples of tests performed on full-scale mock-ups of the core elements, shutdown systems and pumps are reported in Sects. 6.3.3 - 6.3.5.

6.3.2 Assessment of reduced-scale mock-ups

6.3.2.1 Need for scale reduction

Many fast reactor structures are too large as to be tested in full scale. This is due to the size limitation of the shaking tables and the other laboratory excitation devices (Sect. 6.2.2). Thus, the experimental validation of the numerical methods requires reduced-scale mock-ups for these structures. Some of the structures of interest, for example vessels, are less non-linear than others e.g. the core elements and the shutdown systems. Or, the tests are needed for analysis validation, but not to provide design input data.

For example, seismic test of a whole core in full scale is practically impossible (it might be possible only on a very large shaking table, such as the Japanese Tadotsu). But the assessment of the data necessary for the core seismic verification may be obtained from the tests performed on a limited number of elements tested in full scale. Tests on a whole core in reduced scale, such as the French Rapsodie mock-up (Sect. 6.3.6), is essential to validate the numerical models.

6.3.2.2 Comments on the similitude laws

In the case that reduced-scale mock-ups are used, the first problem is how to interpret the phenomena observed and the quantities measured on the mock-up. The change of scale creates distortion of several different phenomena. If not enough care is taken, the mock-up behaviour may strongly differ from the behaviour of the actual structure and results may not be relevant.

The respect of the similitude laws consists in conserving in the mock-up the ratios which exist in the actual structure between the seismic load and the different forces associated to the various
fluidelastic phenomena. It is never possible to respect all these quantities at the same time. Therefore, one must preserve the similitude of the important phenomena occurring in the case under study.

This points out the fact that the experimental methodology is also affected by uncertainties, similar to the analytical procedures. Indeed, analytical and experimental tools are complementary. On one side the numerical results must be validated experimentally, but on the other side the measurements must be interpreted by use of computer programs.

It should also be noted that the exact conservation of the ratios of the various forces is frequently not needed: in practice, certain distortion of these quantities is admissible. Sometimes, it is better not to complicate the mock-up concept by trying to respect inessential similitude laws. For instance, the introduction of added masses, whose fixation is not well controlled, or the use of not well known materials, should be avoided.

A detailed discussion on how to treat scaling problems is already contained in the IAEA Technical Report of ref. [17] with regard to the problem of flow-induced vibrations. We can thus make reference to this report, since the problem of seismic analysis has many aspects in common with that of flow-induced vibrations.

The main difference is that external forces i.e. the seismic loads, are applied to the fluid-structure system. These loads are the dominant excitation forces. They can be dimensionally expressed, using a nomenclature similar to that of ref. [17] for the structural parameters, as \( \rho_t^2 \omega^2 L^3 l^1 \). In this expression \( \rho_t \) is the mean density of the fluid-structure system, \( \omega \) is the pulsation of the seismic load, \( L \) is the characteristic length and \( l \) is the amplitude parameter characterizing the input seismic excitation.

Furthermore, with regard to the fluid forces, the scaling problems are usually easier in the seismic case than in that of the flow-induced vibrations, because the inertia forces associated with fluid vibration motion are generally much more important than those related to the fluid permanent flow. Also other effects, like surface tension or especially, fluid compressibility, are usually negligible. Only the gravitational forces are important in the case of fluid sloshing.
In the following remarks we assume, as in ref. [17], the respect of the geometric similitude - characterized by the scale factor \( \lambda \) - although in some cases different approaches may be convenient or necessary. For instance, in most shell problems structural stiffness corresponds either to bending a shear modes: in the related mock-ups the ratio \( h/L \) (\( h \) = shell thickness) may be changed if the Young's modulus is varied by using a material different from the actual structure material (see the example of Sect. 6.3.7).

### 6.3.2.3 Similitude laws for the structure

As far as the structure is concerned, let us indicate the density as \( \rho_s \) and the Young's modulus as \( E \). The need of conserving the ratio between the elastic forces (\( EL_1 \)) and the seismic loads implies the conservation of \( (E/\rho_t)/((L\omega)^2) \). If the mock-up uses the actual materials (if the ratio \( E/\rho_t \) is not changed), this means that the time contraction of the seismic excitation must be proportional to the geometrical scale factor \( \lambda \) (\( \omega \) must vary proportional to the inverse of \( \lambda \)).

This also means that, if we assume equal values for the amplitude parameter characterizing the input excitation and the geometry characteristic length (\( 1 = L \)) and we conserve the ratio \( E/\rho_t \), the mock-up velocity remains equal to the actual value. Indeed, the velocity dimensional expression is equal to \( \omega L \).

The criterion for conserving velocity is often judged important in order to preserve the actual dissipative effects. It also enables to vary displacements proportional to the geometrical scale factor \( \lambda \), if the similitude laws are all respected (in this case, the displacement dimensional expression is equal to \( L \)).

Furthermore, we note that the dimensional expression of the input acceleration is \( \omega^2 L \). Thus, the conservation of the ratio \( E/\rho_t \) (which implies \( \omega L = \) constant) and the assumption \( 1 = L \) mean that the input seismic acceleration amplitude must be increased proportionally to its frequency, i.e. proportionally to the inverse of the geometrical scale factor \( \lambda \).
With regard to the conservation of the ratio between the structure inertia forces \( (\rho_S L^3 \omega^2) \) and the seismic load, we note that this is achieved by keeping the ratio \( \rho_S / \rho_L \) constant. This means that the structure density must vary proportionally to the overall fluid-structure system density.

6.3.2.4 Similitude laws for the fluid

Only the case of negligible fluid flow effects is considered here. The methodology for dealing with the fluid flow effects is contained in the mentioned IAEA Technical Report of ref. [17]. Furthermore, an example of application of similitude laws to account for fluid flow effects is reported in Sect. 6.3.3.4.

For the purpose of the fluid-structure interaction phenomena, let us define the fluid density as \( \rho_f \) and the gravity acceleration as \( g \), as in ref. [17].

The conservation of the ratio between the fluid inertia forces due to vibration \( (\rho_f L^3 g) \) and the seismic loads implies that the quantity \( (\rho_f / \rho_L) \) must be conserved. If the ratio concerning the structural inertia forces is also conserved, this means that the fluid-to-structure density ratio \( (\rho_f / \rho_S) \) must be kept unchanged.

This is true, for instance, if the actual materials are used for the structure and fluid in the mock-up. This is approximately true in the case that liquid sodium is replaced by water, and if the actual structure materials are used.

In the case of sloshing experiments, the fluid gravity effects play an important role. Thus, the ratio between the gravity force \( (\rho_f L^3 g) \) and the input seismic load must be conserved. The gravity acceleration being constant, this means that the quantity \( (\rho_f / \rho_t) / (L \omega^2) \) must be conserved. If the fluid-to-structure density ratio remain constant, \( L \omega^2 \) must also be kept constant. Because the dimensional expression of the input acceleration is exactly \( L \omega^2 \), the consequence is that this acceleration must remain unchanged.

Also in the analysis of the sloshing problems \( L \) is usually set equal to \( L \). In this way, the vertical displacement of the liquid free-surface varies proportionally to the geometrical scale factor \( \lambda \), if the ratio
between the fluid inertia forces \( (\rho_f^3 L^2 \omega^2 L) \) and the input seismic load is conserved (i.e. if \( \rho_f/L \) is kept unchanged).

However, we note that the condition \( L \omega^2 \) = constant is not compatible with that entailed by the simultaneous conservation of the quantity \( E/\rho_t \) and the ratio between the structure elastic forces and the input load (\( \omega L \) = constant). This means that, if both gravity and elastic forces are important and if \( L=L \), the velocity cannot be kept constant, and furthermore, the materials have to be changed so as to allow the ratio \( E/(\rho_t L) \) to be kept unchanged.

In the case that \( L \) is set equal to \( L \), the above-mentioned quantity is proportional to the square of the ratio between the fluid-structure system frequency \( f_t \) and the sloshing frequency \( f_g \) (ref. [17]). Indeed, \( f_t \) is proportional to \( (E/\rho_t L^2)^{1/2} \), while \( f_g \) is proportional to \( (g/L)^{1/2} \) (i.e. to \( L^{-1/2}, g \) being constant).

We remember that the condition \( (E/\rho_t L^2) = constant \) is valid to conserve the ratios among the seismic input load and the inertia, elastic and gravity forces - in the case that the geometric similitude is fully respected (i.e. including wall thickness). In many cases, however, the need to change materials to respect both the similitudes which are related to the fluid gravity and the structure elastic forces, or other needs (mock-up weight limits, minimum thickness feasible, etc.), also implies the modification of wall thickness (h) using a scale different from that used for the overall geometry. An example is shown in Sect. 6.3.7.

6.3.3 Examples of core tests using element mock-ups in full scale

6.3.3.1 Need for the core full-scale tests

Correct values of the damping and natural frequencies of the core elements can only be obtained by use of full-scale tests on prototypes of the actual elements. These mock-ups must reproduce correctly not only the mass and stiffness axial profiles, but also the mobile parts which constitute the element internals (for instance, the pin bundle of the fuel elements) and the spike-can clearance profiles. These items make the seismic response of the individual elements highly non-linear. In the work of ref. [6], these affected the element vibrational response.
This was the reason why, in the experiments performed for the dynamic characterization of the PEC core elements (ref. [6]), the only difference between the mock-ups and the actual elements was a simplification of some parts which are not essential from the dynamic point of view and the replacement of some materials e.g. fuel pellets by equivalent materials. The internals of the PEC element mock-ups were constructed exactly as the actual elements. In addition, in the final tests, prototype spikes and support tubes were used.

Furthermore, tests on full-scale mock-ups of the core elements are judged necessary to achieve a realistic representation of the impact phenomena occurring between the elements. The impact phenomena are also very non-linear. Their characterization includes not only peak values, but also shock duration, number of impacts per unit time and impact velocity. These data are necessary for a reliable structural and functional verification of the element wrappers (refs. [6, 7]).

Detailed experimental analysis using full-scale mock-ups of the core elements were carried out for PEC, SUPERPHENIX-1 and MONJU. We summarize below the fundamentals of these tests.

6.3.3.2 Core tests performed for PEC

In the case of PEC (ref. [6]), tests on simplified and prototype elements (3 m length) were performed in air and water, using shaking table. Seismic excitation was gradually increased up to above the SSE level. Single prototype elements, different pairs of element types which are adjacent in the core, and a group of three fuel elements were studied in detail.

Tests allowing for the application of the actual shock forces which exist in the core (up to the SSE) were also performed on prototypes, by simulating the core-restraint ring. In these tests direct measurement of the shock forces was made (Fig. 6-3).

Finally, fluid-structure interaction tests were carried out on various clusters of 7 and 19 simplified elements of the same and different types. The restraint ring was again simulated in some tests (Fig. 6-4).
These tests provided for the design data, at both the SSE and the OBE levels i.e. natural frequencies, damping values, shock forces and fluid effects. With regard to fluid effects, it is interesting to note that these were found negligible in the PEC case, perhaps because of the thick pads which are attached to the element shrouds at two axial levels.

As mentioned, test results obtained for PEC were also used to verify experimentally the core element structural and functional requirements (ref. [7]). Furthermore, numerical analysis of the experimental data
provided a large contribution to non-linear code validation (the one-dimensional CEA/ENEA program CORALIE [8] and the two-dimensional program CLASH of Belgonucliaire [9], see Fig. 6-5).

![Diagram](image)

Comparisons between maximum and effective displacements computed and measured for the five instrumented elements of the 19-element group of fuel and reflecting mock-ups tested on the MASTER table with restraint (a = tests in air; b = tests in water; exp.; O = CORALIE; ▲ = CLASH).

\[ (A_{RMS}) = \text{cm/s}^2 \]

Fig. 6-5: PEC core seismic analysis

6.3.3.3 Core tests performed for SUPERPHENIX-1

In the case of Superphénix-1, tests on full-scale mock-ups were limited to groups of up to four fuel elements (not simulating the internal details) and four dummy subassemblies (ref. [10]). But a wide-ranging, complementary, experimental programme was performed on a
whole core in reduced scale (Rapsodie mock-up, see Sect. 6.3.6) and also on clusters of Phénix elements.

Tests on the Superphénix-1 elements were carried out in air and water, using the snap-back technique. They provided damping values (5% of the critical) which were subsequently used in the design calculations and correlations of the shock parameters.

The conservative nature of the previously mentioned damping values was confirmed by tests performed in air on clusters of 7 and 19 subassemblies of Phénix geometry: these showed that damping increases with increasing number of elements.

6.3.3.4 Core tests performed for MONJU

Finally, the experiments performed in the framework of the MONJU core seismic verification included a complete line across the core diameter (29 elements) and the cluster of 37 elements which characterizes the core centre (see ref. [11] and Fig. 6-6). The experiments were performed on shaking tables, both in air and water.
Test results provided data for validation of simplified and detailed, non-linear, numerical approaches developed in Japan (VIOLLON and FINAS, see ref. [12] and Fig. 6-7). They have also shown that the line model gives a good representation of the whole core behaviour and is suitable for the design calculations.

Fig. 6-7: Code validation based on Monju core mock-up tests
6.3.4 Examples of full-scale mock-up tests of shutdown systems

6.3.4.1 Need for the shutdown system full-scale tests

The demonstration of shutdown feasibility in an earthquake makes it necessary to test actual systems, or at least mock-ups in full scale which are prototypic in all the parts of interest. The reasons have already been explained in Sect. 5.11.

The full-scale shutdown systems have been of the most recent fast reactor designs and projects have been seismically qualified (refs. [13-16]). Some description of these tests is provided below.

6.3.4.2 Shutdown system tests performed for SUPERPHENIX-1

In the case of Superphénix-1 both primary system types were tested in water using prototypes (ref. [13]). The systems (21 m length) were simultaneously excited at three axial levels (Fig. 6-8):

- rotating plug, from which the system is suspended;
- top of the control rod assembly;
- pad level of the control rod assembly.

The different excitations were those computed at the different axial levels (non-linear CORALIE analysis was used for the core).

Because no facility existed in France which enabled this kind of slender structures to be tested, a new facility (VESUBIE) was designed and installed in an existing pit located at the Saclay nuclear center of CEA. The test objectives were:-

- to demonstrate control rods insertion;
- to proof absence of seismically induced damage of the system;
- to measure increase of scram time;
- to obtain data for code validation and calibration.

Ten OBEs and one SSE were applied to the system. The effects of the excitation direction (in plane or out of plane) were studied, together with those of the initial drop time and the control rod absorber initial position.
Fig. 6-8: SPX-1 primary shutdown system mock-up with locations of the excitations and the transducers.

Fig. 6-9: Control rod stress.
All the test objectives were fully attained. In the worst conditions, drop time increased by 10% by the earthquake. Furthermore, no system damage occurred, at the SSE level.

With regard to code validation, a good comparison between computed and measured response time-histories was achieved as shown in Fig. 6-9.

6.3.4.3 **Shutdown system tests performed for PEC**

In the case of PEC, a full-scale mock-up of the shutdown system was tested (ref. [14]). This was prototypic with regard to the in-vessel part of the system. The masses of the ex-vessel part were also simulated. Four simultaneous excitations were applied to the mock-up (Fig. 6-10):

- at core diagrid level, where the control rod guide elements are supported;

- at the upper pad level of the control rod guide elements, which is located 442 mm below top of the core;

- at the core hold-down level, located above the core, whose petals are traversed by the control rod guide tubes (these tubes constitute the system upper part);

- at the rotating plug level.

Although the control rod guide elements interact with the other core elements at both pad levels, excitation was not provided at the lower pads (1440 mm above core diagrid), because controlling the motion at the upper pads had been found sufficient. Even the control of four simultaneous excitations was a difficult to achieve, from the experimental point of view.

On the contrary, excitation at the hold-down system level was found to be essential, because of the possible interactions between the petals and the control rod guide-tubes and due to the need of simulating the vessel rotation.
View of the mock-up of the PEC shutdown system used in the seismic tests.

Absorber vertical displacement and velocity measured at TSS in the PEC shutdown system experiments.

Fig. 6-10: PEC shutdown system experimental analysis
The test objectives, modalities and results were very similar to those of the Superphénix-1 tests. Tests were performed in air and water. Excitation time-histories were those calculated, again using CORALIE with regard to the displacements applied by the core to the control rod guide elements. Six OBEs and two SSEs were applied in the final seismic tests, carried out in water. Similar to the Superphénix-1 analysis, the effects of the initial drop time and those of the initial control rod position were studied and code validation was performed (see also Sect. 5-11.2). Fig. 6-10 shows the measured absorber motion during the SSE.

6.3.4.4 Shutdown system tests performed for MONJU

As far as the MONJU reactor is concerned, all the three types of shutdown systems which are present in the reactor were tested (ref. [15]). The purpose was to verify the design requirement that the systems maintain the function of inserting control rods into the core in 1.2 s to 85% of the full stroke. This requirement had been conservatively determined to guarantee seismic safety.

The upper core structure was modelled in the tests, using the same structure of MONJU, because it had been found that it affects system seismic response. For the same reason, the fuel assemblies adjacent to the control-rod elements were included. This is a different approach with respect to that used for Superphénix and PEC, where the calculated displacements of the control-rod element resulting from core non-linear calculations were directly applied to the systems.

Two simultaneous excitations were applied to the MONJU systems (Fig. 6-11): these were obtained by applying safety margins to the calculated values. As usual, tests were performed in air and water. The effects of pressurized gas (which assists control rod insertion in MONJU) were reproduced.

Unlike for Superphénix-1 and PEC, the effects of the fluid flow inside the guide elements was also simulated, because they had been considered significant. To do this, the actual flow velocity was reproduced in the water tests. As shown below, this assumption was conservative, because it implied larger fluid resistance in the tests, with respect to the reactor conditions.
Indeed, the use of water instead of liquid sodium leads to only slight changes of the fluid density and the superficial tension, but to considerable changes of the viscosity. By keeping flow velocity (V) unchanged, the actual ratio between the inertial fluid forces due to flow and the gravity forces (Froude number $= \frac{V^2}{gL}$, see ref. [17] and Sect. 6.3.2) was conserved. Obviously, also the ratio between the fluid inertia forces due to vibration and those due to flow (Strouhal number $= \frac{WL}{V}$) was conserved, together with all the structural inertia and elastic forces, because the actual sizes and excitations were used.
On the contrary, the Reynolds number \((\rho_f VL/\mu)\), which characterizes the ratio between the fluid flow inertia forces and the fluid flow viscosity forces, became 3 times lower, due to lower density and especially, higher viscosity of water. Furthermore, the Weber number \((\rho_f V^2/\sigma_f)\), which describes the ratio between the fluid flow inertia forces and the superficial tension forces, became 10% higher.

However, higher Reynolds numbers and lower Weber numbers in the tests implied higher fluidic forces and larger viscosity resistance with respect to the actual reactor conditions. This proves the previous statement, that test conditions were conservative.

We note that also the tests performed on the MONJU shutdown systems confirmed adequacy of the system design in an earthquake (see, for instance, Fig. 6-12).

<table>
<thead>
<tr>
<th>Test M.</th>
<th>Item</th>
<th>Eigen Frequency</th>
<th>Edge Displacement</th>
<th>Initial Displacement</th>
<th>Relative Displacement</th>
<th>Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Static</td>
<td>0 Hz</td>
<td>0 mm</td>
<td>0 mm</td>
<td>5 mm</td>
<td>$\circ$</td>
</tr>
<tr>
<td>2</td>
<td>Simultaneous Sinusoidal Excitation</td>
<td>7.1 Hz</td>
<td>2.0 mm</td>
<td>0 mm</td>
<td>5 mm</td>
<td>$\circ$</td>
</tr>
<tr>
<td>3</td>
<td>Sinusoidal Excitation</td>
<td>7.1 Hz</td>
<td>2.2 mm</td>
<td>0 mm</td>
<td>5 mm</td>
<td>$\triangle$</td>
</tr>
<tr>
<td>4</td>
<td>Sinusoidal Excitation</td>
<td>7.1 Hz</td>
<td>2.4 mm</td>
<td>0 mm</td>
<td>6 mm</td>
<td>$\square$</td>
</tr>
</tbody>
</table>

Fig. 6-12: Test results obtained for the Monju shutdown system
6.3.4.5 Tests performed for the ASD system of CDFR

Finally, with regard to CDFR, the tests performed on the Alternative Shutdown System (ASD) described in Sect. 5.11.6 (Fig. 5-17) are particularly interesting, due to the system inherently safety features.

Seismic tests were performed in air and water by exciting the system with a shaped Fourier spectrum, which had been suitably broadened to account for uncertainties (ref. [16]). Although the limitations of the test rig made it impossible to increase excitation levels up to the SSE, encouraging results on the inherent safety features of this system have already been attained. Indeed, no significant increase of either the drop time and the maximum levitation flows were found.

6.3.5 Tests in sodium on the SUPERPHENIX-1 primary pump bearing

Pump bearing is another component where full-scale tests may be necessary, to adequately represent the dissipation forces and other important bearing parameters (ref. [17]). For this reason, the seismic tests were performed in sodium on the primary pump shaft of Superphénix-1 beside those carried out in water (ref. [18]). Tests in water were performed on shaking table at CEA (ref. [35]), while tests in sodium were carried out in operating conditions at the ENEA Center of Brasimone on the CPV-1 rig (Fig. 6-13).

![CPV-1 test rig diagram](image)

1) Pivoterie-1 vessel  
2) Duty pump  
3) Metal frame  
4) Hinge pins  
5) Electrohydraulic actuator

Fig.6-13: Sketch of the CPV-1 test rig used for dynamic tests in sodium of the SPX-1 primary pump shaft
They constituted the final phase of a detailed experimental campaign which analysed the shaft behaviour in all the operating conditions of interest. Concentrated excitations (including seismic time-histories) were applied at the flange supporting the hydrostatic bearing. The excitation level attained values representative of the SSE loading. The tests contributed to the qualification of the pump rotating parts. In particular, the hydrostatic bearing stiffness was compatible with seismic requirements. Fig. 6-14 shows the hydrostatic bearing response during the highest excitation test.

Fig.6-14: Seismic test results of the SPX-1 primary pump shaft (SSE)

6.3.6 Tests on a whole core in reduced scale (Rapsodie mock-up)

The "Rapsodie" mock-up is an example of seismic tests on a whole core in a reduced scale (ref. [10] and Fig. 6-15). This mock-up makes use of hexagonal fuel elements of the Rapsodie core, which roughly reproduce those of the Superphénix-1 core with a geometrical scale factor \( \lambda = 1/3 \). To describe the whole core behaviour, simplified, cylindrical, steel models of the neutron shield elements (PNL) were constructed. Only two element types, which were judged to dominate the core seismic response.
Fig. 6-15: Seismic core tests on the Rapsodie mock-up
The model consisted of 91 fuel elements located at the centre and 180 PNLs. In this way, the geometry scale factor for the whole core was also maintained.

It is noted that this similitude could not be fully maintained for some local geometrical parameters such as gaps between the core elements, also because the pad design of the mock-up elements is different from that of the actual elements. Although, these parameters affect the non-linear core behaviour (shocks), they cannot be precisely reproduced in a reduced-scale mock-up.

The structural materials simulated correctly the actual materials, with regard to both density and Young's modulus. This was sufficient, due to the fact that core elements can be considered equivalent to flexible beams. Moreover, maintaining the actual material was possible because gravity effects do not play any role in the core seismic analysis. Because the free surface of the liquid sodium in the vessel is far from the top of the elements, this was also simulated, in the mock-up by keeping the fluid surface well above the top of the core, to avoid free surface effects.

The demonstration that the elements were correctly simulated was verified by test. The first natural frequency was measured in air for the individual mock-up elements. These were about three times higher than the corresponding Superphénix-1 elements which demonstrated a correct scaling.

It is worth mentioning that the natural frequencies of the neutron shield elements of Superphénix-1 were later found to be lower than those used to define the PNLs for the mock-up. The reason is that the large spike-can clearance effects had been neglected in the calculations. Thus, the PNLs used in the tests do not represent very well the real neutron shield elements of Superphénix-1. But this does not invalidate the results, because tests mainly were used for validating the non-linear codes CORALIE and CLASH.

Due to the geometry scale factor used and the fact that actual materials had been correctly reproduced in the mock-up, the seismic excitations which were applied to the mock-up were obtained from those computed at the Superphénix-1 diagrid by contracting time by a factor 3.
(conservation of the elastic forces) and amplifying acceleration amplitudes also by a factor 3 (conservation of the velocities).

Furthermore, water was used instead of sodium. Because water density is very close to density of the liquid sodium and actual structure density was maintained, the condition regarding the conservation of the fluid inertial forces, was also satisfied. Finally, we note that, because the fluid flow effects are negligible in comparison with the fluid inertial forces related to seismically-induced vibrations, tests were performed in a closed water container (zero flow).

The Rapsodie tests contributed to the validation of the non-linear numerical models used for the Superphénix-1, PEC and CDFR core design verification and qualification by analysis. They enabled improved procedures, such as the homogeneization methods described in the section 4.2.2.2, to be validated. Tests also showed that the whole core seismic response can be well calculated in the case that single element behaviour, shock phenomena and fluid-structure effects have been characterized adequately. It is noted, however, that recent analysis of the test line model for the Rapsodie mock-up, and thus, the need of calculations of the entire core (ref. [36]).

![Graph showing comparisons between Coralie calculations and Rapsodie mock-up test results](image)

**Fig. 6-16:** Comparisons between Coralie calculations and Rapsodie mock-up test results
6.3.7 Sloshing analysis on a MONJU reactor-block mock-up in reduced scale

As example of a reduced-scale mock-up for the sloshing analysis, let us consider the case of the sloshing tests performed for the MONJU reactor vessel (ref. [20]). The mock-up of the MONJU reactor vessel for seismic sloshing tests is shown in Fig. 6-18. It is a 1/4 scale model of the actual vessel (the full scale vessel is about 7 m diameter, 16 m length and 50 mm wall thickness). The mock-up consisted of the reactor vessel (R/V), core internals (C/I), inner barrel (I/B), dip plate (D/P) and upper core structure (U.C.S.).

The tests objectives were to investigate the sloshing effects on the dip plate integrity, to verify whether this plate is effective in mitigating or suppressing sloshing, and check that sodium outlet nozzles are not exposed to cover gas due to sloshing waves. Thus, the mock-up was designed to simulate free surface waves, by conserving the ratio between the input seismic load and the gravity forces. Also the ratio between the structural and fluid inertia forces (Sect. 6.3.2.4) has been preserved. Specific tests to analyse the MONJU reactor-block vibrational behaviour were performed on a different mock-up.

The scaling laws applied to the model are given in Fig. 6-19. Plastic material was used in the mock-up. The wall thickness was decreased by a factor 1.66, i.e. different from the overall geometry.
Fig. 6-18: Mock-up of reactor block for vibration testing (1/4, plastic)

<table>
<thead>
<tr>
<th>Physical Quantities</th>
<th>Symbol</th>
<th>Dimensions</th>
<th>Scale</th>
<th>Value</th>
<th>Remarks</th>
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<td>L</td>
<td>L</td>
<td>n</td>
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<td></td>
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<tr>
<td>Young's modulus</td>
<td>E</td>
<td>ML^{-1}T^{-2}</td>
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<td>fixed values</td>
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<td>T</td>
<td>n^{1/2}</td>
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<td></td>
</tr>
<tr>
<td>Wall thickness</td>
<td>h</td>
<td>L</td>
<td></td>
<td>1.66</td>
<td></td>
</tr>
<tr>
<td>Acceleration</td>
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<td>1</td>
<td></td>
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<tr>
<td>Structure density</td>
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<td>n^{-1/3}(E*)^{-1/3}(ρ_f)^{-1/3}</td>
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<td></td>
</tr>
<tr>
<td>Dynamic pressure</td>
<td>p</td>
<td>ML^{-1}T^{-2}</td>
<td>nρ_f</td>
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</tr>
<tr>
<td>Stress</td>
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<td>n^{1/3}(E*)^{1/3}(ρ_f)^{2/3}</td>
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<tr>
<td>Deflection</td>
<td>δ</td>
<td>L</td>
<td>n</td>
<td>4</td>
<td></td>
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a) Subscript s means the structure and f means liquid. And p means prototype, m means model.

Fig. 6-19: Similarity law for sloshing of 1/4 scale plastic model (sloshing tests on a Monju vessel)
scale factor, due to the very large decrease of $E/\rho_f$ entailed by the use of plastic material instead of steel. Masses were attached to the shell of the model in order to allow for the conservation of the ratio between the inertia forces of the structure and the fluid. To vary the wave height in proportion to the geometrical scale factor $\lambda$, the input displacements were changed accordingly by $\lambda$.

The use of the additional masses and input displacements, together with the adoption of the similarity law for the gravity forces led to the increase in the excitation frequency by a factor 2 for the mock-up ($\omega$ varies in proportion to $[l/(l/4)]^{1/2}$). It also made it necessary to use the actual input acceleration in the tests (see Sect. 6.3.2.4).

It is worth mentioning that the use of amplitude parameter (1) equal to the characteristic length (L) was very important for evaluating efficiency of the dip plate and the requirement concerning nozzles (it is obvious that geometric similarity must be also maintained for the sloshing waves).

The use of the dip plate was found to be very beneficial as shown in Fig. 6-20. Test results also allowed for the validation of numerical methods (Figs. 6-21 and 6-22).

![Wave Height](image)

**Fig.6-20: Efficiency of the dipped plate to mitigate sloshing in the Monju vessel**
### Table

<table>
<thead>
<tr>
<th>Method</th>
<th>Model</th>
<th>Mode</th>
<th>Maximum wave height (mm)</th>
<th>Maximum pressure (kg/m²)</th>
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<td>$1.25 \times 10^{-3}$</td>
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<td></td>
<td>axisym.</td>
<td>1st + 2nd</td>
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<td>1.23</td>
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<tr>
<td></td>
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<td>1st + 2nd</td>
<td>279</td>
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<td></td>
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</tr>
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</table>

### Fig. 6-21: Sloshing frequencies by analyses and tests

<table>
<thead>
<tr>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
</tr>
</thead>
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<tr>
<td>1st</td>
<td>0.726</td>
<td>0.726</td>
<td>0.682</td>
</tr>
<tr>
<td>2nd</td>
<td>1.297</td>
<td>1.297</td>
<td>1.251</td>
</tr>
<tr>
<td>3rd</td>
<td>1.733</td>
<td>1.773</td>
<td>1.831</td>
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</table>

<table>
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<th>0.731</th>
<th>0.730</th>
<th>0.687</th>
<th>0.454</th>
</tr>
</thead>
<tbody>
<tr>
<td>FINAS 3D</td>
<td>2nd</td>
<td>1.308</td>
<td>1.308</td>
<td>1.263</td>
<td>3.965</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>1.785</td>
<td>1.785</td>
<td></td>
<td>6.269</td>
</tr>
</tbody>
</table>

| Potential theory | 1st    | 0.721  |        |        | 0.447  |
|                  | 2nd    | 1.227  |        |        | 3.768  |
|                  | 3rd    | 1.555  |        |        | 5.327  |

| Experiment | 1st    | 0.71   | 0.71   | 0.68   | 0.43   |
|           | 2nd    | 1.21   | 1.21   | 1.18   |        |
|           | 3rd    | 1.53   | 1.53   | 1.59   |        |

### Fig. 6-22: Validation of sloshing analysis based on test results of a mock-up of the Monju reactor block

#### 6.4 Instrumentation

Contrary to the case of the flow-induced vibrations, classical transducers (accelerometers, strain gages and displacement transducers) can usually be used for the experimental seismic analysis. For many non-linear problems, displacement transducers have been found to be much more useful than accelerometers, to provide reliable results for code validation and assessment of input data.
It is noted that new techniques for displacement measurements are under development. Measurements by optical means, for instance, have been successfully tested in the cited experiments of the Rapsodie core mock-up (see ref. [10] and Fig. 6-23). They allow for a correct measurement of orthogonal components of the motion even in the case of strongly two-dimensional response. It is possible to use displacement and acceleration transducers for tests in water.

![Diagram](image)

**Fig.6-23:** Displacement measurement by optical system in the Rapsodie core tests (see also Fig.6.15)

Pressure transducers are also needed, for instance in the buckling tests due to pressure. Seismometers and accelerometers are used in the on-site experiments. Finally, quartz force transducers have successfully been tested to analyse shock phenomena (ref. [6]).

With regard to the instrumentation of actual plants to monitor the reactor behaviour during an earthquake, it is noted that this instrumentation cannot withstand high doses and temperatures. Thus, the seismic monitoring system cannot include direct measurement of the seismic response of many safety-related components. Indeed, the radiation doses and temperatures are very high on these components during reactor operation, and there are access problems for maintenance or replacement.
6.5 **Excitation types**

Various types of excitation can be applied in the seismic tests. They may be both mono-frequential and multi-frequential. Multi-frequential excitations can be steady-state (random) or transient. The latter include seismic excitations. In the case of non-linear structures, mono-frequential excitations were found to be unsatisfactory (ref. [6]) and the use of random excitations provides better results.

Seismic excitations must also be applied. They are artificial earthquakes which must be consistent with the floor response spectra calculated at the structure support in the frequency range of interest (usually, up to 25 – 30 Hz for the full scale structures). Broadening of these spectra is often required to account for uncertainties. The seismic excitation duration must be sufficiently long as to be representative of a real earthquake (at least 10 – 20 s, although in many countries the value is not specified by the codes).

It should be noted that the seismic excitation shape might have a non-negligible effect on the structure response, in the case of non-linear structure behaviour. Thus, both transient and random excitations should be applied to the non-linear system. It may be also convenient to make use of several different seismic time-histories, consistent with the spectrum, in order to evaluate the shape effects. This is also suggested by ref. [21].

Excitation should be gradually increased, usually up to the SSE level. The use of some intermediate steps (not limited to the OBE level) is important to evaluate the effects of the amplitude, which could be very important for non-linear systems. The SSE level should be fully attained in the tests, because of possible difficulty in extrapolating the experimental data.

In the case of systems with multiple simultaneous excitations at various points, the phase difference effects should be evaluated (see, for instance, Sect. 5.11). Tests in which the individual excitations are applied are necessary to characterize the system.
Fragility tests are considered very useful to assess safety margins. These tests may be useful not only to improve design criteria and methods, but also to allow for the use of probabilistic risk assessment related to beyond design basis events.

6.6 Data treatment

For a matter of seismic code validation, reliable results in both the time and the frequency domains are frequently necessary. With regard to the time response, peaks are often not sufficient for validation. Effective (root-mean-square) values and time-histories are frequently needed (for instance, for the core analysis, see ref. [6]).

Furthermore, as far as frequency values are concerned, we note that the experimental peak-response frequencies are often not adequate for a direct use in the computer codes in terms of natural frequencies of the system. This is especially the case of very non-linear systems, for which the natural frequencies to be used in the codes should be better based on the criterion of providing the best possible approximation of the frequency response function in the whole frequency range of interest (see again ref. [6]).

With regard to the type of frequency functions, it is worth noting that the use of transfer functions is theoretically not correct for the non-linear systems with transient excitations. However, transfer functions can be used for the purpose of comparison between the measured and the calculated functions. For the above-mentioned cases, the single Fourier transforms of the response and even spectra of the response can be also used.

Finally, we note that transfer functions between input acceleration and response parameters such as displacements have been found very useful for comparison with the calculated data in the case of highly non-linear problems such as core analysis (ref. [6]).

6.7 Seismic qualification of equipments

Usually, experimental seismic qualification of fast reactor equipments does not differ significantly from that adopted for the other nuclear reactor plants. It is more complicated in the case that testing
in sodium is judged necessary (for instance, for pumps): but usually tests in water are considered sufficient.

The difference with respect to the thermal reactors is that complicated experimental studies have also to be performed in support to the design. We have provided some examples of these tests in Sects. 6.3.3-6.3.7. However, strictly speaking, such tests do not constitute a qualification (at least, in the majority of the cases), because they do not use the actual component and/or they do not provide a full simulation of the actual reactor conditions. In these cases qualification by analysis is done in parallel.

With regard to the rigorous qualification tests, the components must be artificially aged to simulate the effects of their life in the reactor (thermal aging, vibration aging, etc.). The component is usually vibrated on a shaking table, using artificial earthquakes. The number of OBEs required by the codes applicable must be applied. OBE tests must be followed by a SSE test. No repair between the mentioned tests is permitted.

Methods using one or two directional, mono- or multi- frequential excitations are defined by the codes. We note that an extensive use of two directional simultaneous excitations (one horizontal direction + the vertical direction) has been made. In this case test spectra must envelope the design spectra with a certain margin, to account for the actual three-directional nature of the earthquake excitation (20% is a typical value, see ref. [22]). The new multiaxial shaking tables (Sect. 6.2.2.1) will allow for the improvement of qualification methods of equipments, using simultaneous excitations in the two horizontal directions, as well. The electrical components are tested in operation to detect possible malfunctioning which might occur during seismic excitation.

6.8 On-site analysis

6.8.1 On-site experience for fast reactors

Detailed on-site dynamic analysis of fast reactors with forced excitations has been performed up to now only for PEC (ref. [2]). Some on-site tests with forced excitation were also carried out for other
reactors, but they were limited to very particular items. For instance
snap-back tests were performed on the piping system of the FFTF reactor
in the U.S.A. (ref. [23]).

With regard to Superphénix-1 forced vibration tests were not judged
feasible under significant excitations because of the large sizes of the
buildings and vessels (ref. [24]). However, extensive measurements were
made in air during pre-operational tests on components and internal
structures (ref. [24]). The main objective was to determine the vibration
characteristics of the primary circuit for the understanding of the
signals recorded during normal operation by the permanent instrumentation.

Among other results, the Superphénix-1 start-up tests confirmed
adequacy of the first natural frequency values computed in the design
analysis for relevant components such as the intermediate heat exchanger,
the above-core structure and the internal vessel.

6.8.2 On-site experience for thermal reactors

Some on-site tests have been performed or are in progress on thermal
reactors in Japan, the F.R.G., Switzerland, France, the U.S.A. and Italy
(ref. [2]). In Japan, on-site tests have been performed on Unit 3 of the
Hamaoka BWR (ref. [25]), the SN PWR (ref. [26]) and Unit 2 of the Tsuruga
PWR (ref. [27]). Tests were performed in the period 1983-1986. After
ambient vibration, these plants were excited using mechanical vibrators
which applied forces in the range of 100 - 1500 kN). The SN reactor also
experienced a real earthquake, whose effects were measured by the reactor
instrumentation.

In Switzerland, interesting on-site tests were performed on the Bota
building of the Beznau PWR (ref. [28]). Forced excitation was provided by
blasting in bore-hole and a mechanical vibrator.

Forced excitation tests in France and the U.S.A. have been limited
to components (ref. [2, 29]). Mechanical vibrators have been used.

In Italy, besides the on-site experiments which have already been
carried out on the PEC fast reactor (Sect. 6.3.8), tests are in progress
on the Garigliano plant using blasting in bore-hole (ref. [30]). This
plant is being decommissioned.
But the most interesting on-site tests on thermal reactors are those conducted on the Heissdampfreaktor in the F.R.G. (ref. [31]). These tests are being performed in co-operation with the U.S. EPRI. They are the only on-site tests on actual reactors in which excitation was increased up to very high levels (intensity X of the MCS scale was reached in the tests performed in 1986). This large excitation was achieved by use of a very powerful eccentric mass vibrator, which applied a force of 10,000 kN (Fig. 6-24).

Fig. 6-24: Eccentric mass coast-down shaker and installation on the 30m floor of the HDR reactor building

One should note that such high excitation levels were made possible by the very particular conditions of the plant, which remained in operation for a short time only (2000 hours). Thus, the safety problems related to possible contamination were minimized.

To conclude, we note that tests carried out on the "Heissdampfreaktor" demonstrated plant inherent safety features, in spite of the very high excitations applied and although the plant had not been designed against earthquakes.
6.8.3 On-site tests performed on the PEC fast reactor

On-site tests were performed for PEC on both the reactor building and the main vessel (refs. [2, 32]). PEC is the only non-Japanese reactor on which detailed on-site tests with forced excitations were performed during the construction phase.

6.8.3.1 Building tests

A very detailed on-site experimental campaign was carried out on the PEC reactor building in the period 1983 - 1985, at three construction stages (ref. [5]). The last tests were performed after completion of the building construction. The experiments were significant in spite of the low excitation levels because of the linear soil behaviour.

Tests started before construction was completed, in order to check that the building first natural frequency computed in the design analysis was correct. After ambient vibration processing, the following forced excitation techniques were used:

- excitation with a back-rotating mass mechanical vibrator, located in various positions, axial levels and directions inside the reactor building;

- vibrations generated by blasting in bore-hole (two charges of 300 kg TNT were exploded at about 2 km from the reactor and a depth of 60 m);

- hydraulic actuators (flat jacks) mounted in a small gap (5 cm) between the foundations of the reactor building and much larger fuel handling building (Fig. 6-25).

The mechanical vibrator generated forces up to 100 kN, which were sufficient to obtain reliable results because of relatively small reactor sizes. The hydraulic actuators were capable of transmitting much higher forces (2500 kN). The excitation limits had been established in a very conservative way in agreement with licensing authorities. They also took into account that mounting of some components and systems (fuel transit channels, piping) was in progress at the time of the experiments.
Very detailed instrumentation network was used. In the last tests, for instance, the reactor building motion was measured by a network of 55-unit transducers, and 24 further measuring positions were located in the adjacent buildings. The latter were used to evaluate building-to-building coupling.

The processing of the experimental results allowed the vibrational behaviour of the reactor building to be determined in detail, taking into account soil-structure interaction. It also made it possible to determine the main parameters which characterize the other buildings.

The analysis of the experimental data confirmed the main results of the design analysis (natural frequencies, floor response spectra). Furthermore, it enabled a detailed three-dimensional model of the building to be validated and calibrated (ref. [33]). This model takes into account the measured soil-structure interaction effects. Its use allowed for a detailed verification of the design analysis, which had been conducted with a simplified, lumped-mass, model (Fig. 6-26).
Finally, tests and three-dimensional calculations provided the data necessary for the optimization of the PEC seismic monitoring system described in Sect. 5.14.

6.8.3.2 Main vessel tests

Also the on-site tests of the PEC main vessel (ref. [34]) were necessary to verify that the design modifications were sufficient to guarantee the reactor seismic safety (Sect. 5.9.2). Tests were performed by means of the back-rotating mass mechanical vibrator.

At the time of these tests (1983) the vessel had already been completely mounted. No internals were present when tests were initiated (the diagrid was replaced by a thick plate, used for vessel bottom hydrostatic tests). The dynamic excitation was applied either by fixing the vibrator to the mentioned plate inside the vessel (Fig. 6-27) or by fixing it to the floor which is located about 2 m above vessel support level (working area level).
A second series of tests was performed some months later, to improve understanding of some behavioural modes (ref. [34]). At this time, the diagrid was already in the vessel, thus, excitation was only provided at the working area level.

The effects of different directions of vibration were analysed. The experiments allowed for the evaluation of the first two natural frequencies and the modal shapes of the empty vessel. They also enabled the stiffness parameters of the vessel supporting system (horizontal, vertical and rocking) to be determined (Fig. 6–28). As mentioned in Sect. 5.9.2, the stiffness of this system governs vessel first natural frequency, and thus, amplification of the seismic motion due to vessel rotation.

![Graph showing horizontal translation and rocking stiffnesses versus frequency.](image)

**Fig.6–28:** Horizontal translation and rocking stiffnesses of the PEC main vessel supporting structure versus frequency, evaluated on the basis of the vessel on-site test results
On the basis of the experimental data, the actual vessel behaviour was well understood and suitable detailed modelling in the computer codes became possible. The detailed numerical models, validated on the measured data, allowed the design methods to be adequately calibrated to take into account effect of vessel ovalization, which had been previously neglected.

REFERENCES


7. CONCLUSIONS AND RECOMMENDATIONS

A large amount of work has already been performed in the framework of seismic analysis of fast reactor block components and piping. This work consisted of the development of advanced numerical techniques and wide-ranging experimental programmes, which covered all the items necessary for demonstrating seismic safety.

Although adequate methodologies to demonstrate fast reactor seismic safety exist, further improvements are necessary for the future reactors, in order to achieve a better knowledge of some physical phenomena, and thus to allow more realistic, less conservative, criteria and techniques to be adopted. The aim is a general improvement of the design, that may result not only in a decrease of plant costs, but also in a further increase in the overall reactor safety. In fact, the use of over-conservative criteria and methods, beside the cost consideration, could be detrimental for the performance in normal operation and often complicates the safety demonstration of design bases accidents which are more frequent than the earthquakes.

Further R & D work is necessary on fluid-structure interaction in general, sloshing and buckling phenomena, core behaviour and pump analysis. Studies should be done to understand how to exploit ductility margins in some components. Also the PRA methodology should be improved and applied in particular to the beyond design earthquake. Finally, on-site test techniques should be improved and applied.

The needs, problems and benefits, of new design solutions, that are capable of substantially reducing the earthquake effects, such as seismic isolation should be studied in depth.

The development and application of advanced seismic monitoring systems, capable of providing a detailed description of the reactor seismic behaviour in the time and frequency domains, is advisable.

Finally, it is noted that the adequate application of the improved methods and design solutions to the future fast reactors requires rationalization of the criteria used for defining seismic input (especially for the OBE) and the use of more reliable ground acceleration
data. In particular, a better knowledge of vertical acceleration component should be achieved, together with that of the long-period waves (these are of fundamental importance in the case of seismically isolated structures).

In conclusion, following recommendations are made:

- The criteria for definition of seismic input should be rationalized;

- To improve the design criteria and methods further R & D work should be carried out, from both the numerical and the experimental points of view, on the following topics:
  
  . soil-structure interaction,
  . fluid-structure interaction,
  . sloshing phenomena,
  . buckling phenomena,
  . core seismic behaviour,
  . pump seismic behaviour,
  . strain criteria - ductility margins,
  . PRA methodology;

- On-site test techniques should be improved and applied to verify the actual dynamic behaviour of structures and components;

- The benefits, needs and problems of new design solutions (e.g. seismic isolation), which have a large potential to reduce the effects of earthquakes event on the reactor design should be studied;

- Advanced seismic monitoring systems which provide detailed information on the reactor excitation and response, should be adopted;

- The exchange of information and the existing international co-operation, mainly in the field of the R & D studies, should be enhanced.