Structural Engineering | Part 2

Beyond the Otherness between Art and Technique

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This continues and concludes the reflections on Structural Engineering as an Alterity between Art and Technique. It begins with the regulatory framework that Structural Engineering cannot ignore.

Speaking of standards...

Structures must be designed to ensure their safety. However, good practice may often be insufficient, especially if the structures are complex. Safety is defined through the use of specific standards that determine approaches, criteria, rules, and relationships that must be considered during the design phase so that the design is not over-dimensioned and does not have any deficiencies that may compromise its safety.

Historically, structural engineering has used the calculation and verification criteria associated with the Working Stress Design (WSD) method, also known as the Allowable Strength Design (ASD) method. It is based on purely deterministic criteria, i.e. it assumes that all loads considered cannot exceed their nominal value. This same assumption also applies to the value of material resistance, which is obtained by dividing the characteristic strength (which may be the yield strength) by an appropriate safety coefficient.

In this sense, the WSD method uses a single safety factor irrespective of the type of load, although, for conditions defined by environmental loads, the "basic" admissible stress may be increased (e.g. by 12.5% according to the CNR-UNI 10011 standard, now out of use, or by 33% according to the AISC — Manual of Steel Construction 9th Edition standard), provided that the stresses caused by these environmental loads are lower than those caused by permanent loads.

But how should one proceed if one is dealing with a calculation action that has greater uncertainties (and is, therefore, ill-defined) compared to other design actions? To simplify, one may decide to proceed by adopting the Load and Resistance Factor Design (LRFD) method (or Limit-State method), which is supported by many years of research and which has actually been made almost compulsory by current standards such as EN (1990 to 1999), ISO (19900 to 19904), and NTC 2018 in Italy.

The method allows higher safety margins (in the form of higher partial safety factors) to be applied to design parameters that are considered less predictable or that could have a negative impact on the design. This provides a more explicit way of accounting for the uncertainties introduced by design parameters, compared to the WSD method.

Therefore, behind the seemingly trivial relationship:

$$E_d \leq R_d$$

whereby E_d and R_d are defined respectively as the design values of the generic effect (E_d) taken into consideration, and of the corresponding resistance (R_d) within the limit state examined (Ultimate — ULS, Serviceability — SLS), one must verify, by means of the method of partial coefficients, that no limit state is violated in any design situation. There are also statistical studies of structural reliability that have resulted in the definition of values for the various partial safety factors and combination factors that consider the probability of different events occurring simultaneously (and unfavourably) to that specific structure.

Evaluation of the structural response to seismic action

For structures in general, and therefore also for special structures and/or for large structures as well as bridges, determining the response to seismic events is fundamental and must be placed within specific regulatory requirements/standards.

Italy has created an operational framework aimed at enhancing the precise specification of the basic seismic hazard. In the governing Italian regulations, NTC 2018, a basic seismic hazard is defined in terms of the maximum expected horizontal acceleration, a_g, in free field conditions on a rigid reference site with a Category A horizontal topographic surface, as well as the ordinates of the corresponding elastic response spectrum in acceleration, $S_e(T)$, with reference to pre-established PVR exceedance probabilities, in the reference period V_R. NTC 2018 foresees four Limit States for seismic action, two of which are Operational (SLO and SLD) and two Ultimate (SLV and SLC). They are briefly summarized below:

- Immediate Operational Limit State (or *Stato Limite Operativo*, SLO) is particularly useful as a planning and design reference for works that must remain operational during and immediately after an earthquake (e.g. hospitals, military barracks, civil protection centres);
- Damage Limit State (or *Stato Limite di Danno*, SLD) is instead defined as the limit state that guarantees only temporary uninhabitability in post-earthquake conditions; in other words, the damage occurring for this limit state must not put users at risk and must not significantly compromise the structure's resistance and stiffness to vertical and horizontal actions, thereby guaranteeing continued use even if the use of (part of) the equipment is interrupted;
- Lifesaving Limit State (or *Stato Limite di Salvaguardia della Vita*, SLV) in which there is substantial damage to structural components and a (significant) loss of stiffness in relation to horizontal actions, but the structure maintains part of its resistance and stiffness against vertical actions and offers a margin of safety against collapse from horizontal seismic actions;
- Collapse Limit State (or *Stato Limite di Colasso, SLC*), in which the structure has experienced serious breakage and collapse of the non-structural and plant engineering components and very serious damage to the structural components, but still retains a margin of safety for vertical actions and a small margin of safety against collapse from horizontal actions. This last limit state is particularly suitable as a design reference for certain structural types (structures with seismic isolation and energy dissipation).

Consequently, the four limit states allow four different situations to be identified. As the seismic action progressively increases, the four limit states, ordered by increasing seismic action (SLO, SLD, SLV, SLC), are progressively exceeded, corresponding to a steady increase in the damage to the structure, its non-structural elements, and its systems overall. This unambiguously and almost continuously identifies the performance characteristics that are required of a generic construction.

In terms of the contribution of computational methods to determining the structural response to seismic action, four types of analysis are allowed, each of which depends on the geometric simplicity (or complexity) of the structure being analysed and on the design performance target, by which engineers verify that the structure can withstand the design event, and even, often importantly, establish how the structure withstands the event, i.e. to what level of damage.

These four computational methods are:

- Linear Static Analysis (LSA),
- Linear Dynamic Analysis (LDA),
- Non-Linear Static Analysis or Pushover (NLSA),
- Non-Linear Dynamic Analysis (NLDA).

The linear analyses (LSA and LDA) involve elastic analyses for determining the deformations and stresses of each structural component. Any non-linearities are conventionally considered through appropriate parameters. Therefore (as stated in Section 7.3.1 of NTC 2018) linear analyses can be used to calculate seismic demand for both non-dissipative and dissipative structural behaviour. In both cases, whatever modelling is used for the seismic action, seismic demand is calculated by referring to the design spectrum obtained for each limit state and by assuming the limits/values specified in the standard for the behaviour factor q, which are a function of the structural type and ductility class.

In all cases, linear procedures must be used with awareness and rationality since they are likely to provide unrealistic results if the structure's behaviour under earthquake action deviates significantly from the elastic one, or if there are localized ductility requirements, or for tall buildings that are generally characterized by pronounced elastic-plastic behaviour.

Non-linear types of analyses involve static (pushover) analyses by applying monotonically increasing horizontal forces to the structure up to a predetermined limit (NLSA), or dynamic step analyses with direct integration of the equation of motion (NLDA).

Non-linear approaches allow elastic-plastic modelling of the structure with the possibility of considering during analysis all dissipative capacities that the structure is able to exhibit and that cannot be considered directly in a linear procedure.

The following combinations should be considered for LSA, LDA and NLSA methods:

$$1.0\ E_x + 0.3\ E_y + 0.3\ E_z$$
$$0.3\ E_x + 1.0\ E_y + 0.3\ E_z$$
$$0.3\ E_x + 0.3\ E_y + 1.0\ E_z$$

where:

- E_x represents the set of effects (stresses and displacements) caused by applying the seismic action along the chosen horizontal x-axis of the structure;
- E_y represents the set of effects (stresses and displacements) caused by applying the same seismic action along the orthogonal horizontal y-axis of the structure;
- E_z represents the effects (stresses and displacements) arising from applying the vertical component of the seismic action.

Linear static analysis

Linear static analysis consists of representing the structure (via beam and/or shell elements) as a linear elastic system, and the seismic action as a system of static forces applied near the individual nodes/slabs where the masses of the structure/construction are assumed to be concentrated. After implementing the FEM model, we proceed to solve the equation:

$$Kx=F$$

where K is the structure's elastic stiffness matrix, x is the vector of generalized nodal displacements (i.e. displacements and rotations, the result of the solution), and F the vector of equivalent seismic actions.

The calculation of displacements results in determining the stresses acting on the various structural components; their investigation is completed by normative verifications.

This method allows a system of forces approximating the structure’s first mode of vibration to be applied to the structural model. Consequently, linear static analysis returns reliable results only if the structure’s seismic response in each principal direction is not significantly affected by vibration modes greater than the first (NTC 2018, Section 7.3.2).

Linear dynamic analysis

Linear dynamic analysis is actually the so-called modal analysis with response spectrum (again, see NTC 2018). Proposed by R. W. Clough and E. L. Wilson in the early 1960s, this procedure determines the effects of seismic action after first determining the eigenvalues (eigenfrequencies) and eigenvectors (modes of vibration) of the structure considered in the elastic field.

In essence, the equation:

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

which summarizes the dynamics of the structure subjected to an earthquake, where M is the mass matrix (of the structure), C its damping matrix, K its stiffness matrix, and $\ddot{x}_g(t)$ the ground acceleration defined by the seismic event, is solved in the space of eigenvectors identified through modal analysis, with the seismic forcing represented by the elastic spectrum relative to the specific limit state under consideration. The analysis must account for all modes of vibration that contribute significantly to the dynamic response of the structure.

The current standard (see Section 7.3.3.1) requires all modes with significant participating masses to be considered. This criterion is considered satisfied if the sum of the effective modal masses, for all the modes considered, totals a significant percentage of the structure (85%), or if all modes with participating masses above a minimum percentage (5%) are considered.

Each of the vibration modes identified is associated with a participation coefficient, and this in turn makes it possible to evaluate the maximum vectors of the equivalent static forces relative to the various modes in relation to the design spectrum. Once the maximum effect in terms of stresses and displacements at each point of the structure being analysed has been found for each vibration mode, the overall effect is evaluated by considering the contribution of each mode of vibration to the maximum response.

The maximum probable value, E, of any effect (displacement, stress, etc.) is given by statistically derived formulae. The most commonly used combinations of seismic responses to obtain maximum effect values are: SRSS (square root of the sum of the squares of the modal responses), E_i and CQC (complete quadratic combination).

The current standard requires the proper use of the CQC combination, defined by the following relationship:

$$E = \sqrt{\sum_i \sum_j \rho_{ij} E_i E_j}$$

where:
- E_i and E_j are the effects relative to modes i and j
- ρ_{ij} is the correlation coefficient between mode i and mode j, calculated by proven methods such as the one below:

$$\rho_{ij} = \frac{8\sqrt{\xi_i \xi_j}\,(\beta_{ij}\,\xi_i + \xi_j)\,\beta_{ij}^{3/2}}{(1-\beta_{ij}^2)^2 + 4\,\xi_i\,\xi_j\,\beta_{ij}(1+\beta_{ij}^2) + 4\,(\xi_i^2+\xi_j^2)\,\beta_{ij}^2}$$

where
- ξ_i and ξ_j are the viscous dampings of modes i and j
- β_{ij} is the ratio of the inverse of the periods of each i-j mode pair ($\beta_{ij} = T_j/T_i$)

Non-linear static analysis (pushover analysis)

A structure's ability to resist seismic action depends primarily on its ability to deform in a ductile manner. In static and dynamic methods of elastic analysis, possible excursions in the plastic field are conventionally evaluated through the use of the behaviour factor q, which reduces the elastic spectrum but does not provide any information on the actual distribution of inelasticity demand when the elastic limit is exceeded.

This is where pushover analysis (non-linear static analysis on a multi degrees of freedom (MDOF) model) can be performed. It consists of subjecting the structural model with associated non-linearities of material and geometry to gravitational loads and to a system of lateral forces that represent the inertial forces activated by the earthquake, which are increased monotonically so as to increase the horizontal displacement of a control point in the structure (e.g. the centre of gravity of the top floor) until the ultimate conditions are reached.

Numerically speaking, this means that the material response, due to inelasticity, can no longer be predicted by a single parameter (the slope of a straight line in the stress-strain plane), but can only be simulated by following the relationship between these two quantities step by step. This in turn implies a transition to an incremental analysis in which, at each load increment, appropriate solving methods (e.g. the Newton-Raphson iterative method) have to be applied to trace the curve representing the intrinsic elastic-plastic behaviour of the material as closely as possible.

The end result of pushover analysis is the building capacity curve, also known as the pushover curve, which is a diagram in which the abscissa shows the displacement value of the control point and the ordinate the base shear.

After introducing the SDOF (single degree of freedom) system, equivalent to the "real" MDOF structural system (see Section C7.3.4.2 of Circular No. 7 of 21 January 2019 C.S.LL.PP.), we assess the displacements of the structure at predefined seismic load levels and check that the displacement requirements exceed the displacements for achieving the reference performance levels, evaluated according to the pushover analysis on the "real" model.

Pushover analysis also enables the behaviour factor q to be determined and thus permits more reliable linear dynamic analyses in terms of structural behaviour that implicitly accounts for the elastic-plastic response.

Non-linear dynamic analysis (time history)

This type of analysis, also called path following analysis (pushover analysis is a path following analysis) allows the seismic response of the modelled structure to be assessed by directly integrating the equations of motion, thus considering the non-linear behaviour of both material and geometry. Gravity loads and accelerograms compatible with the elastic response spectrum(s) are applied to the three-dimensional model of the structure, which is represented with beam and/or shell elements as appropriate.

This is the most complete procedure for evaluating a structure's stresses and deformations in the time domain, however, it is also the most complex form of analysis requiring close attention to defining a model capable of describing the structure's post-elastic behaviour to load-unload cycles, as well as careful selection of the accelerograms to be used.

For this latter reason, the Italian standard requires the use of at least three triads of accelerograms (each characterized by three

accelerograms acting simultaneously in the three main directions) to calculate the heaviest response. Here it is important to remember that the main qualitative difference between linear and non-linear analysis is the fact that the principle of superposition of effects is lost.

In linear analysis, the structure's response to a combination of different actions can be obtained by totalling the single responses for each of the actions that "belong" to that specific combination; in non-linear analysis, on the other hand, each of the possible load combinations (and not each action) must be analysed.

As already mentioned, incremental path-following analyses require suitable solution methods such as the iterative Newton-Raphson method. One disadvantage of this method is that it does not allow post-peak strength loss (corresponding to softening behaviour) to be captured without the addition of specific numerical techniques.
In fact, due to its formulation, the Newton-Raphson method is a poor choice in cases where the structure's stiffness matrix is not purely positive, impeding analysis in problems that present instabilities in the form of loss of stiffness (of a geometric and/or material nature).

To overcome this difficulty, various numerical strategies are often used with the Newton-Raphson method, including the Arc Length or Modified Riks Method. Used as an extension to the Newton-Raphson method, this is a powerful numerical technique to solve systems of highly non-linear equations efficiently and accurately even where Newton-Raphson fails.

Case study of the seismic improvement of a nuclear power plant

The challenges that arise can have considerable formal and conceptual complexities. Therefore, it is worthwhile analysing the case of the redevelopment of a nuclear power plant because of the breadth and articulation of the activities developed.

For this nuclear power plant, specific studies performed in the last decade of the last century and in the first decade of the 2000s identified a new and higher seismic hazard value, on a probabilistic basis and for a return period of 10,000 years, compared to the one used during the plant's design phase. This resulted in the definition of a precise RLE (Review Level Earthquake), which required a seismic adjustment of the buildings constituting the facility.

The associated RLE spectrum (see Fig. 1) relative to an SL-2-level earthquake according to the International Atomic Energy Agency i.e. with a return period of 10,000 years, is characterized by an average PGA (peak ground acceleration) of 0.143g, conservatively assumed to be 0.17g of horizontal acceleration, with a damping equal to 5% of the critical. The vertical spectrum is assumed to be two-thirds of the horizontal spectrum.

The seismic retrofitting of the power plant, developed by EnginSoft together with the contracting company, was conducted by specifically considering and modelling:

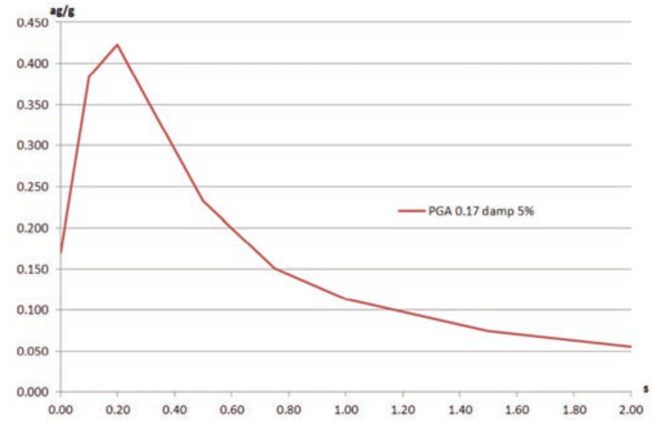


Fig. 1. RLE spectrum used in the multimodal dynamic analyses.

- the Reactor Building with:
  - the actual Reactor Building (housing the Reactor Primary Loop)
  - the Longitudinal Side Electrical Building
  - the Turbine Hall
- the Reactor Primary Loop
- the Auxiliary Building

The objective of the activities undertaken was to determine the seismic response of the above buildings and of the Reactor Primary Loop (RPL) to assess each structural element/component's safety level and to identify any structural improvements necessary to restore the safety margins to acceptable values.

The study, and the (correct) interpretation of over 2000 drawings, were used to generate the FE models of the structures (Reactor Building and Auxiliary Building) and of the RPL.

Shell elements were used to model the reinforced concrete structures (partitions, walls, full-thickness slabs, mixed-structure floors), while beam elements were mainly used to model the steel structures, given their type. Finally, shells, pipe elements and beam elements were used to model the RPL and its components.

The FEM models were statically analysed for the operating conditions defined by self-weight, permanent and temporary loads, snow loads and thermal loads (as well as pressure loads for the RPL). Subsequently, after extracting the eigenvalues (and eigenvectors) using the Block Lanczos algorithm combined with a sparse solver, multimodal seismic response spectrum analyses were conducted specifically for the Reactor Building, the RPL and the Auxiliary Building.

The two RLE spectra (horizontal and vertical) shown above were considered for the Reactor Building and for the Auxiliary Building. For the RPL (housed in the Reactor Building) and for other notable facilities/points identified by the customer the in-structure response spectra (ISRS), calculated from the results of dynamic analyses in the time domain conducted on the Reactor Building for seven different time histories of triplets of accelerations in x, y, z, were considered.

The following procedure was used to determine the ISRS of interest for the RPL. Transient analyses of the Reactor Building model were performed for the seven triplets of acceleration supplied by the customer. These were used to calculate the displacements in the time domain according to the directions defined by the three Cartesian axes, and the accelerations by double derivation. Next, using FFT, ISRSs (as a function of damping equal to 5% of critical damping) were assessed for each significant location of facilities and/or equipment. Then, the three ISRSs (in x, in y, and in z) were determined for each notable point as the average of the seven triads of spectra.

The differences in the structural responses of the Reactor Building were considered negligible when referring to the same equipment in the RPL. Consequent to this assumption, the acceleration spectra applied to the connection points of one equipment are the average of the acceleration spectra of all connection points related to that same equipment.

Regarding the response spectrum analysis of the Reactor Building (which consists of two nearly symmetrical parts separated by an expansion joint) specifically about 9,000+9,000 modes were used, so that a participating mass in the order of 90% of the relative total masses was "activated" for each part.

Therefore, some of the structure's mass is lacking in the dynamic analysis. This was addressing by using the "missing mass method". The high-frequency region of the spectrum ($> f_{ZPA}$ in Fig. 7) has no amplification of the peak acceleration of the input time history. In essence, an SDOF oscillator with a frequency $> f_{ZPA}$ is accelerated in phase and at the same amplitude (acceleration) as the applied acceleration.

A system with a fundamental frequency of $> f_{ZPA}$ is therefore correctly analysed as a static problem subject to a load equal to M times ZPA where M is the Missing Mass and ZPA is the Zero Period Acceleration. This concept can be extended to high-frequency modal responses ($> f_{ZPA}$) of multimodal systems.

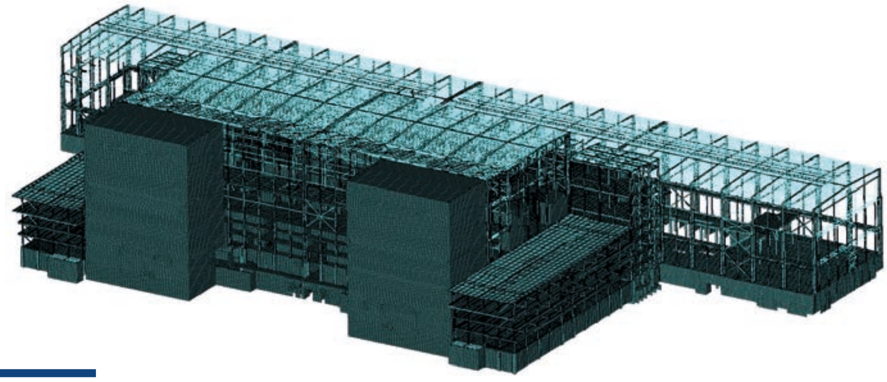


Fig. 2. FEM model (beam and shell) of the Reactor Building, Longitudinal Side Electrical Building, and Turbine Hall.

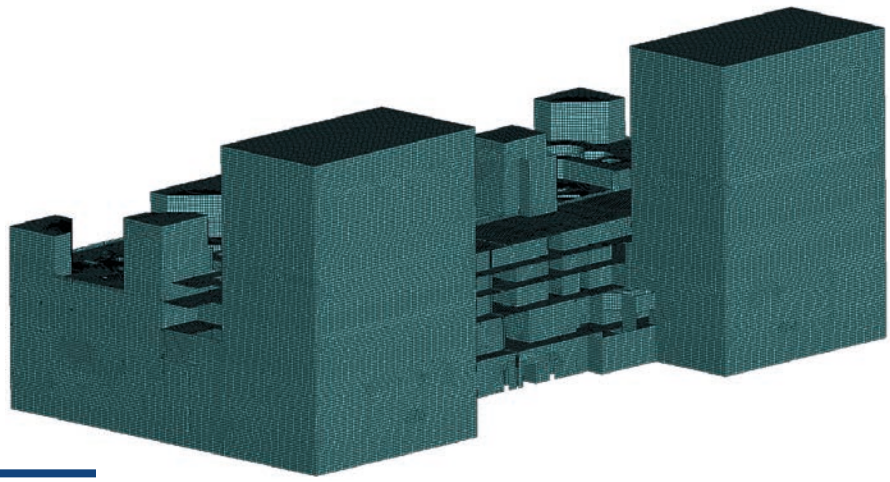


Fig. 3. Reactor Building: shell model of reinforced concrete parts.

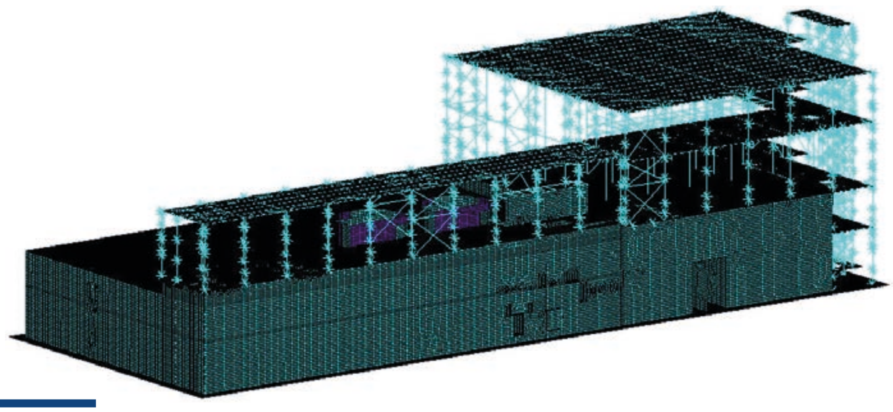


Fig. 4. FEM model (beam and shell) of the Auxiliary Building.

In computational terms, ZPA was assumed to be 0.23g. This corresponds to a frequency of about 30Hz ($T=0.0333s$) if we assume that the response spectrum varies linearly from $T=0s$ $(0.17g)$ to $T=0.1s$ $(0.348g)$. It should be noted here that the highest value of the extracted eigenvalue for the Reactor Building is about 30Hz.

It is common practice, as well as a requirement of some standards (including EN 1998-1 (EC8), which guided the seismic retrofitting of the nuclear power plant) to add some incidental mass eccentricity to increase the distance between the centre of mass and the centre of stiffness.

This is easily resolved and clear for regular structures and/or in situations where the concepts of centre of mass and centre of stiffness are well-defined in general and in different planes. In these cases, eccentricity of mass is often described as an additional torsional moment defined by means of an "additional moment arm" (equal to 5% of the size of the plane perpendicular to the direction of seismic action) being applied to the seismic shear at different levels.

The Reactor Building is anything but a regular structure: it has a variety of partial interplanes, a very uneven distribution of stiffness, and parts of the structure that do not behave like a frame but are closer to a box structure since they somehow extend over more than one floor. As such, concepts such as centre of mass, centre of stiffness, and eccentricity of seismic shear at floor level do not apply to the Reactor Building in a manner that can be unambiguously defined.

This is also confirmed by the analysis of the modal forms: about 9,000 autosolutions (as mentioned above) were considered to capture over 90% of the mass participating in the seismic shear for each of two blocks forming the Reactor Building, and no decidedly dominant modes were found that could suggest how to apply eccentricities to improve torsional behaviour.

Therefore, the EN 1998-1 requirement (to consider accidental eccentricity) was addressed in two distinct phases:

- in the modelling phase, by considering that there is a random error in the quantification of masses, which satisfies or exceeds the effect of any arbitrary eccentricity of 5%;
- in the tension/resistance assessment following the analyses, by carefully examining the status of any critical structural elements (particularly walls and columns in peripheral regions) and suggesting, where appropriate, some feasible modifications to achieve additional safety margins.

To evaluate the seismic response (HCLPF calculation, see below), the loads related to the operating conditions were combined with actions resulting from the RLE earthquake:

- Static load combination: $1.0\ D+1.0\ L+1.0\ E_{stat}$
- Seismic load combination: $1.0\ RLE+1.0\ E_{dyn}$

where:

- D = Permanent loads (including self-weight)
- L = Accidental loads, in concurrent altitude with the presence of the earthquake
- E_{stat} = Static ground pressure
- RLE = Dynamic actions resulting from the earthquake RLE
- E_{dyn} = Dynamic ground pressure due to effects of the earthquake

The $HCLPF$ (high confidence of low probability of failure) approach taken for seismic verification is based on the fact that almost all structures show at least some degree

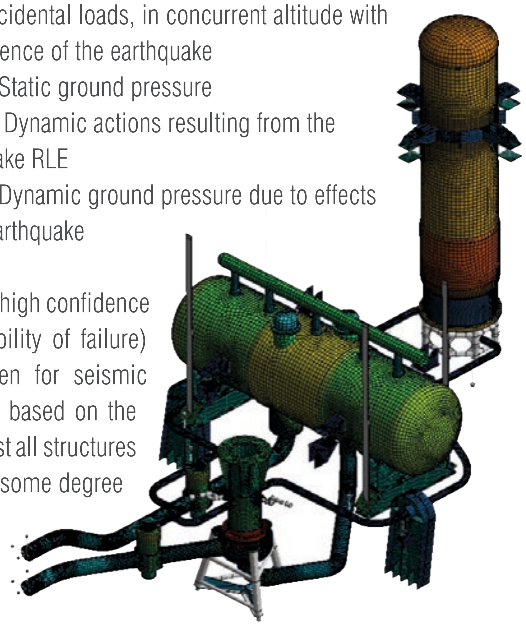


Fig.5. FEM model (beam, pipe, and shell) of the Reactor Primary Loop. All RPL interfaces with the Reactor Building are equipped with dampers, which were appropriately represented in the relevant FE model.

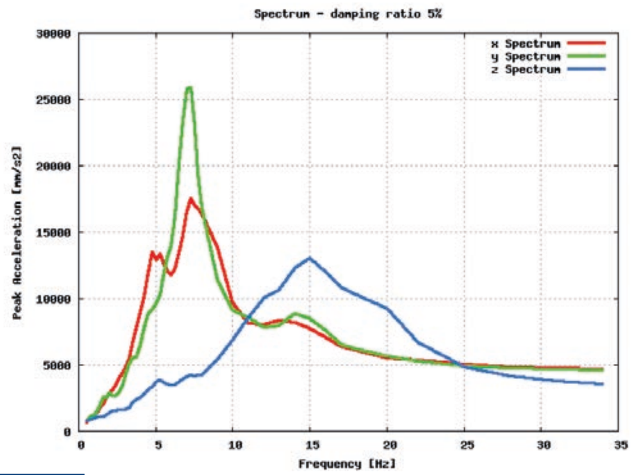


Fig. 6. RPL: in-structure response spectra for one of the notable points.

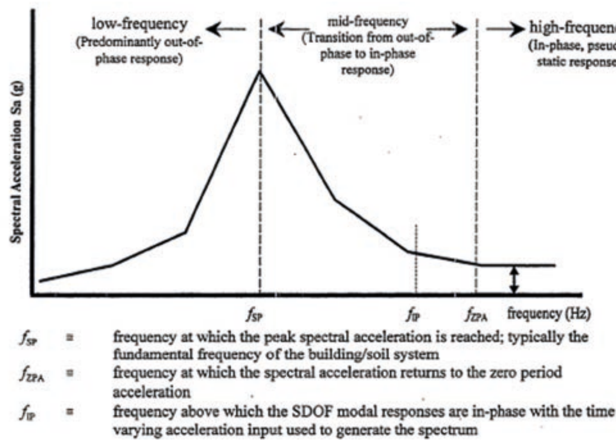


Fig. 7. Definition of ZPA and f_{ZPA}.

of ductility, i.e. the ability to deform beyond the purely elastic limit (ductility is defined by the behaviour factor, also referred to here as ductility factor).

Given the oscillatory nature of seismic motion, the degree of ductility can only increase the seismic margin against failure of structures or components.

HCLPF values for each structural element and for the whole set of structures pertaining to the upgrade, were calculated using the CDFM (Conservative Deterministic Failure Margin) approach as defined by EPRI NP6041-SL "A Methodology for Assessment of Nuclear Power Plant Seismic Margin" August 1991 Rev. 1.
The relationship used in the verifications, which is declined according to the type of structure (reinforced concrete, steel, steel-concrete), is as follows:

$$HCLPF = CDFM = \frac{C_{CDFM} - D_{NS}}{D_{CDFM} + \Delta C_{CDFM}}\ q\ RLE$$

and is derived from the equation “capacity = demand”:

$$(FS)_E D_{CDFM} + D_{NS} = C_{CDFM} - (FS)_E \Delta D_{CDFM}$$

where:

- $(FS)_E = (1/q)(HCLPF/RLE)$ = Elastic scaling factor
- RLE = Review Level Earthquake (0.17g)

- q = Behaviour factor (or ductility factor)
- C_{CDFM} = Deterministic capacity of the section being checked
- D_{CDFM} = Deterministic elastic seismic demand calculated at RLE level
- D_{NS} = non-seismic demand for all non-seismic loads in the load combination
- ΔC_{CDFM} = Reduction in section capacity due to concurrent seismic loads

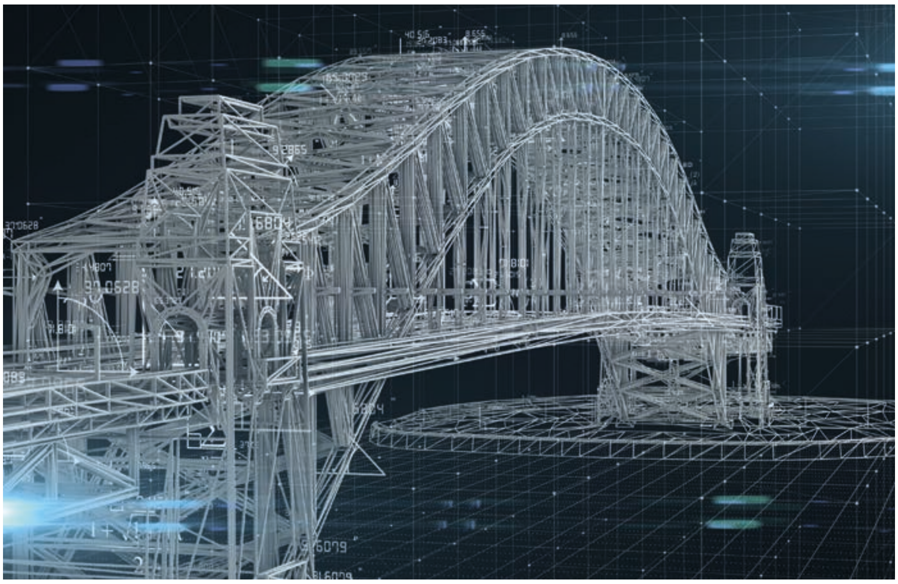
With reference to the behaviour factor q, the following conservative choice was made for all failure modes:

- $q = 1.25$ for systems/plants with dominant frequencies < 8Hz
- $q = 1.00$ for systems/plants with dominant frequencies > 33Hz
- q varying linearly between 1.25 and 1.00 for systems with dominant frequencies > 8Hz but < 33Hz

Final conclusions

There can be no doubt that the introduction of Computational Analysis to Structural Engineering has greatly influenced the development of the design phase, not only in terms of calculation speed, but also in the procedural approach.

The focus of this important innovative and "evolutionary" phase is certainly structural modelling, to be understood as the process by which a structure and the actions acting on it are reduced to a more or less simplified virtual prototype.



Use of the virtual representation of real behaviour is necessary because structures are generally remarkably complex physical systems whose behaviour is influenced by a large number of variables. But implementing a structural scheme that is both "lean" enough to be easily calculable and complex enough to consider the effect(s) of the most important variables is a crucial problem of structural design (or of the redevelopment and retrofitting of existing structures) since both the numerical accuracy of the analyses and the reliability of the results depend on its implementation. Therefore, what is needed is a "digital strategy" (modelling and simulation) that not only considers what needs to be studied/designed, but also the tools, methods, models, data and IT infrastructure available.

This remains the major task of the Structural Engineer who must be able to operate at different levels of complexity and make choices to ensure that representativeness and reliability are not affected by approximations that relate more to decision-making than numbers.

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Without stones there is no arch… We are the stones… and we can build the arch!

Ponte della Maddalena at Borgo a Mozzano (LU).