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SEISMIC MARGIN ASSESSMENT OF PRESSURE BOUNDARY COMPONENTS FROM EXISTING DESIGN CALCULATIONS

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ABSTRACT

Seismic margin assessments (SMA) can often prove difficult due to time constraints, limited documentation and the complexity in component interactions. This document details the work undertaken at the Koeberg Nuclear Power Plant in South Africa aimed at being a simple, conservative calculation methodology that has been utilized for a SMA for pressure boundary components at an increased review level earthquake (RLE). The presented approach is based on the EPRI (1991), EPRI (1994) and DoE guidelines (1997). Considered were also the IAEA (2003) guidelines and the ASME B&PVC - Section III (1980), which was utilized for the design of KNPS, and ASME B&PVC - Section III (2004).

INTRODUCTION

Koeberg Nuclear Power Station (KNPS), a South Africa nuclear power plant, was constructed during the late 1970's. The KNPS is the only nuclear power station on the African continent and is a pressurized water reactor (PWR) of French design. KNPS was designed to a design basis earthquake of 0.3g zero period ground acceleration (zpga), however, recent developments, such as a better understanding of the ground motion response spectra (GMRS) and the events of the Fukushima Daiichi nuclear disaster on 11 March 2011, have driven discussions whether the design basis ground motion may differ from that considered in the plant design basis.

A Review Level Earthquake (RLE) in the seismic margin assessment (SMA) was determined based on discussions with Eskom on potential cliff edges of the KPNS, and consideration of the guidance provided in EPRI (1994), in the context of maintaining the cooling of the fuel in the reactor and spent fuel pool. In particular, tanks that could be used as sources of water to supply the steam generators after a beyond design basis seismic events were considered. At this stage it was concluded that an RLE in the region of 0.5g zpga would be suitable.

The primary objective of the SMA is to make an overall safety assessment of the seismic capability of the plant to define the level of ground motion that the plant is capable of withstanding without experiencing core damage. The process of a SMA is well-documented EPRI (1991) and an accepted method for determining the seismic capability of a nuclear power plant. The SMA process includes the identification of a review level earthquake, the identification of critical plant systems and line-ups down to components level, an assessment of the plant seismic design basis, an experience-based seismic walk down of selected components or groups of components by experienced seismic capability engineers, assigning High Confidence Low Probability of Failure (HCLPF) seismic capability values to equipment, screening of equipment and further assessment of equipment that does not meet screening criteria. This particular work presents the SMA undertaken for pressure components, such as piping, heat exchangers, steam generators, pumps and structural supports related to the aforementioned that where identified during a the seismic walkdown. Components were analyzed on the basis of the walkdown findings; either to further assess the component or for a complementary, comparative assessment. The walkdown procedures are presented in a separate paper submitted to Smirt-22, tilted “Seismic Stress Test of Koeberg NPP” by McCormick (2013).

This paper presents the work undertaken in the SMA; the aim is to detail a simple, conservative calculation methodology, undertaken with an international team, with members from South Africa, Russia, Czech Republic, United States of America and the United Kingdom.

OVERVIEW OF THE SEISMIC MARGIN ANALYSIS

There are two methodologies to evaluate the seismic safety for a given structure; one is the seismic Fragility Analysis (FA), and the other is the conservative deterministic failure margin method of analysis (CDFM). The approach detailed in this document utilizes a conservative deterministic failure margin (CDFM) approach detailed by EPRI (1991) and, for instances where design analysis documentation is limited, a combined CDFM FA approached detailed by EPRI (1994) is utilized. It is worth noting that since KNPS is of a French design, the CDFM methodology does not typically apply, however, if used cautiously the CDFM methodology can provide deterministic values for seismic margins.

The SMA presented herein is based on established criteria, design specifications, existing qualification test reports, established basic design characteristics and configurations, and public domain generic data to demonstrate margin over the safe shutdown earthquake (SSE) of 0.5g.

Safe Shutdown Earthquake

For the design basis earthquake, the peak ground acceleration during the SSE is 0.3g in the horizontal direction. It is assumed that both the horizontal components are identical and that the vertical component is 2/3 of the horizontal component and occurs in phase (Eskom, 1980). This SMA assumes a similar response scaled to 0.5g for both horizontal and vertical components, based on the original design basis Dames and Moore (DM) input Ground Motion Response Spectra (GMRS).

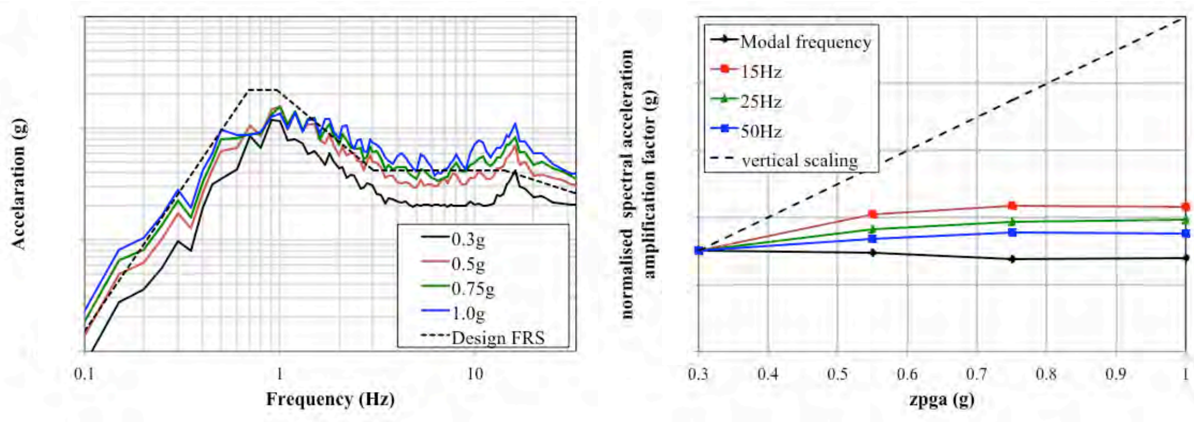
What makes KNPS SMA different to conventional SMA is that the power plant is constructed on multiple neoprene rubber and sliding bearings to partially isolate the Nuclear Island SSC from seismic motions in the horizontal direction. As such, the seismic response cannot be directly scaled in the horizontal direction. Furthermore, the aseismic bearings are capable of sliding, thus introducing an element of non-linearity.

To investigate the response of KNPS at an increased zpga a sensitivity study was undertaken using the original design Stickmodel (NSE, 2012). The aim of the sensitivity study was to show that; (1) the spectral acceleration in the horizontal direction *does not* scale linearly where the vertical *does* scale linearly, and (2) that the vertical spectral acceleration dominates at increased zpga. A brief summary of the results is presented below.

Sensitivity study of Koeberg Nuclear Power Station

The nuclear safety related structures are constructed on a common foundation, referred to as the Aseismic or Nuclear Island (NI). The function of the NI is to adapt the reference station design to more severe seismic conditions required at the Koeberg site, without making major changes to the structures and power plant and so preventing damage during the design basis seismic event. The NI consists of two components, namely the foundation and the NI structures. The foundation consists of a lower raft supported on the soil-cement sub-foundation and bedrock. The lower raft supports reinforced concrete pedestals and the aseismic bearings that, in turn, support the upper raft and the NI structures. The upper raft forms the base to all the nuclear safety related structures on the Aseismic / Nuclear Island.

The sensitivity study utilized three time histories to fit the DM GMRS. These three TH are scaled to four pga’s, namely the design SSE of 0.3g and beyond design basis zpga of 0.5 g, 0.75 g and 1.00 g to investigate the effect of increased seismic loading on the FRS. Additionally, the bearing stiffness is varied by ± 10% of the median stiffness in combination with a bearing friction variation of ± 25%. These



(a) (b)

Figure 1. An example of the sensitivity study output: (a) Comparison of the FRS of the upper raft at various zpga. (b) Normalized spectral acceleration amplification factor vs. zpga of upper raft at the first model frequency of 0.9 Hz, and 15Hz, 25Hz and 50 Hz. The dashed line shows the linear scaled spectral acceleration for comparison.

ranges in neoprene stiffness and sliding couple coefficient of friction are representative of the lower and upper bound specifications on these parameters. This generated 108 models that were analyzed to generate 108 Floor Response Spectra (FRS) per point of interest.

The results of the model calibration are best demonstrated by comparing FRS from the current model (sensitivity study) and the original DM work extracted from reference (Eskom, 1980) as shown in Figure 1a for the upper raft. The analysis shows that the beyond design basis earthquake the seismic response is *dominated* by the vertical component, as the horizontal response supported on the isolated NI is less sensitive to an increased seismic loading (Figure 1b). This means that the horizontal acceleration will not increase above a certain level as the nuclear island starts to slide. The vertical acceleration was however shown to increase linearly as expected.

Major assumptions utilized in the Seismic Margin Assessment

In the SMA, careful consideration was given to those areas that were recognized as important to the plants seismic risk. As aforementioned, components were analyzed on the basis of the walkdown findings; either to further assess the component or for a complementary, comparative assessment. In addition to paying special attention to critical components which have HCLPF values close to the RLE, the SMA considers potential interaction with both safety-related and non-safety-related systems or structures, as well as adequate anchorage load transfer and structural ductility.

The SMA approach relies on conservative assumptions to simplify the analysis significantly, which are stated below. Note, in the event that components cannot be screened out using these conservative assumptions, the assumptions are reviewed and the conservatism is relaxed.

- The seismic demand is calculated by an elastic seismic response analysis using an elastic response spectrum. This means that the vertical and horizontal components are seen to behave linearly and can be scaled linearly to the RLE. As shown above, this is conservative due to the presence of aseismic bearings and this assumption may be relaxed if components are not screened out.
- Seismic demand on equipment supports may be determined from original design calculations or by calculating new seismic demand loads using the equivalent linear static seismic approach.
- The seismic demand calculations are based on a static analysis where dynamic forces are calculated as equivalent maximum static load. This allows for the simple load combinations according to the

following: (1) Only primary stresses are considered for SMA at service level D conditions for pressure boundaries (ASME BPVC Section III). Secondary stresses such as restraint of thermal expansion and support displacements are not included in load combinations due to the relatively low number of seismically induced cycles does not result in failure (IAEA, 1993). (2) The demand is combined with normal operating conditions (NOL) and SSE conditions, which are expected to occur concurrently using load factors of unity. (3) Since SMA is based on a linear analysis, load combinations may be expressed in terms of 'stresses', 'forces', 'moments', 'displacements' or any similar relevant concept. (4) To account for post-elastic behavior, as this is desirable during a SSE (as it acts as the main damping mechanism), an inelastic energy absorption factor is introduced. The inelastic energy absorption factor is a function of strain ductility, i.e. the ratio of permissible inelastic to yield deformation, and is associated with a permissible level of inelastic distortions specified at a failure probability level of approximately 5%.

- The lowest natural frequency of the components is checked. If the natural frequency of the component is known to be high (> 33Hz), then the equipment is considered "rigid" and the zpga is used in the demand calculations. If the natural frequency is known, the maximum acceleration from the response spectrum for the frequency range of interest (from component natural frequency to 33 Hz) can be used instead of the peak. If the natural frequency is estimated (but not known) to be below 33 Hz (this is generally true for piping), the equipment should be considered "flexible" and the peak of the response spectrum should be used as a conservative estimate of the demand.
- No failure due to fatigue is considered due to the relatively low number of seismically induced cycles. The margin in low cycle fatigue failure is incorporated into the ductility factor.
- For piping that is schedule 40 or higher, (which is normally the case for nuclear safety related piping) pipe buckling will not be considered when subjected to earthquake loading. For supports, the buckling capacity is determined using current ASME design acceptance criteria (i.e., ASME B&PV Code Section III, Subsection NF-3322).
- It is assumed that the component is undamaged and no significant erosion or corrosion is taken into consideration.

Failure modes

Failure modes are evaluated during the walkdown process, where the failure modes most likely to be caused by a seismic event are identified. Identification of the credible modes of failure is thus largely based on the analyst's experience and judgment. It is worth noting that the failure mode considers structures to have failed functionally when they cannot perform their designated functions. In general, structures are considered to have failed functionally when inelastic deformations under seismic load are estimated to be sufficient to potentially interfere with the operability of safety-related equipment attached to the structure, or yielded sufficiently so that equipment attachments fail. These failure modes represent a conservative lower bound of seismic capacity since a larger margin of safety against total collapse exists for nuclear structures. Also, a structural failure is generally assumed to result in a common cause failure of multiple safety systems, if these safety systems are housed in the same structure.

CALCULATION OF SEISMIC MARGIN

The calculation method for the available seismic margin predominantly depended on the amount of design calculation information that was accessible. As such, the SMA utilized three approaches, namely; (1) the service level D stresses (NOL and SSE) are known, (2) only the service level D stress is known and a conservative 100% SSE contribution is assumed, and (3) little information regarding the design calculation is accessible (such as for small bore piping) and a combined CDFM FA approach is utilized.

The factor of safety (FS_E) is calculated as the ratio of seismic capacity to seismic demand (D_s). The factor of safety may be calculated using (EPRI, 1991):

$$FS_E = \frac{C - \Delta C_s}{D_s + D_{NS}} \tag{1}$$

where C is the equipment capacity, D_{NS} the concurrent non-seismic demand for all non-seismic loads in the load combination, ΔC_S the reduction in the capacity due to concurrent seismic loadings, and D_S the linear elastic seismic demand.

The elastic safety factor FS_E can be scaled for elastic and permissible inelastic response F_μ. The CDFM is then calculated using the following equation:

$$CDFM = F_\mu \cdot FS_E \cdot PGA_E \tag{2}$$

where PGA_E is the peak ground acceleration of the earthquake for which the FS_E is calculated and F_μ is defined in Table 1 below.

Table 1: Typical F_μ values for seismic evaluation.

Piping (EPRI, 1991)	1.5
Steel component/structure material (EPRI, 1991)	1.25
Cast components such as elbows (EPRI, 1991)	1.25
Nozzles (EPRI, 1991)	1.25
Welds (DoE, 1997)	1.0

It should be noted that PGA_E may be equal to the PGA of the SSE (0.3g) if the seismic demand is calculated from loads determined at SSE level, or it may be equal to the PGA of the RLE (0.5g) if the seismic demand of the SSE is scaled to the level of the RLE with use of scaling factors.

Calculation of component capacity

The component capacity is either defined as the service level D allowable (S_D) or equivalent linear-elastic limit load. That is, level D code allowable is stipulated at 3.0 S_m for piping (but not greater than 2.0 S_y), 2.4 S_m for components, 1.2 S_m for component supports and 1.0 S_m for anchorage (where S_m is the code allowable), or the code yield strength or equivalent linear-elastic limit load is utilized. The load limit is specified at the 95% exceedance probability.

Since components often are welded and provide anchorage (e.g. for piping, tanks, etc.) the SMA requires that a component is assessed for the maximum component stress, the weld stress and the anchorage load. Typically, for piping supports it is safe to assume that support failure is governed by either weld or bolt failure and hence does not necessitate a stress analysis of the support structure. A static linear-elastic analysis is undertaken where normal and seismic loading conditions were considered to determine the available strength factor. Welds were assessed according to Blodgett (1999). Components are modeled using suitable analysis software (Prokon or Strand 7) as three-dimensional beam models to obtain reaction forces and moments at weld joints and anchor points.

Calculation of seismic demand

Approach One: If the SSE and NOL are known the seismic capacity may be simply calculated using Eq. (1) and Eq. (2). Substituting the load combinations as defined by EPRI into Eq. (1) the elastic strength margin factor for piping was calculated using:

$$FS_E = \frac{\min(3S_m, 2S_y) - NOL}{SSE} \tag{3}$$

The CDFM is then calculated using Eq. (2).

Approach Two: In the analysis case where NOL and SSE load is not known (i.e. not sufficient documentation can be found) a conservative approach is used that assumes a 100% SSE contribution in Equation (1). This is conservative. Using Eq. (1) an equivalent strength factor for piping was calculated using:

$$FS_E = \frac{\min(3S_m, 2S_y)}{S_D} \tag{4}$$

where S_D is the given service level D stress. The CDFM is then calculated using Eq. (2).

In the event that the component does not screen out, the NOL operating load is approximated as follows: (1) The dead weight of the component is based on the tributary weight of the component and the supported components and/or adjacent supported spans. It is noted that this is done individually for both horizontal directions and the vertical direction. (2) The center of gravity is calculated based on the tributary weight. (3) The equivalent acting/supported forces are calculated based on the mass and the spatial acceleration. It is noted that if the natural frequency is estimated to be higher than 33 Hz the zpga is taken as the spatial acceleration. If the frequency response is below 33 Hz the peak spectral acceleration is used.

$$\begin{aligned} F_x &= m_x a_x \\ F_y &= m_y a_y \\ F_z &= m_z a_z \end{aligned} \tag{5}$$

where m_x, m_y, and m_z are the tributary weight in both horizontal directions and the vertical direction respectively, and a_x, a_y and a_z are the respective spatial accelerations. The equivalent acting/supported moments are calculated based on the mass, center of gravity and the spatial acceleration.

$$\begin{aligned} Mx &= F_y \cdot l_y + F_z \cdot l_z \\ My &= F_x \cdot l_x + F_z \cdot l_z \\ Mz &= F_x \cdot l_x + F_y \cdot l_y \end{aligned} \tag{6}$$

where l_x, l_y, and l_z are the distances to the centre of gravity from the analysis point to the center of gravity for both horizontal directions and the vertical direction respectively.

An equivalent NOL stress may be calculated using the weight of the component, where the SSE is calculated using

$$SSE = S_D - NOL \tag{7}$$

Approach Three: In the analysis case where deterministic calculations are not possible, a hybrid method, suggested in EPRI, was utilized .The main feature of this method is the development of seismic fragility using a complementary HCLPF capacity (Kennedy, 1984 and EPRI, 1994).

The capacity of the component is determined using the CDFM method by assuming a NOL and SSE load ratio of the level D stress. Expressing the NOL approximations in terms of the allowable stress and allowing for variations, the NOL range for piping systems has been found to be about 0.55 to 0.95 times the allowable stress with a median value of 0.75. Using Eq. (1) an equivalent strength factor for piping was calculated using:

$$FS_E = \frac{\min(3S_m, 2S_y) - 0.75S_D}{0.25S_D} \tag{8}$$

where S_D is the given service level D stress.

The uncertainty resulting in the approximation determined the values subsequent HCLPF calculation. The median capacity A_m consists of three parts (EPRI, 1994); of a median capacity factor F_C, the structure response factor F_{RS} and the equipment response factor F_{RE}:

$$A_m = F_C \cdot F_{RE} \cdot F_{RS} \cdot PGA_E \tag{5}$$

where $F_c = F_\mu \cdot FS_E$.

The high confidence low probability of failure (HCLPF) value was calculated using:

$$HCLPF = A_m e^{-2.33\beta_c} \tag{10}$$

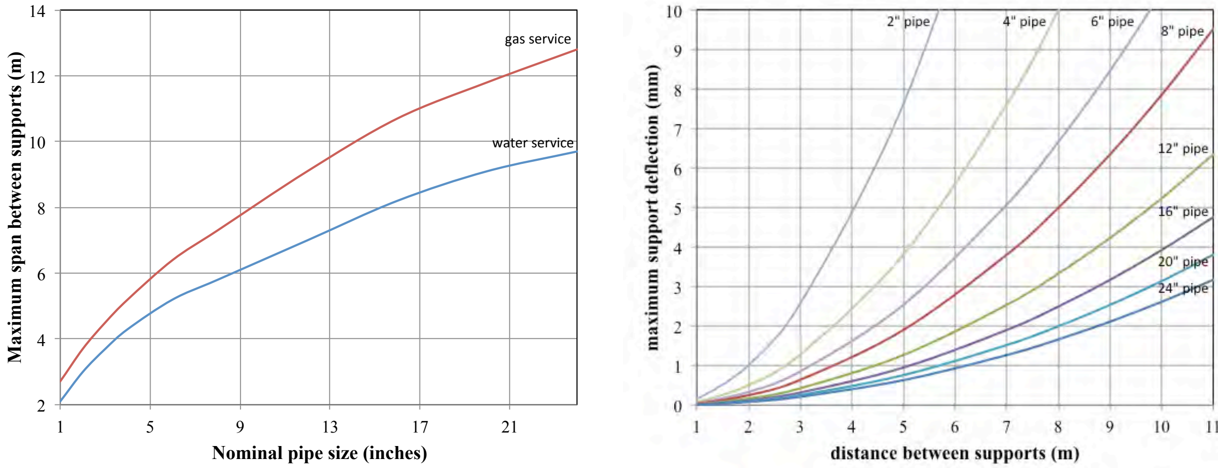
where β_c is the combined factor representing randomness and uncertainty, which are summarized in Table 2.

Table 2: Uncertainty values for combined CDFM and HCLPF methodology for KNPS.

	Factor	β_c
FS_E	1	0.62
F_{RE}	1.31	0.20
F_{RS}	1.86	0.32

Excessive pipe lengths and differential displacements

Potential excessive pipe lengths may results in large deflections that result in high bending stresses and potential failure. Stevenson (2011), based on the ASME code, provided recommendations for supports spacing to limit deflections in the piping to less than 4 mm between supports. These limits are based on a maximum combined bending and shear stress of approximately 16MPa and are presented in Figure 2a. It is noted that Figure 2a is based on Stevenson notes and the assumption of a beam with a uniform loading (due to the seismic acceleration). Furthermore, potential excessive differential support structures may results in large deflections that result in high bending stresses and failure. This is particularly important for piping which bridges the NI and the fixed ground. The limits for deferential support structure displacements are given in Figure 2b for piping (filled with water) and piping supports, however, may also be unitized for other structural members. The data is based on a maximum combined bending and



(a) (b)

Figure 2. (a) Suggested maximum spacing between pipe supports for horizontal straight runs of standard and heavier pipe at a maximum operating temperatures of 400 °C [1,2]. (b) Maximum support deflections for various pipe diameters at a maximum operating temperatures of 400 °C [1,3].

shear stress of approximately 16MPa, similar to Stevenson's approach. It is noted that the Figure 2b is based on the assumption of a beam with a uniform loading (due to the seismic acceleration) plus an additional differential displacement load between the supports[1].

RESULTS AND DISCUSSION

Based on the information that was gathered and analyzed during the SMA, a screening process was undertaken to assess the SMA of pressure boundary components at KPNS. Most of the components included in the assessment were screened out at a RLE associated with a PGA of 0.5g. In comparing the components inspected during the walkdown to other plants, the walkdown specialists made the observation that the plant design is generally rugged although some 'easy fix' issues require attention.

A key difference between KNPS and all other plants is that the power plant is constructed on multiple neoprene rubber and sliding bearings to partially isolate the Nuclear Island SSC from seismic motions in the horizontal direction. This results in the peak spectral accelerations for the horizontal direction to occur at 0.9Hz. Based on the sensitivity study, it was shown that the special accelerations do not scale linearly (where the vertical ones do scale linearly), where the lower frequencies at around 0.9 Hz are not affected by an increased zpga. Furthermore, the bearing displacements at increased zpga of 0.5g are not of concern. It is important to note that the walkdown team considered the effect of the NI when performing the inspection, especially on services crossing the NI, and the effect of impact between NI and surrounding structures.

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1 Does not apply when there concentrated loads between supports such as flanges, valves, specialities, etc.
2 Please see notes from Stevenson (2011).
3 The deflection calculations are based on Stevenson's limit deflections in the piping to less than 0.125 inches (3.2 mm) which is based on the maximum combined bending and shear stress of approximately
2 Please see notes from Stevenson (2011).
3 The deflection calculations are based on Stevenson's limit deflections in the piping to less than 0.125 inches (3.2 mm) which is based on the maximum combined bending and shear stress of approximately
2300psi (ASME B31.1) and 1500 psi (B&PVC Section III, Subsection NF).

The methodology outlined in this work was simple in that the screening was done as a top down approach. The level of detail required in the SMA was kept to minimum to avoid timely Finite Element Analysis and other lengthy assessment methods. The simplicity of the SMA lies in the database created during the walkdown and subsequent findings; either to further assess the component or for a complementary, comparative assessment. The walkdown procedures are presented in a separate paper submitted to Smirt-22, tilted "Seismic Stress Test of Koeberg NPP" by McCormick (2013).

CONCLUSION

The methodology presented was undertaken for the KPNS SMA. The SMA process utilized was shown to be a methodology that allows for simple, yet conservative in this SMA. The intention was to have a SMA approach that allows for the presence of the aseismic bearings that can be utilized to screen out components and equipment. In this particular paper the focus has been on the pressure boundary components and supports.

It is further worth pointing out that the analysis undertaken showed that stresses induced by seismic differential displacements can be included in assessment to obtain a more realistic seismic margin of members and supports under seismic loading.

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