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Study of the seismic behaviour of GEN III plant: the influence of ageing

R. Lo Frano, S.A. Cancemi
University of Pisa
Lg Lucio Lazzarino
56122, Pisa, Italy
rosa.lofrano@ing.unipi.it

P. Gehl
BRGM
3 avenue Claude-Guillemin, BP36009
45060 Orléans, France

E. Foerster
Paris-Saclay University - CEA
Blg. 454 Room 226
91191 GIF-sur-YVETTE Cedex, France

ABSTRACT

External events are a significant source of hazards to nuclear power plant (NPP) operation, and for this reason it is of meaningful importance to investigate the existing operational nuclear reactors, even aged, in order to verify their structural capacity adequacy.

This study aims at investigating numerically, by means of a deterministic approach, the dynamic behaviour of the structures, systems and components (SSCs) relevant for the safety of the plant when subjected to earthquake event.

A quite refined finite element model (FEM) was thus set up and implemented considering suitable aged/not aged materials behaviour and constitutive laws for the SSCs material. Damping ratios of building materials of height >50 m (reinforced concrete and steel) have been adopted as well.

Fifty acceleration time histories, representative of different soil conditions, were inputted to the transient analyses.

The obtained results were used to appropriately check mainly the considered NPP containment strength reserve.

Results seem to confirm the overall containment reliability even though buckling could affect some internal components. It was observed also that ageing induces a reduction of the plant structural capacity by about 20%.

1 INTRODUCTION

The long-term operation (LTO, see Figure 1) of nuclear power plants (NPPs) is a topic of growing interest as many countries using nuclear energy are committing to ambitious

decarbonisation targets [1]. During their lifetimes, most of the plant structures, systems and components (SSCs) are replaced generally as part of normal maintenance procedures.

There are, however, some SSCs, such as the reactor pressure vessel (RPV) and concrete containment structures, for which the replacement may be unfeasible for technical and/or economic reasons. Consequently, an evaluation of critical life-limiting components is felt necessary to have a clear understanding of the main effects of the operational loads conditions on the components that shall guarantee the safe nuclear operations.

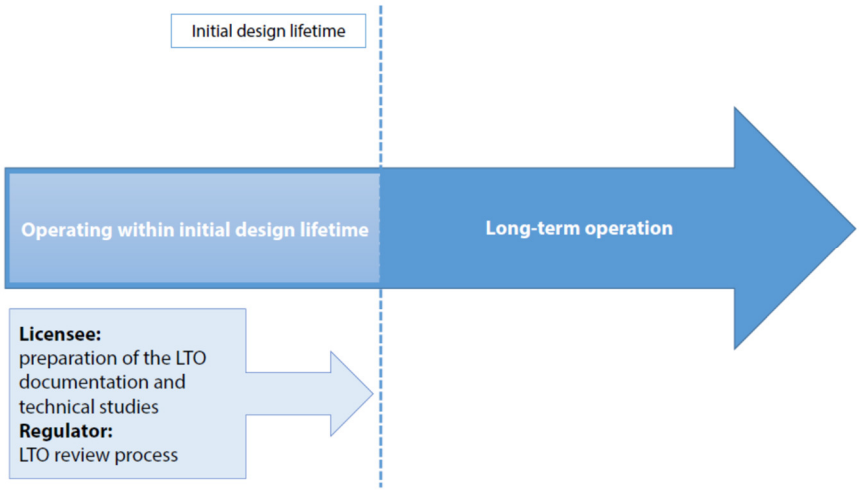


Figure 1: Generic definition of the time frame known as the LTO period

In this study the seismic analysis of a Gen III NPP, that is required as one of the conditions for the design and construction approval, is presented.

Earthquake resistant design of structures requires realistic and accurate physical and theoretical models to describe the response of nuclear SSCs that depend on both the ground motion characteristics and the dynamic properties of the structures themselves: to improve the plant design, and assess its vulnerability, the dynamic behaviour of structures subjected to critical seismic excitations that may occur during their expected/beyond service life must be evaluated. This evaluation is complex and requires the knowledge and understanding of several factors, such as type and magnitude of the site-dependent earthquake event, NPP material behaviour, plant geometry, restraints fastening and/or anchorage system, ageing, etc.

Ageing issues require a comprehensive evaluation of the actual conditions of the plant in order to identify the most critical ageing-related phenomena during the foreseen LTO period. While the main ageing mechanisms are generally common across all nuclear power plants, there can be specific concerns depending on the type of design, and the operational history and experience of the plant.

Therefore a seismic study that accounts for ageing effects becomes extremely relevant especially for plants with an average age of more than 30 years [1], because it contributes to a further reduction of the plant structural capacity.

2 STRUCTURE AND MODEL DESCRIPTION

The standard design of a light water reactor (LWR) refers to an envelopment of site conditions such that the NPP would be suitable for construction on any given site without necessity of site specific analysis and design.

A generic III+ pressurized water reactor (PWR) equivalent to a 1300 MWe with large dry containment, proposed in the EU NARSIS project [2], is considered. The heat source and main

part of the plant is a reactor core with thermal power 4500 MWth. The core is located inside the leak-tight RPV being a central part of the four cooling circuits.

Figure 2 shows a vertical section of the considered PWR plant, whose characteristics and technical details are described in the Del. 4.1 of [2]. It may be assumed as founded on rigid foundation at the base, which joints the inner structures to the top soil deposit[3][4].

The containment of the PWR reactor consists of an outer containment and an inner containment. The outer containment shell is a reinforced concrete structure with large wall thickness and protects the inner containment from direct effects of external hazards. The leak-tightness function is ensured by a steel liner on the inner surface of the containment that is anchored in the inner containment wall by L-profiles (so-called “continuous anchors”) and by headed studs.

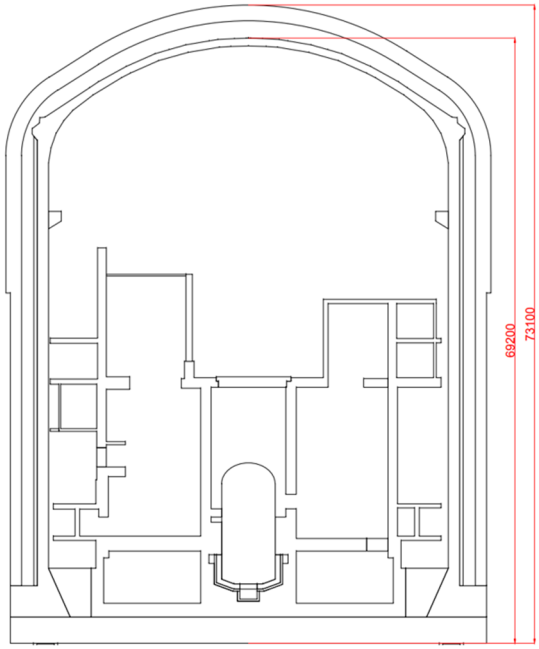


Figure 2: Overview of the inner and outer containment. The inner containment diameter is about 46 m

The inner containment structure consists of the base slab, the cylindrical part and the dome part. The base slab is connected to the cylindrical part by the gusset area in which the wall thickness increases considerably.

Cylindrical and dome part are joined by a ring beam, the function of which is to withstand the bracing forces caused by the dome and to enable the tendons anchoring. The cylindrical part has an inner diameter of about 46 m. The wall thickness of the dome part is about 1.0 m. The total height from the basement to the dome centre of the external containment is about 73 m.

High strength class for both concrete and steel reinforcement is considered [5][6]: the tensile stress value is about one-tenth to one-fifth the compressive strength (5.6e+07 Pa) for concrete.

Steel reinforcement consists of plain carbon steel bar that typically conforms to ASTM A615 or A706 specifications. The minimum yield value of A615 ranges from 280 MPa (Grade 40) to 520 MPa (Grade 75), while 420 MPa (Grade 60) is the most common value considered for steel bars. Steel reinforcement is also used in compression members to safeguard against the effects of unanticipated bending moments that could crack or even fail the member. Therefore the effectiveness of reinforced concrete as a structural material depends mainly on the interfacial bonding between the steel and concrete and on their thermal expansion.

The containment openings, such as the equipment hatch, the fuel transfer tube, pipe, and cable penetrations, etc. were considered through a reduced equivalent area of the cross section.

In the set up model all the RPV and the steam generators (SGs) internals, as well as the water inventory, were considered as a set distributed masses linked respectively to their reference location.

Table 1 summarises the main mechanical material properties of the SSCs implemented in the NPP model, while Table 2 those for rebars (A 615 grade 60).

Table 1: Material properties of the main NPP components

Item	Young modulus [Pa]	Poisson's ratio [-]	Yield stress [Pa]	Density [kg/m3]
RPV	2.1e+11	0.3	2.4e+08	7800
SGs	2.1e+11	0.3	2.75e+08	7850
Piping	2.1e+11	0.3	2.4e+08	7800
Containment	1.97e+10	0.25	5.6e+07	2000
Anchoring/ support system	2.1e+11	0.3	2.4e+08	7800

Table 2 – Steel rebars properties

Material property	Value
Density [kg/m^3]	7800
Young's Modulus [Pa]	2.1 e+11
Poisson ratio [-]	0.3
Yield Stress [Pa]	375-420 e+6
Elongation to Fracture [%]	> 14

All the material properties were assumed to be time independent. Moreover in the deterministic analyses performed, ageing was considered by assuming a constant reduction of material properties (e.g., strength, σ, or Young' modulus, E, etc.) representative of 30 years' operating life of plant.

According to the IAEA SSG-2 (Rev. 1) [7], conservative assumptions were made in the analysis concerning the availability of plant systems included:

(a) Normal operation systems that are in operation at the beginning of the postulated initiating earthquake event

(b) No safety or mitigating system operation.

(c) Safety features specifically designed for design extension conditions should not be credited in the analysis.

3 NUMERICAL ANALYSIS

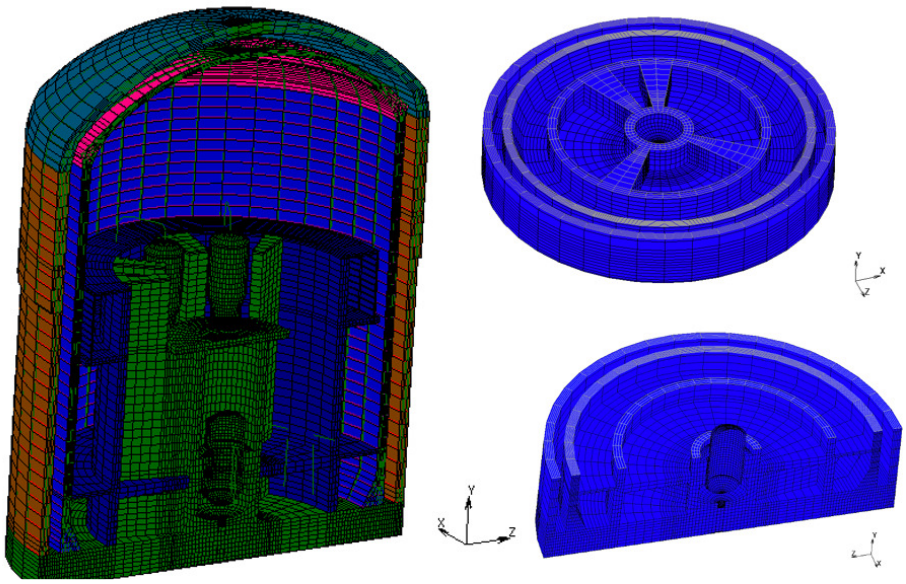
To proceed with the verification of earthquake resistant structures, a deterministic approach is adopted, in the first step. A goal was to develop a representative model of the reference NPP structures, in order to closely simulate their dynamic behaviours as accurately (and completely) as possible.

Figure 3 shows the whole 3D model implemented in the item's element (FE) code MSC.MARC, which is a powerful tool for nonlinear analysis of item behaviour under dynamic and multi-physics loading scenarios. Figure 4 shows the sub-model of the four SGs with joint piping. The whole model consists of more than 130,000 solid elements. Elements' type are thick shell, solid brick, and solid section beam with global displacements and rotations as degrees of freedom. For these elements, stiffness is calculated at their own Gaussian integration points.

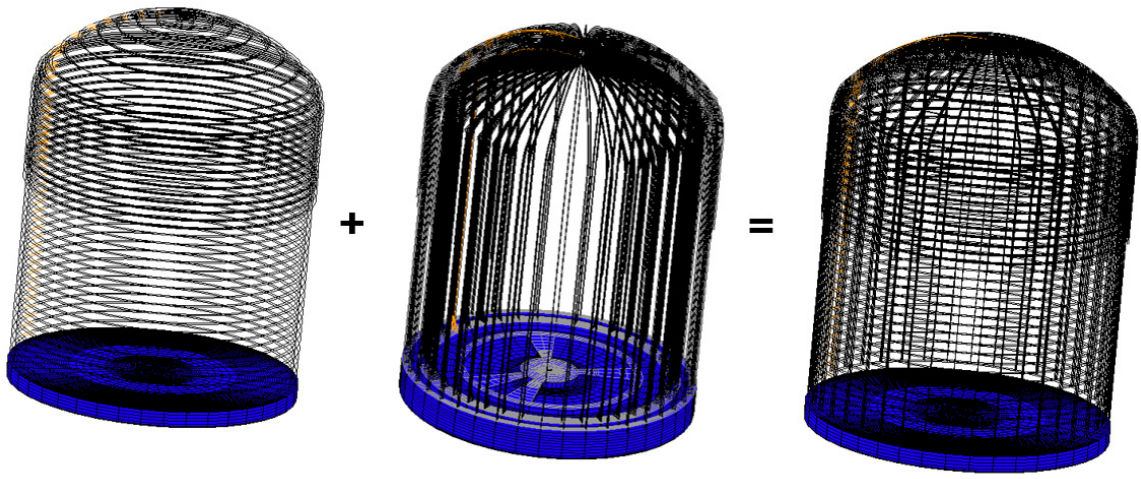
The behaviour of concrete was assumed to be linear elastic up to the point of failure. During a seismic event, the structural components may be subjected to repeated cyclic load reversals and combined axial, flexure and shear effects. Therefore they may undergo inelastic deformations during the severe ground shaking caused by major earthquakes. As a result of accumulated damage during inelastic excursions, the seismic response of reinforced concrete structures may exhibit stiffness degradation and strength deterioration. Consequently, to simulate properly their behaviour, a material degradation method was taken into account in compression and in tension.

Damage is associated with cracking and crushing; in scalar damage theory, the stiffness degradation is isotropic. Compressive stiffness is recovered upon crack closure as the load changes from tension to compression, but the tensile stiffness is not recovered when the load changes from compression to tension. It should be noted that the recovery of stiffness affects bending dynamic behaviour of the damaged structure.

The behaviour of the steel reinforcements, which were embedded into the concrete walls, was assumed to be elastic-perfectly plastic. The thickness of the steel bars has been determined by assuming that their cross-sectional area are uniformly spread along the respective pitch of the layers. The equivalent Von Mises yield criterion and the piecewise linear method have been adopted to describe the plastic behaviour.



(a)



(b)

Figure 3 a, b: Overview of the FE model of Gen III NPP (a). In (b) the horizontal and vertical steel reinforcement arrangements are shown.

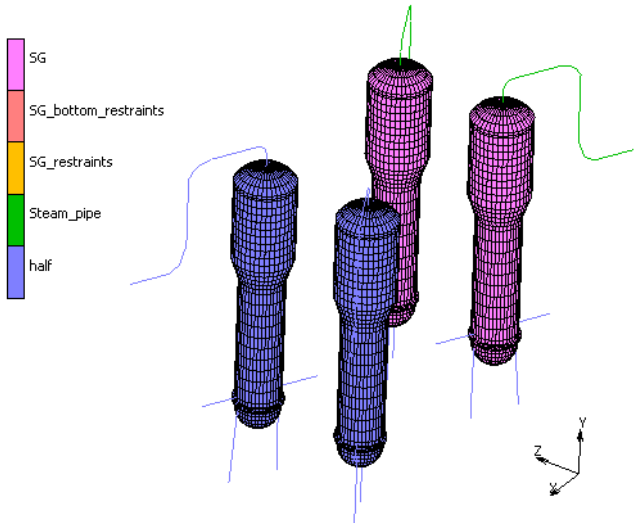


Figure 4: Detail of the NPP model: arrangement of 4 SGs, bottom and lateral lower supports included, and piping. The term half identifies the symmetric part of the systems

4 NUMERICAL INVESTIGATION

After having adequately modelled the main structures in 3D FE models, the seismic analysis was carried out by adopting the time history approach.

The input acceleration data were provided in forms of time histories of accelerations (ATHs) calculated for an appropriate soil damping input ground motion provided at surface level. For the purpose of the seismic response analyses, we used 50 records calculated based on the conditional mean spectrum (CMS) [9]. Each input ATH (provided by BRGM) was obtained from the hazard-site seismic disaggregation analysis for a time-averaged shear-wave velocity to 30 m depth (Vs_{30}) of 800 m/s.

In doing that, site categorization has been considered as [10]:

— Type 1 sites: Vs > 1100 m/s;

— Type 2 sites: 1100 m/s > Vs > 300 m/s;

— Type 3 sites: 300m/s > Vs;

where Vs is the best estimate shear wave velocity of the foundation medium just below the foundation level of the structure in the natural condition, for very small strains.

Sets of mutually orthogonal and statistically independent artificial time history accelerations were used simultaneously as input for the dynamic transient analysis: two horizontal component and one vertical component, in agreement with the R.G. 1.92 rule [11].

The duration of each input ground motion and transient analysis performed is 20 s; the timestep of the transient analysis was 0.01 s.

Before the development of the dynamic transient analysis, modal analyses were carried out to evaluate the natural dynamic behaviour of structures by means of frequencies and modes of all the most relevant components of the considered system with a sufficient number of modes. The understanding of the dominant modes of the plant structures allowed to calculate Rayleigh damping coefficients implemented into the FE model to account for the structural damping. Damping ratios of materials for building of height >50 m were from [12].

4.1 Results Discussion

The results obtained from the simulation in forms of accelerations at some monitoring points show that acceleration amplifies along the containment height, as expected.

The amplification was greater especially in correspondence of the dome cross-section, where, considering the model assumptions, it can be 3 to 4 times higher than the input ground motion (V_{s30} of 221 m/s), due to the overall containment building flexibility in the entire range of frequency of the earthquake.

Figure 5 shows the plots of the acceleration of the worst simulation case, which was characterised by this amplification. The plots of the accelerations were obtained at several monitoring points in/along the internal containment cylindrical to identify the structural weak part of the unaged structure.

Figure 6 and Figure 7 show the comparison of acceleration and equivalent stress of Von Mises for both aged and unaged NPP, respectively, for the only monitoring point located at the roof of the internal containment. The degradation of the material, prolonged over the time, results in a detriment of structural capacity of the plant. This may suggest that local loss of stability could occur.

The plots of the equivalent Von Mises stress (Figure 7) confirm that the greater the reduction of the material strength, the larger the damage suffered by the concrete. Although the local stress is close to the limit strength, the mean stress value was below the one that could determine the failure of the internal containment.

As for the relative horizontal displacement, it may reach a mean value of about 12 mm at e.g. SG elevation, suggesting that structural problems may arise for piping and penetrations.

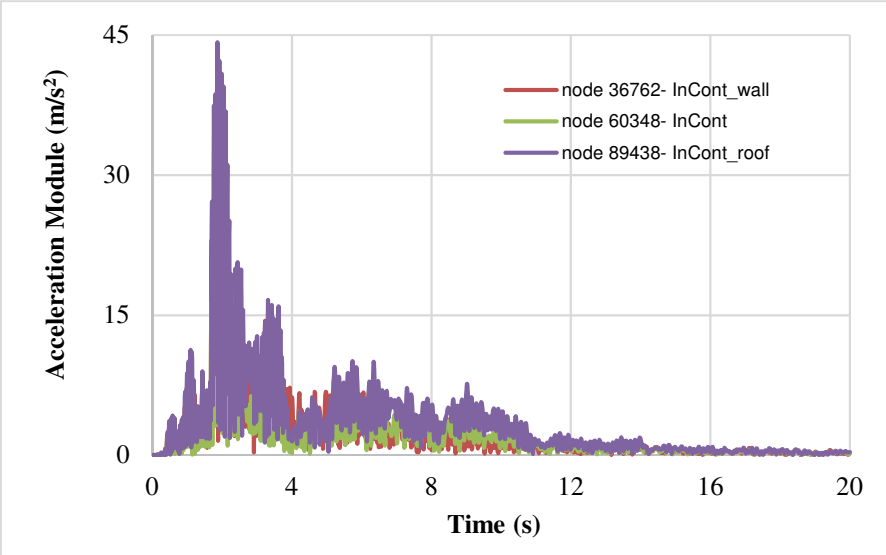


Figure 5: Module of the acceleration for ATH50

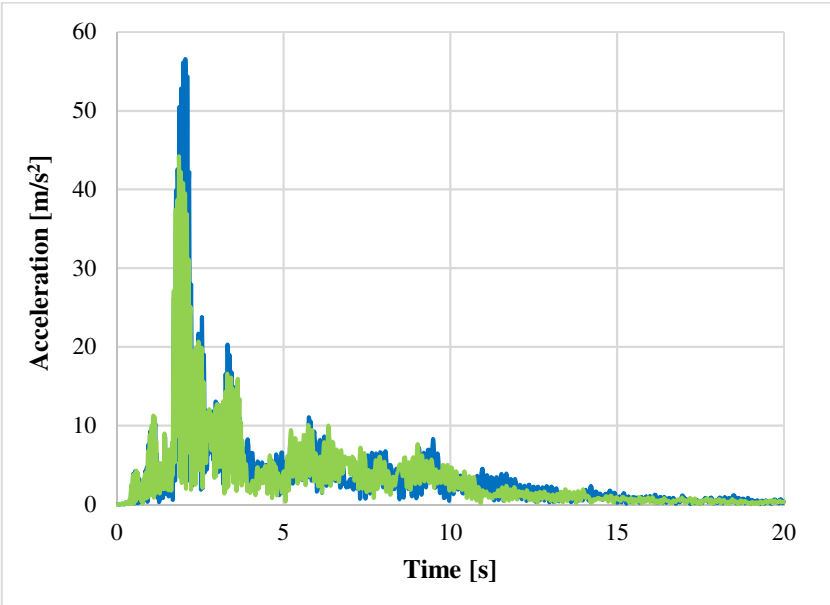


Figure 6: Acceleration comparison for ATH50 between aged (blue curve) and unaged (green curve) NPP at the internal containment hemispherical roof wall.

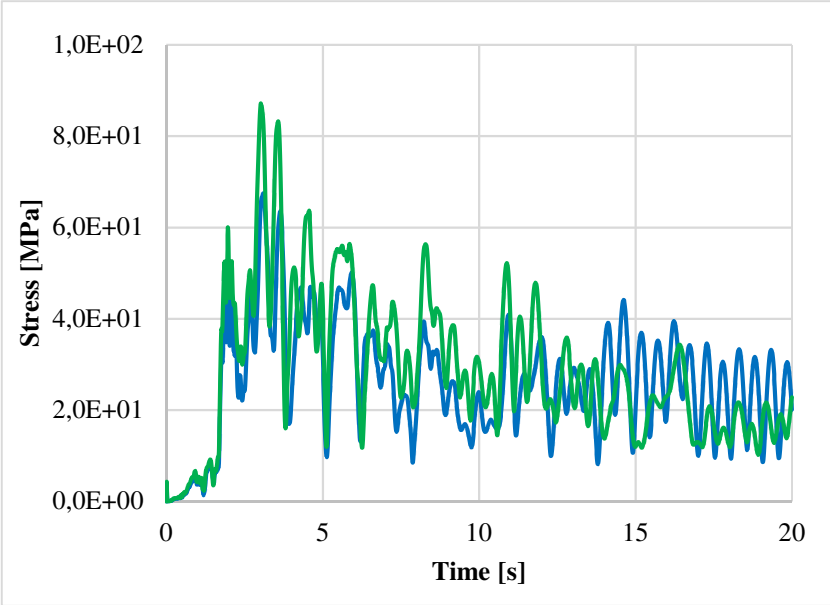


Figure 7: Example of Von Mises stress comparison for ATH50 between aged (blue curve) and unaged (green curve) NPP at the internal containment hemispherical roof wall. For ageing assessment temperature dependent material properties were taken into account.

5 CONCLUSION

A preliminary evaluation of the reliability of Gen III NPP, for the configuration and assumptions made, subjected to earthquake events, having different PGA and acceleration vs. frequency content, was carried out taking also into account the influence of the ageing.

The seismic simulations were performed by adopting a deterministic approach and using a qualified FE code for this purpose.

Results show the following points:

- The greatest amplification of the seismic loadings occurred for an ATH input with Vs30 of 221 m/s.
- The acceleration at the dome of the internal containment was 4 times higher than the input ground motion
- The equivalent Von Mises stress confirmed that the greater the reduction in material strength because of ageing, the greater the reduction in the structural capacity and the larger the damage suffered by the concrete.
- The mean value of relative horizontal displacement was about 12 mm at e.g. SG elevation, suggesting that structural problems may arise for piping and penetrations.

6 REFERENCE

[1] International Energy Agency (2021), Net Zero by 2050 A Roadmap for the Global Energy Sector.

[2] http://www.narsis.eu/

[3] Y. Kitada et al., Models test on dynamic structure–structure interaction of nuclear power plant buildings, Nuclear Engineering and Design 192 (1999) 205–216.

[4] Y. Zhao et al. (1996). Nonlinear 3-D dynamic time history analysis in the reracking modifications for a nuclear power plant", Nuclear Engineering and Design 165,199-211.

[5] R. Lo Frano, G. Forasassi, Preliminary evaluation of the reliability of Gen II or III reactors in BDBE conditions Proceedings of the 2012 20th ICONE20-POWER2012, July 30 - August 3, 2011, Anaheim, California, USA
[6] American Concrete Institute, Code Requirements for Nuclear Safety-Related Concrete Structures, ACI Standard 349-01 and Commentary, 2001.
[7] International Atomic Energy Agency, Deterministic safety analysis for nuclear power plants, IAEA Safety Standards Series No. SSG-2 (Rev. 1), 2019.
[8] R. Lo Frano, G. Pugliese, and G. Forasassi (2010). Preliminary seismic analysis of an innovative near term reactor: Methodology and application, Nuclear Engineering and Design, vol. 240, issue 6, 1671-1678.
[9] J. W. Baker (2011). Conditional Mean Spectrum: Tool for Ground-Motion Selection, J. Struct. Eng., 137(3), 322-331
[10] International Atomic Energy Agency, Geotechnical aspects of site evaluation and foundations for nuclear power plants, Safety Standards Series No. NS-G-3.6, 2004
[11] U.S. Nuclear Regulatory Commission (NRC), 2012. Rev. 3, Regulatory Guide 1.92. Combining Modal Response and Spatial Components in Seismic Response Analysis
[12] H. Bachmanet et al., Vibration Problems in Structures, Birkhäuser Basel, 1995.

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