AN APPROACH TO EVALUATE THE HCLPF CAPACITIES IN THE MODIFICATION DESIGN OF SSCs

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Abstract

High-Confidence-of-Low-Probability-of-Failure (HCLPF) capacities of critical Structures, Systems, or Components (SSCs) are used to determine the core damage frequency of a nuclear power plant in the Seismic Probabilistic Risk Assessment (SPRA) or the Seismic Margin Assessment (SMA). To maintain the validity of the conclusions in the SPRA or SMA, the minimum HCLPF capacities need to be satisfied during the modification design of these critical SSCs.

Generally, after the modification design of these critical SSCs, a seismic assessment is performed to determine their HCLPF capacities. However the design may need to be modified several times before the minimum HCLPF capacities are met. To improve the design's efficiency, a simplified approach based on the Conservative Deterministic Failure Margin (CDFM) method is proposed. In this approach, the minimum HCLPF capacities of SSCs can be satisfied in the modification design process by adding load combinations that are related to the minimum HCLPF seismic demand.

Keywords: HCLPF, CDFM, SPRA, SSCs, modification design.

1. Introduction

High-Confidence-of-Low-Probability-of-Failure (HCLPF) capacities of critical Structures, Systems, or Components (SSCs) are used to determine the core damage frequency of a nuclear power plant in the Seismic Probabilistic Risk Assessment (SPRA) or Seismic Margin Assessment (SMA). In the SPRA or SMA of an existing nuclear power plant, the plant core damage frequency and the related HCLPF capacities of the critical SSCs were determined. To keep the core damage frequency and the conclusions in the completed SPRA or SMA valid without reassessing the entire plant after each SSC modification, the minimum HCLPF capacities are usually specified for the SSCs to be modified. These minimum HCLPF capacities need to be satisfied during the modification design process.

Generally, after the modification design of these affected critical SSCs, a seismic assessment is performed to determine their HCLPF capacities. Since the behaviors of SSCs under design conditions may be different from their behaviors under the minimum HCLPF seismic load conditions, their design and assessment may need to be iterated several times before the minimum HCLPF capacities are met.

In this paper, a simplified approach is proposed. In this approach, the Conservative Deterministic Failure Margin (CDFM) method in the SMA methodology [2] is modified based on the required minimum HCLPF capacity of a SSC. The HCLPF load combinations related to the minimum HCLPF seismic demand of the SSC are added to the design process, thus eliminating the additional seismic assessment and the design iterations.

2. The application of CDFM method in SMA and SPRA

The methodology for SMA was developed and documented in EPRI NP-6041 SL [2]. Equations 2-6 to 2-8 in [2] are used to determine the CDFM SME (or HCLPF) level of a SSC. These equations are shown below:

$$(FS)_E = \frac{C - D_{NS}}{D_S + \Delta C_S} \tag{1}$$

$$(FS)_I = \frac{(FS)_E}{K_\mu} \tag{2}$$

$$CDFM\ SME = (FS)_I\ SME_R \tag{3}$$

$(FS)_E$ and $(FS)_I$ are the elastic and permissible inelastic response scale factors for the determination of CDFM SME (or HCLPF). SME_R is the reference seismic margin earthquake represented by the Peak Ground Acceleration (PGA). C is the SSC capacity based on the ultimate strength in the structural codes (ACI, AISC, etc.), service Level D (ASME) allowable stress, or functional limits. D_S is the seismic demand based on the reference earthquake. D_{NS} is the concurrent non-seismic demand (loading) for all non-seismic loads in the SMA load combination. ΔC_S is the reduction in the capacity due to concurrent seismic loadings under the reference earthquake. For example, the shear capacity of a shear wall is reduced by the seismically induced vertical tensile load on the shear wall. K_μ is the ductility reduction factor. It is the inverse of the inelastic energy absorption factor F_μ. F_μ can be conservatively chosen as 1.25 for ductile failure and 1.0 for non-ductile failure.

The methodology for SPRA is documented in report EPRI TR 103959 [3]. An SPRA application guide is provided in report EPRI TR 1002988 [4]. Several clarifications and updates of the methodology in these two reports are provided in report EPRI TR 1019200 [1]. It is clarified in EPRI TR 1019200 [1] that "*In the philosophy for the SMA methodology the HCLPF is defined to be at about the 1% exceedance probability when there is no separation of variability into randomness and uncertainty parts. This is equivalent to the HCLPF being defined at approximately the 95% confidence of about a 5% probability of not being exceeded when the variability is separated.*" Therefore it is concluded that the HCLPF capacities determined based on the CDFM method in the SMA methodology [2] can be used in the SPRA if the target response spectrum shape used in the CDFM evaluation is identical to the target Uniform Hazard Response Spectra (UHRS) in the SPRA. The approach developed in this paper is applicable to both SMA and SPRA.

3. The approach

Equations (1) and (2) can be combined as one equation below:

$$(FS)_I = \frac{C - D_{NS}}{K_\mu (D_S + \Delta C_S)} \tag{4}$$

Equation (3) can be modified to be generally applicable to the HCLPF calculation for SMA and SPRA:

$$HCLPF = (FS)_I \, E_R \tag{5}$$

Where: *HCLPF* is the HCLPF capacity of a SSC. E_R is a reference seismic margin earthquake (SME) for SMA or a reference UHRS earthquake for SPRA.

Equations (4) and (5) can be used to determine the HCLPF capacity of a designed or existing SSC with a known reference earthquake.

When a required HCLPF capacity is specified in the design criteria, its reference earthquake level is also known since the HCLPF capacity of a SSC is based on a reference earthquake; therefore, the ratio between the HCLPF and the reference earthquake is also known. Rearrange equation (5) and use factor F_{HCLPF} instead of $(FS)_I$.

$$F_{HCLPF} = \frac{HCLPF}{E_R} \tag{6}$$

Equation (4) becomes,

$$F_{HCLPF} = \frac{C - D_{NS}}{K_\mu (D_S + \Delta C_S)} \tag{7}$$

Rearrange equation (7),

$$F_{HCLPF} K_\mu (D_S + \Delta C_S) + D_{NS} = C \tag{8}$$

In equation (8), both D_S and ΔC_S are the seismic effects on a SSC under the reference earthquake E_R. For example, the total seismic effect on a concrete shear wall includes the transverse seismic shear force (D_S) and the vertical seismic tensile force ΔC_S.

When a SSC is designed to satisfy equation (8), it meets the HCLPF requirement. When the left side of this equation is less than the right side, the SSC has a HCLPF capacity more than the required HCLPF. Therefore the equation below can be used in the design to ensure that the SSC has or exceeds the required HCLPF capacity.

$$F_{HCLPF} K_\mu (D_S + \Delta C_S) + D_{NS} \leq C \tag{9}$$

In equation (9), the left side can be considered as a seismic load combination with the reference earthquake effects multiplied by a factor of $F_{HCLPF} K_\mu$. $(D_S + \Delta C_S)$ is the total seismic demand under the reference earthquake E_R. This load combination can be added to the other load combinations in the design process so that the designed SSCs will have HCLPF capacities equal or greater than the required HCLPF capacities.

K_μ is the inverse of the inelastic energy absorption factor F_μ, and F_μ can be conservatively chosen as 1.25 for ductile failure and 1.0 for non-ductile failure, therefore the value of K_μ is 0.8 for ductile failure and 1.0 for non-ductile failure. Separate load combinations will be required if a SSC has both ductile failure modes and non-ductile failure modes. For example, a steel frame can be considered as ductile failure, but its anchorage to the concrete is considered as non-ductile failure.

Since equation (9) is derived from the CDFM equations (1), (2) and (3), and the CDFM method is also applicable to SPRA with UHRS seismic input based on Section 2, all the inputs from SMA and SPRA are applicable in equation (9).

4. Example

An electrical cabinet with a height of 2400m, width of 760mm and length of 1000mm is mounted on an elevated concrete floor by four anchor bolts. The base of the cabinet has a strong stiff frame through which the bolts are attached near each corner of the cabinet so that the anchorage capacity is controlled by the anchor bolts, but not by the cabinet base. The cabinet is estimated to weigh about 1100kg. A seismic analysis has been performed for a 1×10^{-5} UHRS reference earthquake with 0.407g PGA. It is estimated that the cabinet is subjected to a 1.3g spectral acceleration force in each of the two horizontal directions, and 0.6g in the vertical direction based on the floor response spectra with 5% damping. The cabinet centre of gravity is estimated to be 1200mm from the concrete floor. The supporting concrete floor has strength of 20MPa. Based on the design criteria, the anchorage is required to have a minimum HCLPF capacity of 0.56g. Design the anchor bolts based on CSA A23.3 [5] Annex D to meet the minimum required HCLPF capacity.

Steps to apply the approach in this design:

i. The anchor bolts in concrete is considered as non-ductile and therefore K_μ=1.0.

ii. Use equation (6):

$$F_{HCLPF} = \frac{HCLPF}{E_R} = \frac{0.56g}{0.407g} = 1.376$$

iii. The total seismic load in each horizontal direction is:

$$V_{HT} = F_{HCLPF} K_\mu \times 1.3g \times 1100kg = 19.30kN$$

The total seismic load in the vertical direction is:

$$P_{VT} = F_{HCLPF} K_\mu \times 0.6g = 8.91kN$$

The total seismic overturning moment in each horizontal direction is:

$$M_{H1} = M_{H2} = 19.30kN \times 1.2m = 23.15kNm$$

To convert the moments into tensile forces P_{H1} and P_{H2}per bolt, the support level arms L_{H1} and L_{H2} for the two horizontal directions shall be estimated. Because the base is a stiff frame, each arm is estimated to extend from the compressive edge of the cabinet to the anchor bolts under tension, as $L_{H1} = 660mm$ and $L_{H2} = 900mm$.

$$P_{H1} = \frac{M_{H1}}{2L_{H1}} = \frac{23.15kNm}{2 \times 660mm} = 17.54kN$$

$$P_{H2} = \frac{M_{H2}}{2L_{H2}} = \frac{23.15kNm}{2 \times 900mm} = 12.86kN$$

The vertical seismic tension force on each anchor is:

$$P_V = \frac{P_{VT}}{4} = \frac{8.91kN}{4} = 2.23kN$$

The horizontal seismic shear force on each anchor in each direction is:

$$V_{H1} = V_{H1} = \frac{V_{HT}}{4} = 4.82kN$$

The dead load compression per bolt $P_{DL}$is:

$$P_{DL} = \frac{-1100kg \times g}{4} = -2.70kN$$

Using the Square-Root-of-the-Sum-of -Square (SRSS) method to combine the seismic load from different directions, the total tension in one bolt is:

$$P_S = \sqrt{{P_{H1}}^2 + {P_{H2}}^2 + {P_V}^2} - P_{DL} = 19.17kN$$

The total shear in one bolt is:

$$V_S = \sqrt{{V_{H1}}^2 + {V_{H2}}^2} = 6.82kN$$

The CDFM tensile and shear capacities of an anchor bolt based on CSA A23.3-04[5] Annex D can be found in HILTI "Limit States Design Guide for Canada", 2013 edition [6]. There is no opening close to the anchor bolts and the minimum anchor bolts space is 660mm. From Table 1 in [6], the factored tensile resistance for a M12 single HDA-P anchor in 20MPa cracked concrete is governed by the concrete pull out capacity. Consider the seismic reduction factor of 0.75 per CSA A23.3-04[5] Annex D:

$$P_R = 0.75 \times 34.9kN = 26.18kN$$

From Table 5 in [6] the factored shear resistance for a M12 single HDA-P anchor in 20MPa cracked concrete is controlled by the bolt shear capacity. Consider the seismic reduction factor of 0.75 per CSA A23.3-04[5] Annex D:

$$V_R = 0.75 \times 20.7kN = 15.53kN$$

The shear and tension interaction ratio for anchor bolt is checked based on CSA A23.3-04[5] Annex D:

$$IC = \frac{P_S}{P_R} + \frac{V_S}{V_R} = \frac{19.17kN}{26.18kN} + \frac{6.82kN}{15.53kN} = 1.172 < 1.2, \text{acceptable.}$$

The requirement of 0.56g minimum HCLPF capacity is satisfied when four (4) HILTI HDA-P Undercut anchor bolts with 125mm minimum embedment are used.

Note that the actual anchorage HCLPF capacity can be calculated using equations (4) and (5). The calculated actual anchorage capacity in this example is 0.572g, which is greater than the required minimum HCLPF capacity of 0.56g.

5. Conclusions

When the required HCLPF capacities are specified for the modification of existing SSCs, the CDFM method can be modified so that the HCLPF seismic assessments can be included in the design process by adding HCLPF related seismic load combinations. This approach eliminates the requirement of the additional HCLPF assessments and design iterations to meet the HCLPF capacities during the design process. This approach is applicable for the specified HCLPF capacities based on both the SPRA and SMA methods.

6. References

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