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SEISMIC PERFORMANCE ASSESSMENT OF A REACTOR STRUCTURE IN A REFINERY COMPLEX

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ABSTRACT

The seismic design and assessment of structures built in refinery installations present a number of challenges to the design engineers considering that the consequences of a seismic event would be severe compared to conventional civil structures. The structures built in these plants can be generally grouped into building structures and non-building structures. A primary structure in steel or reinforced concrete supporting non-building structures such as reactors, equipments etc. known as "coupled structure-equipment" represents a unique class from the perspective of seismic performance assessment. The seismic response of coupled structure-equipment is essentially controlled by the interactions between the supporting structure and supported non-building structures. The paper presents the seismic performance assessment study carried out on a typical gas phase reactor structure consisting of reinforced concrete primary structure supporting on its top a vertical reactor vessel. The structure is adapted from a refinery complex built in a region of moderate seismicity. A coupled model representing the structure and the vertical vessel is used for the study. The seismic response is evaluated using both the nonlinear static procedure and incremental dynamic analysis. The performance evaluation is done using FEMA 356 guidelines. The study shows that the reactor structure satisfies the intended performance objectives under earthquakes with the corresponding hazard levels.

KEYWORDS: Equipment Structure Interaction, Coupled Structure-Equipment, Nonlinear Static Analysis, Incremental Dynamic Analysis, Fragility Curves, Performance Assessment

INTRODUCTION

The seismic design and assessment of various structures including the non-structural components such as piping systems and equipments is vital for refinery installations built in seismic regions considering that a failure would result in financial losses to the owner apart from the societal and environmental impacts. All structures built in a refinery plant except the actual buildings are generally treated as non-building structures (NBS). ASCE 7 [1] provide two categories of NBS namely those similar to buildings and others not similar to buildings. NBS similar to buildings have lateral force resisting systems such as moment frames and braced frames, while the NBS not similar to buildings do not have such systems. The non-building structures such as vessels and equipments can also be supported on primary building structures and comes under the category of "coupled structure-equipment" referred hereafter as "coupled system". The seismic response of a coupled system is largely affected by the interactions between the supporting structure and the supported NBS and accordingly the seismic code regulations which are mainly developed for conventional building structures shall be properly applied to the design of such systems.

The structural engineers responsible for the detailed engineering of coupled systems built in refinery plants normally follow the practice of applying the weight of NBS at preselected locations on the supporting structure without considering the effect of dynamic interactions. The design verifications for NBS performed by their suppliers also do not account for dynamic characteristics of the supporting structure. This practice could be attributed to the stringent project schedule associated with plant engineering followed by the lack of understanding among design engineers in performing seismic assessment of a coupled system. The above design approach neglecting the dynamic interactions between the SS and NBS would ultimately result in majority cases costly designs for both the structure and the NBS. The structural engineers and NBS vendors shall collaborate closely to list out critical NBS that require detailed investigation into the dynamic interactions with supporting structure to optimize the design.

DE-COUPLING CRITERIA FOR COUPLED STRUCTURE-EQUIPMENT

The de-coupling criteria required for the selection of a suitable analysis procedure for coupled systems is specified in various international codes of practice (ASCE 7 [1]; ASCE 4 [2]). Decisions regarding the extent of dynamic interactions between the supporting structure and the NBS shall be taken based on these criteria. The de-coupling criteria provided in ASCE 7 [1] is used in the present study. The code provides three categories of coupled structure-equipment as presented in Figure 1 for a proper choice in the method of analysis & design.

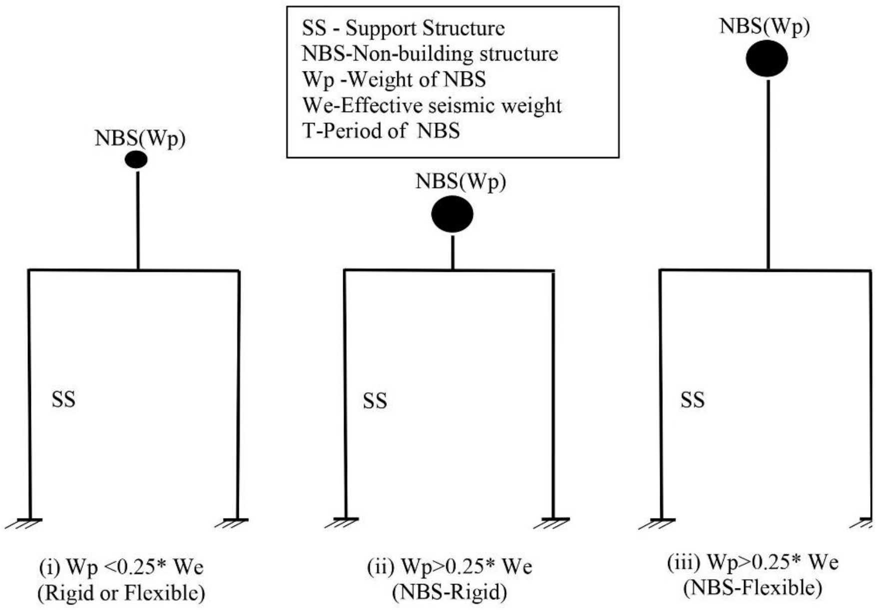


Fig. 1 Categories of "Coupled structure-equipment" ASCE 7 [1]

The de-coupling criteria given in ASCE 7 [1] is based on (i) the weight ratio (R_w) defined as the ratio of the weight of non-building structure (NBS) to the combined effective seismic weights of the NBS and the supporting structure (SS), and (ii) fundamental period (T) of NBS. If R_w is less than 0.25 as shown by case-1, the system can be decoupled and the SS designed as a building structure with the weight of NBS lumped at appropriate location. If R_w is greater than 0.25, the analysis procedure shall be based on the fundamental period (T) of NBS. Where T is less than 0.06 s (case-2), the SS and NBS can be decoupled as in case-1. In cases, where T is greater than or equal to 0.06 s (case-3), the NBS and SS shall be analyzed as a coupled model with appropriate distribution of their mass and stiffness and the seismic performance of SS shall be obtained from a coupled analysis.

Azizpour and Hosseini [3] investigated the ASCE code recommendations for the analysis of coupled systems for a pipe-way structure and found that the seismic response is largely affected by the percentage of piping weight. Prabhakar et al. [4] examined the decoupling criteria for the assessment of nuclear power plant structure using both the coupled and decoupled models. The coupled analysis performed for a nuclear reactor building structure is found to result in lower responses, economical designs and avoid unnecessary costs in the case of seismic retrofitting (Subramanian et al. [5]; Yadira et al. [6]). A new dynamic decoupling criteria based on frequency and response for secondary systems relevant to nuclear industry is proposed (Fouquiau et al. [7]). The engineering report published by the petrochemical energy committee of ASCE (ASCE [8]) provide detailed guidelines along with sample calculations for the base shear estimation of different categories of coupled systems.

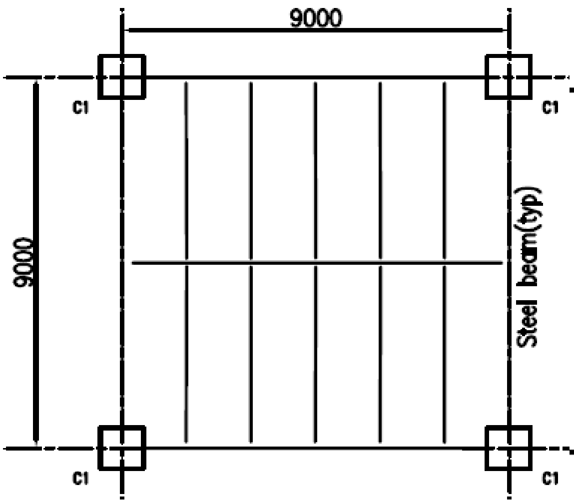
The de-coupling criteria for systems and components in nuclear power plant structures is detailed in ASCE 4 [2]. The de-coupling criteria is based on the mass ratio (R_m) defined as the ratio of total mass of supported subsystem to that of the supporting system and the frequency ratio (R_f) which is the ratio of fundamental frequency of the supported subsystem to the dominant frequency of the supporting system. As per the de-coupling criteria, (i) if $R_m < 0.01$, de-coupling can be done for any R_f , (ii) if $0.01< R_m <0.1$, decoupling can be done if $0.8> R_f >1.25$, and (iii) if $R_m > 0.1$, a subsystem model should be included in the primary system model. If a coupled analysis will not increase the response of key design parameters of the primary system over that of a de-coupled analysis by more than 10%, then also a coupled analysis is not required.

DESCRIPTION OF REACTOR STRUCTURE

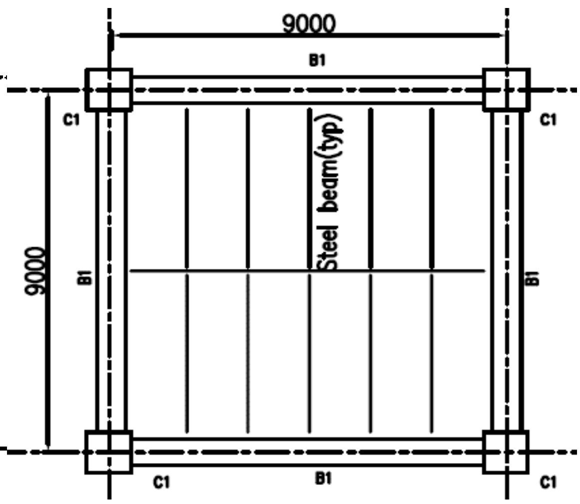
The representative gas phase reactor structure in reinforced concrete (RC) adapted from a refinery complex and considered for the study is shown in Figure 2. The structure with four columns is 9.0 x 9.0 m in plan with a total height of 17.5 m. Steel platforms with grating floors are provided at elevations 4.0, 7.0 and 10.5 m. The elevation 17.5 m consists of a 750 mm thick reinforced concrete deck slab supporting the reactor. The lateral force resisting system for the structure consists of RC moment resisting frames in both directions. The dead load (DL) at elevations 4.0, 7.0 and 10.5 m is taken as 0.50 kPa to account for the self-weight of steel platforms. The live load (LL) on all floors is taken as 2 kPa except the top deck slab at 17.50 m where it is 5 kPa. The total operating weight of gas phase reactor is 4000 kN with its top at an elevation of 47.5 m. The structure was designed for earthquake loads computed as per Eurocode 8 [9] considering a peak ground acceleration of 0.30 g, ground type C and an important class III. The sizes and reinforcement details of columns and beams are given in Table 1.

STRUCTURAL MODELLING

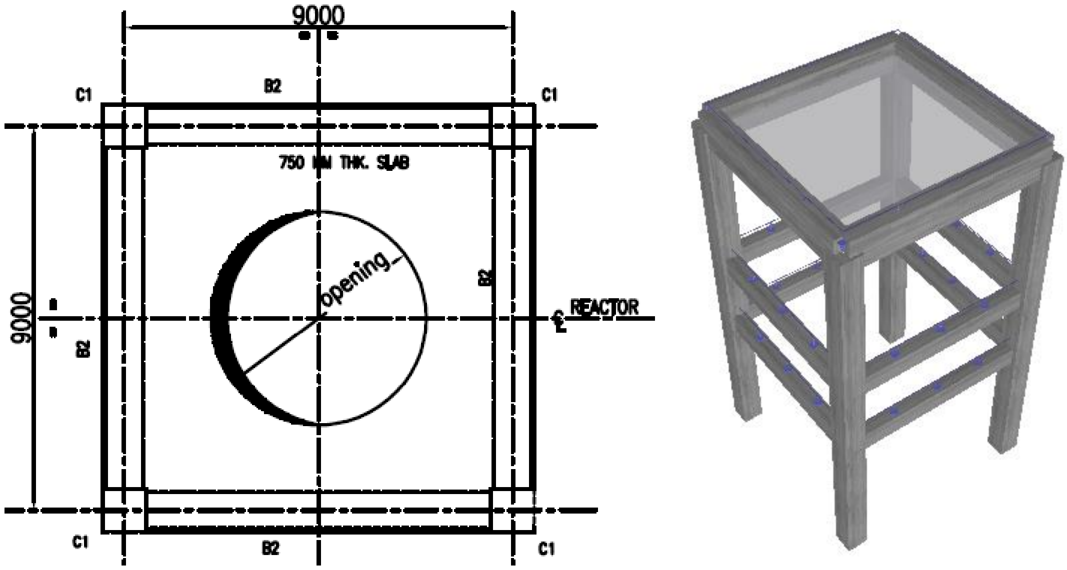
The structural models used for the present study are developed using the finite element package SEISMOSTRUCT (Seismosoft [10]) capable of handling both geometric nonlinearities and material inelasticity. The models use force based (FB) distributed inelasticity elements for column and beam members with cross-section response being simulated by means of the fibre approach assigning a uniaxial stress-strain relationship at each fibre. A schematic representation of the fibre approach for a beam element is shown in Figure 3. Five integration sections are used along the member length with cross section subdivided into 200 fiber elements for section equilibrium computations. The Mander et al. (Mander et al. [11]) nonlinear concrete model is employed for defining the concrete materials. The concrete used has a mean cylinder compressive strength of 28 MPa and a mean tensile strength equal to $(1/10)^{th}$ of compressive strength. The Menegotto-Pinto steel model (Menegotto and Pinto [12]) is used for modelling reinforcement rebars with a yield strength of 500 MPa. Typical cyclic stress strain curves for concrete and steel reinforcement as per the material models are shown in Figure 4. The modelling of RC deck slab at elevation 17.50 m is realized through rigid diaphragms. As the reactor is essentially a vertical cantilever connected to the main structure at a single level, it is represented by a stick model in the coupled analysis using elastic frame element with a circular hollow section, the sectional details being taken from relevant vendor equipment drawings.



(a)



(b)



(c) (d)

Fig. 2 Reactor structure considered for the study: (a) plan at elevation 4.0, (b) plan at 7.0 & 10.5, (c) plan at 17.5, and (d) 3-D model for supporting structure

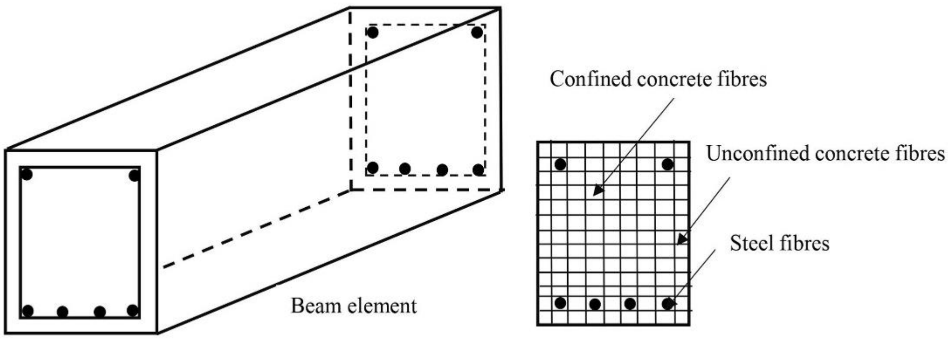
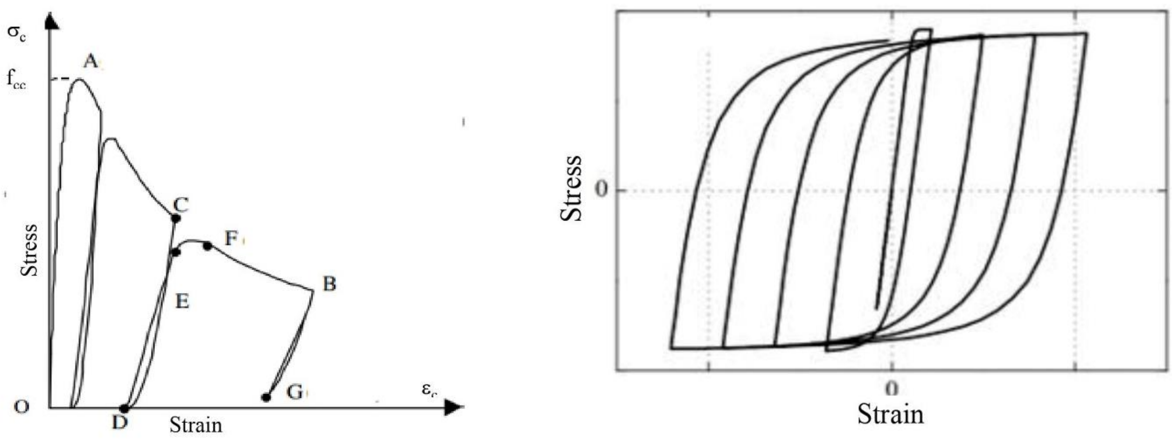


Fig. 3 Fibre approach for a reinforced concrete beam element



(a) (b)

Fig. 4 Typical cyclic stress strain curves: (a) Concrete: Mander et al. model, and (b) Steel reinforcement: Menegotto-Pinto model

Table 1: Section and Reinforcement Details

Beam				column			
Mark	Size (mm)	Long.[b] reinf. (%)	Transverse reinf.	Mark	Size	Long. reinf. (%)	Transverse reinf.
B1	600x900	2.91	ϕ10@200[c]	C1	1000x1000	2.57	ϕ10@200[a]
B2	750x1400	1.12	ϕ10@200[c]				

[a] 4-legged in each direction; [b] longitudinal steel equally distributed at top and bottom; [c] 4-legged

EIGENVALUE ANALYSIS

For the loading considered, the weight of reactor (Wp) is found to be greater than 25% of the combined effective seismic weights of the reactor and the supporting structure (SS). The effective seismic weight on the structure is computed from dead load, operating load of reactor and 25% of live load. The computed natural period of reactor alone is 0.25 sec and is greater than the limiting value 0.06 sec. Therefore, as per ASCE 7 [1], the reactor shall be considered as a flexible equipment and the seismic response shall be evaluated from a model combining the reactor and supporting structure. An eigenvalue analysis is performed on two models, namely a lumped model (Figure 2(d)) wherein only the supporting structure (SS) is modelled with the mass of the reactor lumped on SS and a coupled model as shown in Figure 5 wherein the SS is modelled along with reactor having proper distribution of mass and stiffness. As the coupled model is symmetrical about both horizontal axes, all assessments are performed only in one transverse direction. The summary of eigenvalue analysis results relevant to excitation in transverse direction for the two models is presented in Table 2. It is observed that the lumped model is governed by the fundamental mode alone whereas the coupled model is mainly influenced by two modes. An increase of modal periods, as expected, is also noted in the coupled model. Therefore, the coupled model considering both the supporting structure and the reactor reflecting the actual dynamic characteristics of the system is considered in all investigations.

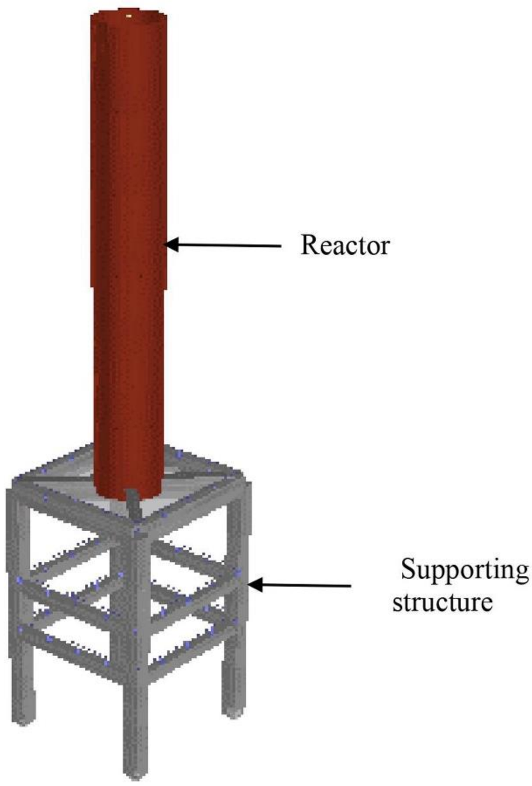


Fig. 5 Coupled model

Table 2: Results of Eigenvalue Analysis

Lumped model			Coupled model		
Mode	Period (sec)	MPR[a]	Mode	Period (sec)	MPR[a]
1	0.72	0.909	1	1.35	0.507
4	0.13	0.073	3	0.49	0.410

[a] MPR= mass participation ratio

NONLINEAR STATIC ANALYSIS

The performance assessment of structures are normally carried out either by a nonlinear static procedure (NSP) popularly known as a pushover analysis or by an incremental dynamic analysis (IDA). In NSP, a mathematical model of the structure that includes all lateral force resisting elements with their nonlinear material behavior adequately represented is subjected to an invariant lateral load pattern that is monotonically increased till the structure is taken to near collapse condition or a precalculated target displacement representing the maximum displacement the structure would experience for a considered seismic hazard level. The stiffness of structure is automatically updated to account for the spread of inelasticity in the structure. The analysis provides a capacity curve in terms of base shear versus control node displacement. The choice of an appropriate load pattern is critical in an NSP for the prediction of seismic response quantities in the case of an earthquake event. At least two lateral load patterns namely a modal pattern and a uniform pattern shall be used in the NSP (FEMA 356 [13]). In a modal pattern, lateral load distribution is kept consistent with the one obtained from an elastic analysis whereas in a uniform pattern lateral forces are proportional to masses regardless of elevation. However, a modal pattern is applicable only in cases where the relevant mass participation exceeds 75% (FEMA 356 [13]).

The lateral load patterns described above are applicable to structures with a dominant fundamental mode and would provide incorrect responses in cases where the higher mode effects are significant. The modal pushover analysis (MPA) includes the contributions of all modes of vibration that significantly affect the structural response (Chopra and Goel [14]). The MPA procedure demands a lot of effort and neglects the coupling of modes. The adaptive pushover method (APM) uses adaptive load patterns that change based on the instantaneous dynamic characteristics of the structure (Gupta and Kunnath [15]). The adaptive pushover methodology may be either force based (Reinhorn [16]) or displacement based (Antoniou and Pinho [17]).

The objective of performing an NSP in the present study, in spite of its inherent limitations, is to obtain an estimate of the seismic demand as well as the structure capacity that could be used as a basis for the accurate estimates by other methods. The NSP for the coupled model uses the method of modal combinations (MMC) (Kunnath [18]; Kalkan and Kunnath [19]). The approach is simpler compared to MPA and APM and considers the higher mode effects in the response calculations. The MMC is based on an invariant lateral load pattern obtained from the factored combination of independent modal contributions. In the MMC procedure, the spatial variation of applied lateral loads is given by the following equation:

$$F_j = \sum \alpha_n \Gamma_n m \Phi_n S_a(\xi_n, T_n) \tag{1}$$

where α_n is a modification factor that can assume positive or negative values; Φ_n is the mode shape vector corresponding to mode n; S_a is the spectral acceleration at the period T_n and damping coefficient ξ_n corresponding to mode n; and

$$\Gamma = \left([\Phi]^T [m][1]\right) \Big/ M_n \quad \text{where} \quad M_n = [\Phi]^T [m][\Phi] \tag{2}$$

If MMC is applied to structures dominated by two significant modes, equation (1) will take the following form

$$F_j = \alpha_1 \Gamma_1 m \Phi_1 S_a(\xi_1, T_1) \pm \alpha_2 \Gamma_2 m \Phi_2 S_a(\xi_2, T_2) \tag{3}$$

For the MMC, the study considers two lateral load patterns resulting from modal combinations namely “mode 1+mode 3 (m 1+m 3)” and “mode 1-mode 3 (m 1-m 3)”. It shall be noted that for the coupled model, the combined participation of modes 1 and 3 exceed 90% of the total participating mass. The values of modification factor α_n is taken equal to 1.0.

The performance levels, qualifying criteria and the corresponding earthquake hazard levels required for the performance based seismic design (PBSD) for both the structural and non-structural components are covered in detail in documents such as FEMA 356 [13] and ATC, A [20]. The study uses the enhanced objectives criteria prescribed in FEMA 356 with the associated performance levels identified as immediate occupancy (IO), life safety (LS) and the collapse prevention (CP). The level IO means very limited post-earthquake damage with all vertical and lateral force resisting systems retaining all of their pre-earthquake strength and stiffness; LS indicates significant post-earthquake damage with some margin against structural collapse and CP indicates the building is on the verge of collapse with a significant degradation in strength and stiffness of lateral force resisting systems. For the verification of performance objectives, a set of earthquake hazard levels are employed having 20%, 10%, and 2% probabilities of exceedance in 50 years and associated elastic spectra are shown in Figure 6.

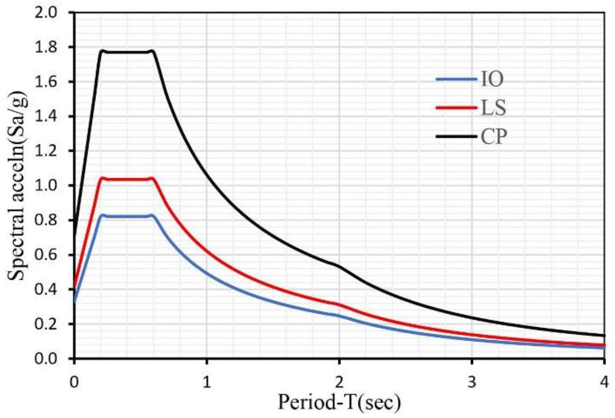


Fig. 6 Elastic spectra for verification of performance levels

The target displacement for different performance levels are obtained using SEISMOSTRUCT which follows the procedure outlined in Annex B of Eurocode 8 [9]. The procedure essentially consists of the development of an idealized elastic-perfectly plastic force-displacement relationship from the capacity curve, computation of an equivalent SDOF system parameters including the spectral displacement, calculation of inelastic spectral displacement and finally its conversion to the target displacement for the MDOF. The control node for displacement monitoring is taken at elevation 17.50 m, the supporting level of the reactor. The capacity curves for the two modal combinations namely mode 1-mode 3 and mode 1+mode 3 are shown in Figure 7. The response quantities such as base shear, roof displacement (target displacement), roof drift and maximum inter-story drift ratio (MIDR) computed for the three performance levels IO, LS and CP are presented in Table 3.

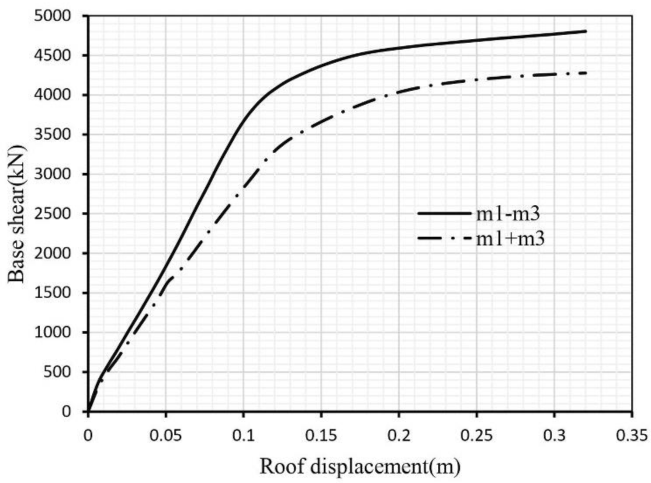


Fig. 7 Capacity curves for the supporting structure using MMC

Table 3: Summary of Response Quantities from NSP

Load Pattern	Performance level	Base Shear(kN)	Roof disp. (mm)	Roof drift θ_r (%)	MIDR (%)
m 1-m 3	IO	3883	109	0.63	0.73
	LS	4293	141	0.81	0.96
	CP	4678	243	1.39	1.82
m 1+m 3	IO	3320	122	0.70	0.83
	LS	3753	160	0.92	1.09
	CP	4233	275	1.58	1.83

INCREMENTAL DYNAMIC ANALYSIS

The nonlinear static procedure (NSP) though is a popular assessment tool used by practicing engineers, it is recommended in structures with their response governed mainly by the fundamental mode with insignificant higher mode effects. The procedure also does not consider the duration and frequency content of earthquake motion and the change in dynamic characteristics of the structure as it is taken through various loading and unloading cycles. Therefore, in the present study, an accurate estimate of the dynamic response required for the performance assessment is obtained through the incremental dynamic analysis (IDA).

IDA is an efficient tool to monitor the response of structures when subjected to earthquakes of varying intensities and estimate dynamic capacity. The method is extensively used in the performance based earthquake engineering (PBEE) of critical structures. The procedure consists in performing nonlinear time history analyses of a structure using a bin of preselected ground motion records. Each record is then scaled in suitable steps to enable the structure to pass through elastic, plastic and finally dynamic instability. The analysis provides a set of capacity curves, each curve corresponding to a ground motion record plotted in terms of Damage Measure (DM) versus the Intensity Measure (IM). The maximum inter-storey drift ratio (MIDR), peak roof drift or the story spectral acceleration could be used as DM, whereas the intensity measure IM could be peak ground acceleration (PGA) or the 5% damped first mode spectral acceleration Sa (T_1, 5%).

A detailed review of IDA covering the theoretical background, characteristics of IDA curves and processing methods is available (Vamvatsikos and Cornell [21]). While the choice of a suitable DM depends on the objective of IDA study, the selection of an adequate IM shall be such as to provide reliable response with least dispersion. Rojit et al. [22] performed a set of IDA on buildings made of cold-formed steel to obtain adequate parameters representing IM. Bojórquez and Iervolino [23] proposed a spectral shape-based intensity measure equal to the geometric mean of spectral accelerations between T_1 and $2T_1$ normalised with Sa (T_1) with an intent to capture the post-yield period lengthening effects. Sufficient number of ground motion records shall be used in an IDA to ensure that the computed seismic responses are fairly accurate. Various studies have shown that IDA performed with ten to twenty number of records could provide sufficiently accurate estimates of seismic demand in the case of mid-rise buildings (Shome and Cornell [24]).

1 IDA Analysis Results

The IDA for the coupled model is performed using SEISMOSTRUCT. A suite of twenty ground motion records with the PGA varying from 0.15 g to 1.0 g is used for performing IDA. The suite corresponds to recorded ground motions with relatively large magnitudes in the range of 6.5-7.5. The duration of records varies from 35-60 sec. For each selected record, the scaling factors are increased in suitable steps to take the structure through elastic, plastic and finally to collapse levels. The capacity curves obtained from IDA for each record in terms of the base shear and the roof drift is shown in Figure 8. It is observed that the capacity curve associated with each ground motion record is unique, the shape of the curve being governed by ground motion characteristics such as frequency content and duration. While majority of capacity curves are simple showing the softening at certain capacity levels, limited number of curves exhibited softening followed by a hardening. Using the multi-record capacity curves, the 16%, 50%, and 84% fractile curves are obtained as shown in Figure 9. For the construction of IDA curves, the spectral acceleration corresponding to the first mode at 5% damping Sa (T_1, 5%) is chosen as the IM and the associated maximum inter-story drift ratio (MIDR) is chosen to represent the

DM. The IDA curves are shown in Figure 10. The 16%, 50%, and 84% fractile curves constructed from IDA curves are given in Figure 11. The summary of MIDR values for different performance objectives are shown in Table 4. The 84% MIDR values computed at different performance levels are found to be within the corresponding maximum limits of 1%, 2%, and 4% prescribed in FEMA 356 for concrete frames.

Table 4: Summary of MIDR Values (%) from IDA

Hazard level	***Sa(g)***	**Performance level**	**16% fractile**	**50% fractile**	**84% fractile**	**FEMA 356 limit (%)**
20% in 50 years	0.366	IO	0.352	0.425	0.513	1.0
10% in 50 years	0.460	LS	0.453	0.542	0.648	2.0
2% in 50 years	0.788	CP	0.835	1.015	1.235	4.0

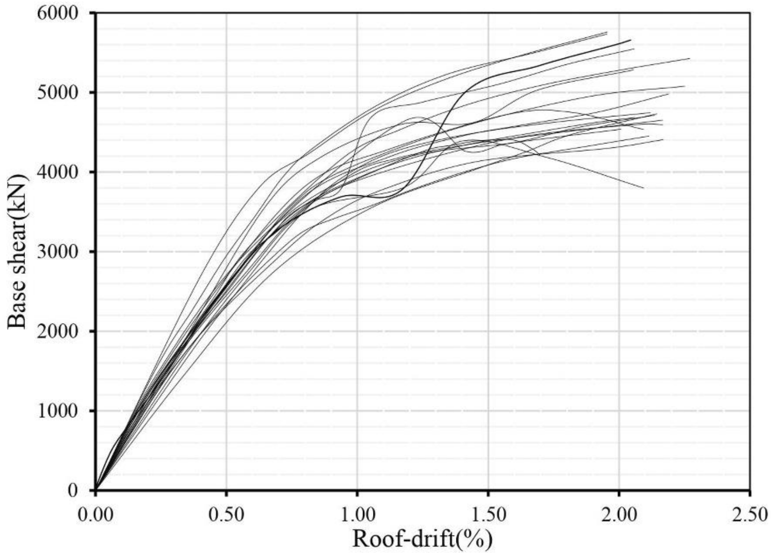


Fig. 8 Capacity curves from IDA

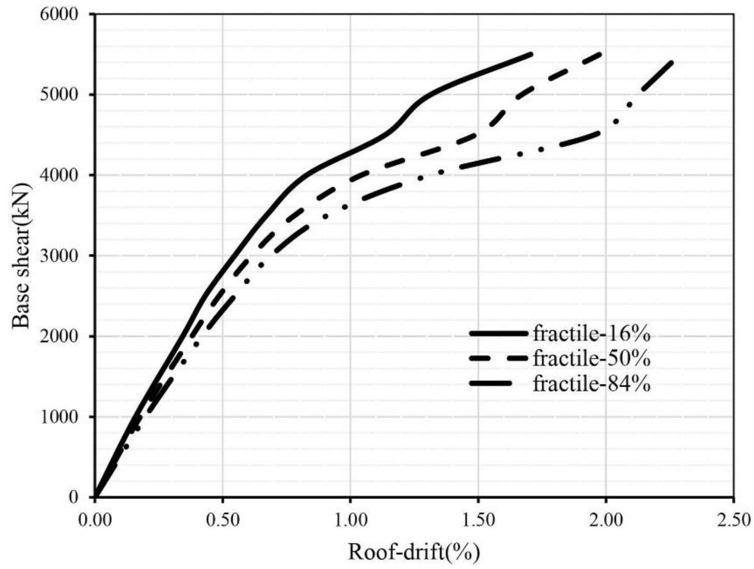


Fig. 9 Capacity curves from IDA for 16%, 50%, and 84% fractiles

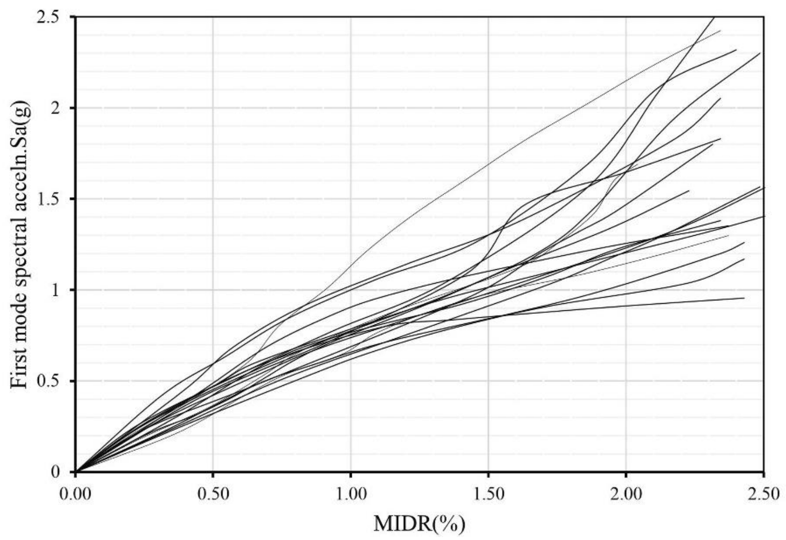


Fig. 10 IDA curves

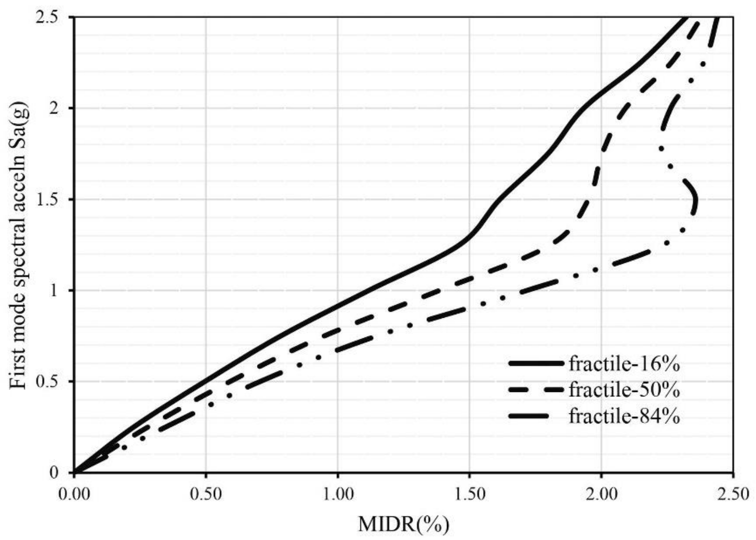


Fig. 11 IDA curves for 16%, 50%, and 84% fractiles

2 Fragility Curves

Fragility curves provide the probability of collapse or reaching other limit states corresponding to a particular seismic intensity measure. IDA can be effectively used for the extraction of data required for the estimation of analytical fragility functions. Various methods are reported in the literature for the seismic fragility assessment using the raw as well as multi-IDA data. Mandal et al. [25] provide a detailed account of various methods for the estimation of seismic fragility using the multi-IDA data. The present study uses a lognormal cumulative distribution function (CDF) to define the fragility function considering that the intensity measure (IM) values of ground motion records for the considered limit state of interest are lognormally distributed. The fragility function is defined as

$$P(C|IM = x) = \Phi\left[\frac{\ln(x/\theta)}{\beta}\right] \tag{4}$$

Where $P(C|IM=x)$ denotes the probability that a ground motion with $IM = x$ will cause the structure to reach the required limit state, $\Phi(\,)$ is the standard normal cumulative distribution function (CDF), θ the median of the fragility function and β is the standard deviation of $ln(IM)$. The median and standard deviation are calculated from the IDA data as:

$$\ln\theta = \frac{1}{n}\sum_{i=1}^{n}\ln IM_i \tag{5}$$

$$\beta = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}\left[\ln\left(IM_i/\theta\right)\right]^2} \tag{6}$$

Where n is the number of ground motion records considered for the fragility and IM_i is the IM value associated with the considered limit state in respect of i^{th} ground motion.

For the development of fragility curves, the threshold values of maximum inter-story drift ratio (MIDR) relevant to the FEMA 356 specified performance levels namely LS, IO and CP are fixed at 0.8%, 1.0%, and 1.8%. These values are selected based on IDA for the considered earthquake hazard level. The fragility curves are shown in Figure 12.

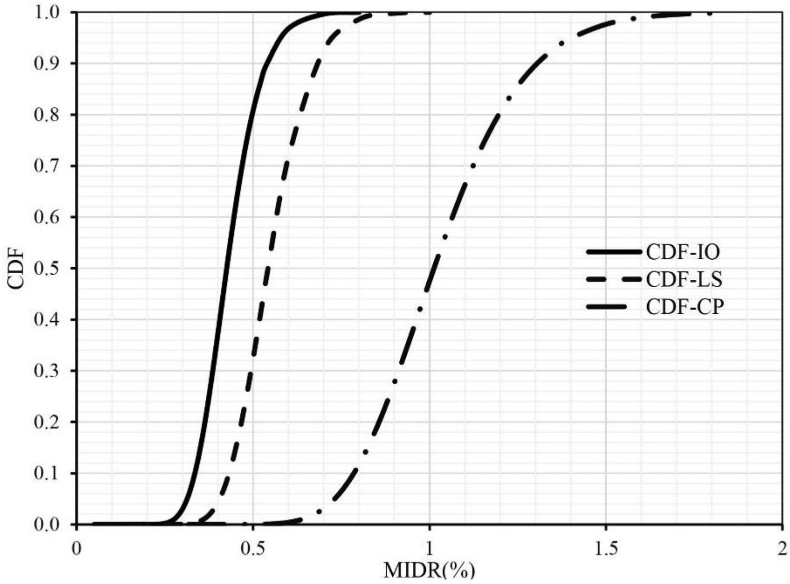


Fig. 12 Fragility curves

CONCLUSIONS

The seismic performance assessment of coupled systems built in installations such as refineries and petrochemical complexes represent a challenging task for the design engineers involved in the detailed engineering of both the supporting structure (SS) and the non-building structure (NBS). The seismic evaluation in such cases shall properly take into account the dynamic characteristics of NBS to decide upon the requirement of a coupled analysis. The seismic assessment of a gas-phase reactor structure in reinforced concrete, a typical of coupled system, adapted from a refinery complex is presented. The coupled analysis is considered in the present study to include the dynamic interaction effects in the response calculations. A nonlinear static analysis is first performed to arrive at an estimate of the capacity and seismic demands before undertaking an incremental dynamical analysis to obtain the accurate results. The performance of the structure was monitored for earthquake hazard levels having 20%, 10%, and 2% probabilities of exceedance in 50 years. The hazard levels correspond to performance objectives identified in FEMA 356 as IO, LS and CP respectively. The maximum inter-story drift ratio (MIDR) for the supporting structure at different performance levels were found to be within the corresponding maximum limits specified in FEMA 356. The fragility curves are also developed for threshold limits of MIDR at 0.8%, 1.0%, and 1.8%.

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