Assessment and Rehabilitation of Jacket Platforms

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1. Introduction

The most important recent earthquake showed the importance of seismic assessment of both onshore and offshore structures (Takewaki et al., 2011). Strong earthquake can cause damages to engineering structures. Many strong earthquakes normally take place in offshore such as 2011 Japan earthquake (Moustafa, 2011 and Takewaki et al., 2011) can cause severe damage to offshore structures. The steel jacket structure is a kind of fixed offshore platform that is suitable for construction in water depth from a few meters to more than 100 m. Compared to regular structures, a jacket offshore platform is a complicated system and is composed of many parts include structural and nonstructural elements. Structural modeling includes two major division; structure and pile foundation. Fig. 1 shows parts of jacket offshore platform system and some of related researches.

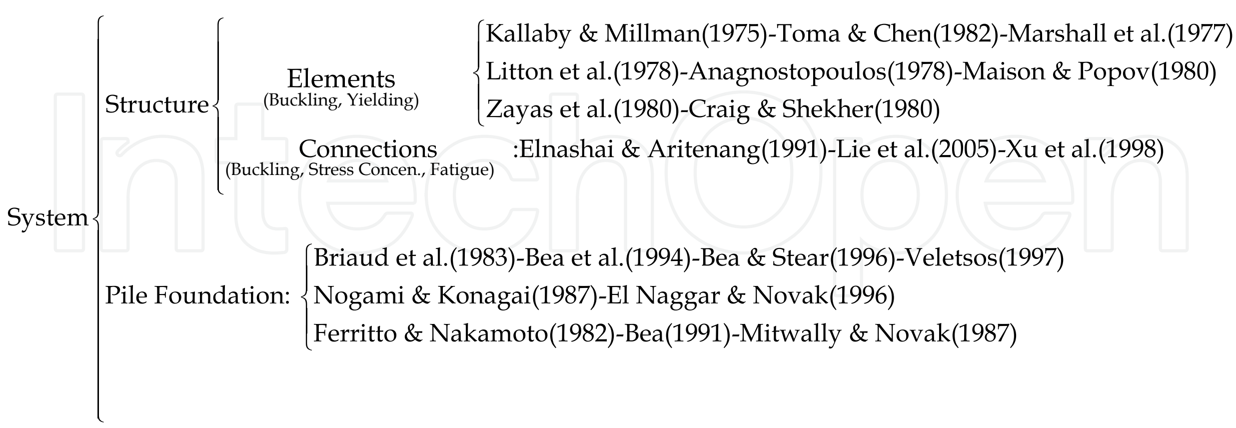


Fig. 1. Parts of jacket offshore platform system.

The major structural components of such an offshore platform are jacket, piles, and deck. A jacket structure which serves as bracing for the piles against lateral loads is fixed by piles driven through the inside of the legs of the jacket structure and into soil many tens of meters deep. The deck structure is fixed upon the jacket structure. Oceans in which offshore platforms are built present a set of complicated and harsh environmental conditions. Dynamic loads including wind, wave, current, and earthquakes dominate the design of offshore structures. Venkataramana (1998) presented a time domain analysis of the dynamic response of a simplified offshore structure to simultaneous loadings by random sea waves and random earthquake ground motions. Kawano and Venkataramana (1999) investigate the dynamic response and reliability analysis of offshore structure under the action of sea waves, currents and earthquakes. The dynamic loads affect not only the routine operation of an offshore platform such as drilling and production activities, but also the safety and serviceability of the structure. Approximately 100 template-type offshore platforms have been installed in seismically active regions of the world's oceans. New regions with the potential for significant seismic activity are now beginning to be developed. Older platforms in seismic regions may have three areas of deficiency:

1. Inadequate ground motions for original design.
2. Structural framing which is not arranged or detailed for ductile behavior.
3. Reduced capacity resulting from damage, corrosion or fatigue.

Many of these platforms are now beyond their original design life (20-25 years), as well. From the economic point of view the continued use of an existing installation will in many cases be preferable compared to a new installation. The assessment of existing platforms under environmental (wave, wind, current etc.) loads and probable future loads (earthquake) (Moustafa, 2011) is a relatively new process and has not yet been standardized as design is. This lack of standardization creates some difficulty in establishing performance requirements which must be developed depending upon the risks (i.e., hazards, exposures and consequences) associated with the future operations of the platform. The present criteria of the offshore structure standards for seismic assessment can be improved using building pre-standards.

Assessment of jacket platforms has rarely been studied. Krieger et al. (1994) describe the process of assessment of existing platforms. Petrauskas et al. (1994) illustrate assessment of structural members and foundation of jacket platforms against metocean loads. Craig et al. explain assessment criteria for various loading conditions. Ersdal (2005) evaluates the possible life extension of offshore installations and procedures of standards in this matter, with a focus on ultimate limit state analysis and fatigue analysis. Gebara et al. (2000) assess the performance of the jacket platform under subsidence and perform ultimate strength and reliability analyses for four levels of sea floor subsidence. The assessment process of building prestandards was studied also. Bardakis and Dritsos (2007) compared the criteria of FEMA-356 and GRECO (Based on the EC-8). Hueste and Bai (2007) described the assessment and rehabilitation of an existing concrete building based on the FEMA-356 procedure and criteria. Golafshani et al. (2009) suggested this idea that API procedures for seismic assessment of jacket platforms can be evaluated and improved with respect to building documents for the first time at 2006.

Fig. 2 shows some standards for offshore structures that include detailed procedures for the assessment of existing structures. Petroleum and natural gas industries Offshore Structures Part 1: General Requirements,' ISO (2002) is one of the most general accepted standards. A detailed assessment procedure for existing structures is found in ISO 19902. The Norwegian regulations (PSA 2004) refer to ISO 19900 (ISO 2002) for the assessment of existing structures. Other standards, like API RP2A-WSD (API 2000) and ISO/DIS 13822 (ISO 2000), also include detailed procedures for the assessment of existing structures.

API RP2A is one of the most useful standards for the design and assessment of offshore structures. Section 17 of this standard has recommendations for the assessment of offshore structures. The assessment criteria of this standard are based on the objective of collapse prevention of the structure under extreme earthquake conditions. The results of an assessment with API give information about the total structure's condition. This standard describes the rehabilitation objective globally and does not present a routine methodology for rehabilitation.

In the last decade, several building documents such as FEMA- 356 and ATC-40 were developed for the assessment and rehabilitation of these structures. In the FEMA-356 document seismic deficiencies identified using an evaluation methodology considering building performance at a certain seismic hazard. The FEMA-356 document developed an extensive assessment and rehabilitation procedure. This document not only has numerical criteria for assessment but also presents design procedures for rehabilitation.

- Codes
  - Assessment Codes
    - FEMA
      - 356
      - 440
    - ATC
      - 40
      - 55
    - ISO 19900 (Petroleum and natural gas industries)(2002)
    - NORSOK N-001(NORSOK 2004)
    - ISO 19902
    - ISO/DIS 13822 (ISO 2000)
    - PPCON 1998
    - API (Chapter 17)
  - Retrofit
    - FEMA 356
    - FEMA 547
    - SAFERELNET 2004

Fig. 2. Some standards for assessment and rehabilitation of jacket offshore platforms

The API approach is based on simple collapse mechanism investigation without describing the methodology in detail. However FEMA-356 consists of detailed processes for both seismic assessment and rehabilitation of building structures. However there are advices in API documents (API RP 2A) in order to seismic assessment of jacket platforms, but because of brief existing comments in this field, it is necessary to use more appropriate pre-standards for seismic assessment of these structures. For example Golafshani et al., (2009) compares FEMA-356 and API approach for assessment of jacket offshore platform structures. Komachi et al., (2009a) presented the performance based assessment of jacket platforms for seismic vulnerability.

Current methods for seismic upgrading of existing structures can be classified into two major groups: traditional and modern. Traditional methods aim to increase the strength and/or ductility of the structure by repairing/upgrading members. Nowadays there are some new technologies (for example seismic isolation and energy dissipation) for seismic protection of the structures. The passive control approach is of current concern to many researchers and there are several attempts exploring its application to offshore structures. Recently, although, there have been several studies for the effectiveness of active and passive control mechanisms in controlling the response of offshore platforms under wave loading.

Incorporation of energy dissipation systems in a traditional earthquake-resistant structure has been recognized as an effective strategy for seismic protection of structures (Soong and Dargush, 1997). New vibration control technologies have been applied to offshore structures in the following cases. Vandiver and Mitome (1979) used storage tanks as Tuned Liquid Damper (TLD) on a fixed platform to mitigate the vibration of the structure subjected to random wave forces. Kawano and Venkataraman (1992) and Kawano (1993) studied the application of an active tuned mass damper to reduce the response of platforms due to wave loading. Abdel-Rohman (1996) studied the dynamic response of a steel jacket platform with certain active and passive control due to wave-induced loading. Lee (1997) used stochastic analysis and demonstrated the efficiency of mechanical dampers for an offshore platform. Suneja and Datta (1998) demonstrated the efficiency of an active control system for articulated leg platforms under wave loading. Vincenzo and Roger (1999) developed an Active Mass Damper for suppression of vortex- induced vibrations of offshore structures. Chen et al. (1999) studied the response of a jacket platform installed with TLD due to earthquake loading. Ou et al. (1999) studied the response reduction of jacket platforms with a viscoelastic damper with respect to ice loads. Terro et al. (1999) developed a multi-loop feedback-control design as applied to an offshore steel jacket platform. Suhardjo and Kareem (2001) used both passive and active control systems for the control of offshore platforms. Ding (2001) studied the response reduction of jacket platforms with a viscous damper due to ice loads. Qu et al. (2001) presented a rational analytical method for determining the dynamic response of large truss towers equipped with friction dampers under wind-excitation and investigated the efficiency of friction dampers. Wang (2002) used Magnetorheological dampers for vibration control of offshore platforms for wave-excited response. Mahadik and Jangid (2003) studied the response of offshore jacket platforms with an active tuned mass damper under wave loading. Patil and Janjid (2005) studied the behavior of a platform with viscoelastic, viscous and friction damper for wave loads. Lee et al. (2006) studied the effectiveness of a Tuned Liquid Column Damper (TLCD), which dissipates energy by water flow between two water columns, for offshore structures and also, Ou et al. (2006) studied the application of damping isolation systems for response mitigation of offshore platform structures. Jin et al. (2007) studied the effect of Tuned Liquid Dampers (TLD) and found that the larger the ratio of water-mass to platform-mass, the higher the reduction of responses. Komachi et al., (2009) presented Friction Damper Devices (FDD) as a control system to rehabilitation existing jacket offshore platforms. Golafshani and Gholizad (2009) studied the performance of friction dampers for mitigating of wave-induced vibrations and used mathematical formulation to evaluate the response of the model. Yoe et al. (2009) used Tuned Mass Damper (TMD) for mitigation of dynamic ice loads.

The service life of an offshore structure can be doubled if the dynamic stress amplitude reduces by 15%. Few studies have reported on the effectiveness of the passive control systems using dampers in controlling the response of offshore platforms under a parametric variation studying the influence of important system parameters and comparative performance of dampers. In order to reduce possible damage to jacket offshore platforms in harsh marine environments, the necessity of carrying out further studies on developing efficient and practical vibration control strategies for the suppression of dynamic responses of existing offshore structures should be emphasized. In this chapter rough and global comments of API are compared with detailed method of FEMA. As an example seismic assessment of the existing 4 legged Service platform placed in the Persian Gulf is presented. A very useful method for rehabilitation of existing jacket platforms is damper. In this study, a Friction Damper Device (FDD) proposed by Mualla (2002) is used to mitigate the vibration of a typical fixed jacket offshore platform in Persian Gulf. The contents of this study mainly include the investigation of the influence of the damping system parameters on vibration control of offshore platforms under the actions of earthquake excitations. This chapter shows that FDD improves the structural behavior and performance of jacket platforms.

2. Assessment

2.1 API recommendations

The API is currently developing recommendations for the assessment of existing platforms including requirements for platforms subjected to hurricanes, storms, earthquakes and ice loading. These recommendations will likely focus on a demonstration of adequate ductility for platforms located in earthquake dominated regions. The focus towards ductility, or demonstrated survivability, under extreme earthquake conditions is based on the objective of prevention of loss of life and pollution. The performance criterion for assessment is essentially identical to that of the Design Level Earthquake (DLE) requirement for new designs.

The structures need to meet one of two sets of global structural performance criteria, depending on the platform's exposure category. In addition, local structural performance requirements for topside equipment and appurtenances must be met, independent of the platform's exposure category classification. In the case of high exposure platforms, they must be shown by rational analysis (Pushover or Nonlinear Time history) to remain globally stable under median ground motions representative of an earthquake with an associated return period of 1000 years. For lower exposure platforms located in areas with high seismic activity, a return period of 500 years must be selected.

2.2 FEMA recommendations

Performance based design (PBD) has been fully described in the guidelines published by FEMA and ATC. These documents do not have the force of codes but provide details of best practice for the evaluation and strengthening of existing buildings. These are continuing to be expanded as PBD becomes more widespread. In these standards a criterion such as drift is applied indirectly when the elements are assessed.

Four levels of building performance consist of Operational (O), Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) in increasing levels of damage is considered in FEMA. The Immediate Occupancy (IO) performance level requires that the building remain essentially functional during and immediately after the earthquake. The last performance level, Collapse Prevention (CP) will result in a building on the point of collapse and probably economically irreparable. The Life Safety level (LS) is the level usually implicit in codes and may also result in a building which is not economic to repair. The rehabilitation objectives are formed of combinations of earthquake hazard and building performance (consisting of structural and nonstructural performance levels).

FEMA sets a desirable goal for rehabilitation, a Basic Safety Objective (BSO) which comprises two targets:

1. Life Safety building performance at Basic Safety Earthquake 1 (BSE-1), the 475 year earthquake.
2. Collapse Prevention performance at BSE-2, the 2500 year earthquake.

Depending on the function of the structure, other objectives may be set. For example, an enhanced objective may set higher building performance levels at BSE-1 and BSE-2 for critical facilities such as hospitals. Component actions are classified as deformation or force-controlled. Table 1 shows some of the action types. A component acceptance criterion for force-controlled actions is based on the force and is independent of the performance level and components shall have lower-bound strengths not less than the maximum design forces. For deformation-controlled actions the criterion is based on the target performance level and in these components shall have expected deformation capacities not less than maximum deformation demands calculated at the target displacement.

Action		Force-Control	Deformation-Control
Braces in Tension and Compression			✓
Columns — Compression		✓	
Columns — flexure	$P < 0.5P_{cr}$		✓
	$P > 0.5P_{cr}$	✓	

Table 1. Type of action for some elements

FEMA provides modeling parameters and numerical acceptance criteria for beams, columns, and braces as a function of parameters such as diameter to thickness ratio. The leg and brace in the jacket act like a column and brace in the steel braced frame, respectively. Therefore, for assessment of these structures using FEMA, criteria of chapter five of this document will be used.

2.3 Comparison of API RP 2A and FEMA-356

In the Table 2 procedures of FEMA and API for assessment are compared to each other. In contrast to API that assesses structures globally, FEMA evaluates each member of the structure for assessment. This matter has some advantages in several manners:

	API RP 2A	FEMA-356
Structure	Jacket Platform	Building
Criteria for	Total structure	Members and structure
Return Period of extreme earthquake	1000 years	2500 years
Modeling: Soil-pile-Structure Interaction Modeling parameters and Criteria for members	 By details Don't have	 Global Have
Simplified procedure for assessment	---	Coefficient Method
Rehabilitation method	Don't have	Have

Table 2. Comparison of API RP 2A and FEMA-356

1. Comparing with buildings, the redundancy of the jacket platform structure is low, and failure of some members can affect not only the routine operation of structure such as production activities, but also the safety and serviceability of the structure. Then the results of FEMA are more reliable and economical than API.
2. Specific criteria can be taken into account for the evaluation of each member with respect to its condition. This matter is more important for a jacket platform in that strength degradation of members due to fatigue and corrosion and etc. is feasible.
3. Rehabilitation of the structure can be performed better knowing the behavior of members.

The return period adopted for collapse requirement ground motions in API RP 2A is lower than that of FEMA-356. Earthquake return periods of FEMA and API are compared to each other in the Table 4. From a point of comparison with FEMA-356, there are four principal reasons why the earthquake return period of API is low:

1. Importance of offshore structures is higher than buildings.
2. Seismic loads imposed on a structure are highly dependent on the stiffness and energy dissipation characteristics of the structural system, including the piling and supporting soils, and so higher uncertainties in soil properties result directly in higher uncertainties in loads.
3. Uncertainties in the estimation of ground motions for offshore structures are higher than those for buildings.
4. Because of lower redundancy, the sensitivity to increase in return period is greater for offshore structures than buildings. This item is an important difference between jacket platforms and buildings.

The FEMA document represents simplified procedures such as a coefficient method that can be used for assessment and rehabilitation of buildings for different loading conditions and this document consists of useful procedures for the rehabilitation of buildings.

2.4 Response determination using nonlinear pushover analysis

Nonlinear time-history analysis can be used for assessment and rehabilitation of all types of structures. This procedure is complicated and time consuming. Nowadays nonlinear static procedures are widely used for the assessment and rehabilitation of structures. These procedures can be used to estimate the response of structures under seismic loading. The target displacement for each level of load is calculated. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The stresses and deformations in each component are then evaluated at this displacement level. FEMA-356 utilizes the Coefficient Method in which several empirically derived factors are used to modify the response of a single-degree-of-freedom (SDOF) model of the structure assuming that it remains elastic. The Capacity-Spectrum Method of ATC-40 uses empirically derived relationships for the effective period and damping as a function of ductility to estimate the response of an equivalent linear SDOF oscillator. Recently these methods evaluated and improved in the FEMA-440 document [16].

2.4.1 Capacity-spectrum method (ATC-40)

In the Capacity-Spectrum Method, the base shear versus roof displacement relationship (capacity) and seismic ground motion (demand) are plotted in Acceleration-Displacement Response Spectrum (ADRS) format. The performance point (maximum inelastic displacement) can be obtained from the intersection point of demand and capacity. This procedure is presented in Fig. 3. In this figure S_a and S_d are spectral acceleration and spectral displacement respectively.

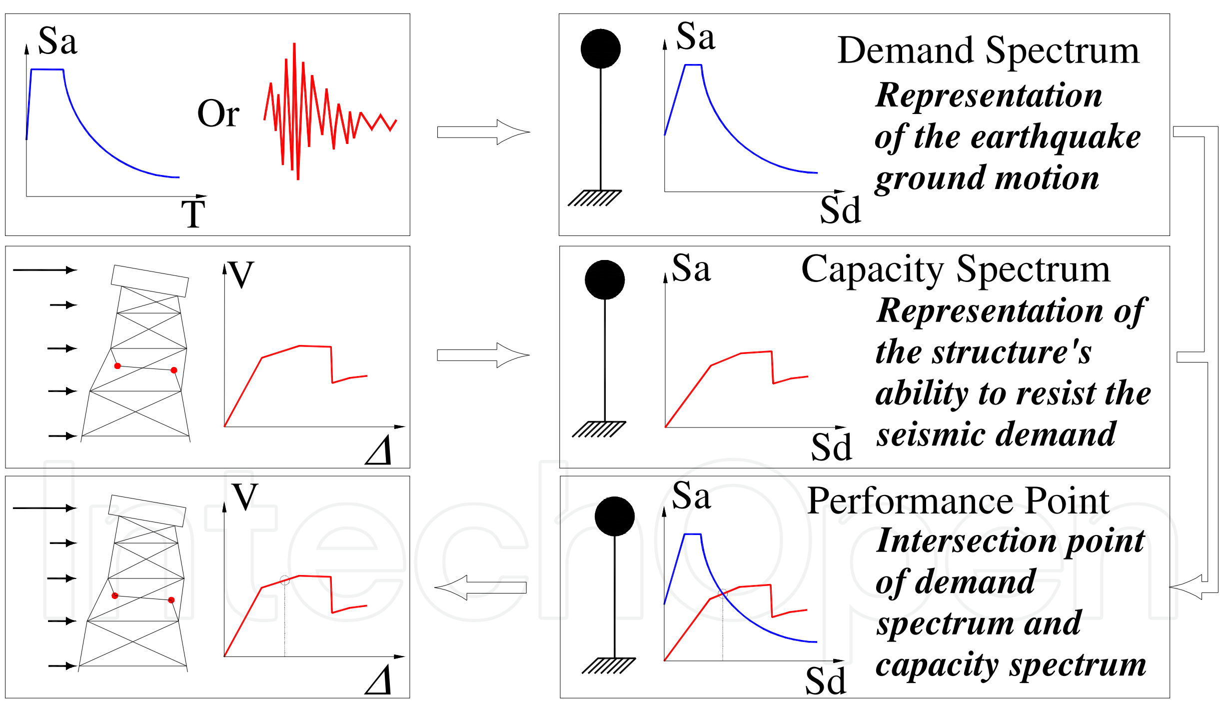


Fig. 3. Capacity-spectrum method.

2.4.2 The coefficient method (FEMA-356)

In the Coefficient Method, the maximum inelastic displacement (Target Displacement) is obtained from multiplying the linear elastic response by a series of coefficients C_0 through C_3. These coefficients are derived from statistical studies of the nonlinear time-history analyses of SDOF oscillators.

2.4.3 FEMA-440 recommendations

The FEMA-440 document evaluates and improves the abovementioned simplified inelastic analysis procedures. Proposed modifications to the Coefficient Method of FEMA-356 relate primarily to the coefficients themselves. For coefficients C_1 and C_2 new relationships are proposed. It is also proposed that instead of coefficient C_3 a limitation on minimum strength be used. The improved procedure for the Capacity-Spectrum Method consists of new estimates of equivalent period and damping. This Linearization Method is calibrated for certain hysteretic loops with different calibration equations for the nondegrading and degrading cases.

2.5 Case study

2.5.1 Description of the jacket platform

The existing offshore complex consists of a drilling platform, a production platform, a service platform and a flare tripod in the field. The field was originally developed and put in production in 1968. There has been some damage imposed during Iran/Iraq war and some other extended damage due to adverse climate conditions afterwards. The service platform consists of a four leg battered jacket and topside located in 67.40 m water depth which is connected to production platform by means of one existing bridge. The service life of the platform is 25 years.

2.5.2 Load cases

For time history analysis of the platform, a `best fit' set of scaled, natural time histories is used provided the velocity spectrum values have been properly modulated to equal or exceed the standard spectrum velocity values at specified periods (0.2 T to 1.5 T) as mentioned in International Building Code (IBC).

2.5.3 Numerical model

Analytical models were created using the open source finite element platform, OpenSees. This program is useful for modeling of jacket platform structures because of its capability of modelling of the post-buckling behavior of tubular members, soil-pile-structure interaction and etc. A two-dimensional model of a single frame is developed for the structure. A force-based nonlinear beam-column element (utilizing a layered fiber section) is used to model all components of the frame. Steel material is modeled using a bilinear stress-strain curve with 0.3% post-yield hardening. Initial imperfections in the struts are accounted for, with a value of 0:001L where L is the length of the member. This idea is useful for modeling the post-buckling behavior of the strut members, respectively.

The mathematical model of the pile-soil-structure system consists of the following sets of elements (Fig. 4):

1. Pile elements, modeled by a number of nonlinear beam-column elements.
2. Far-field soil model representing the free-field motion of the soil column, vertically and horizontally that is unaffected by the pile motions. The soil is modeled using elastic quad elements. The nodes that are at the same depth are constrained.
3. Near-field elements that connect the piles to the soil, vertically and horizontally. The strength and stiffness of these elements depends on the state of the far-field soil and the relative motion of the pile and far-field soil. The interface between the pile and

surrounding soil is modeled using p-y, t-z, q-z nonlinear spring elements. Hysteretic and radiation damping are considered using these elements. The group effects are not considered. The input motion is applied to the fixed nodes at the bottom of the soil column. The seismic record at bedrock is found from the input motion at the surface. Hydrodynamic effects are considered in terms of hydrodynamic damping from drag forces and added masses.

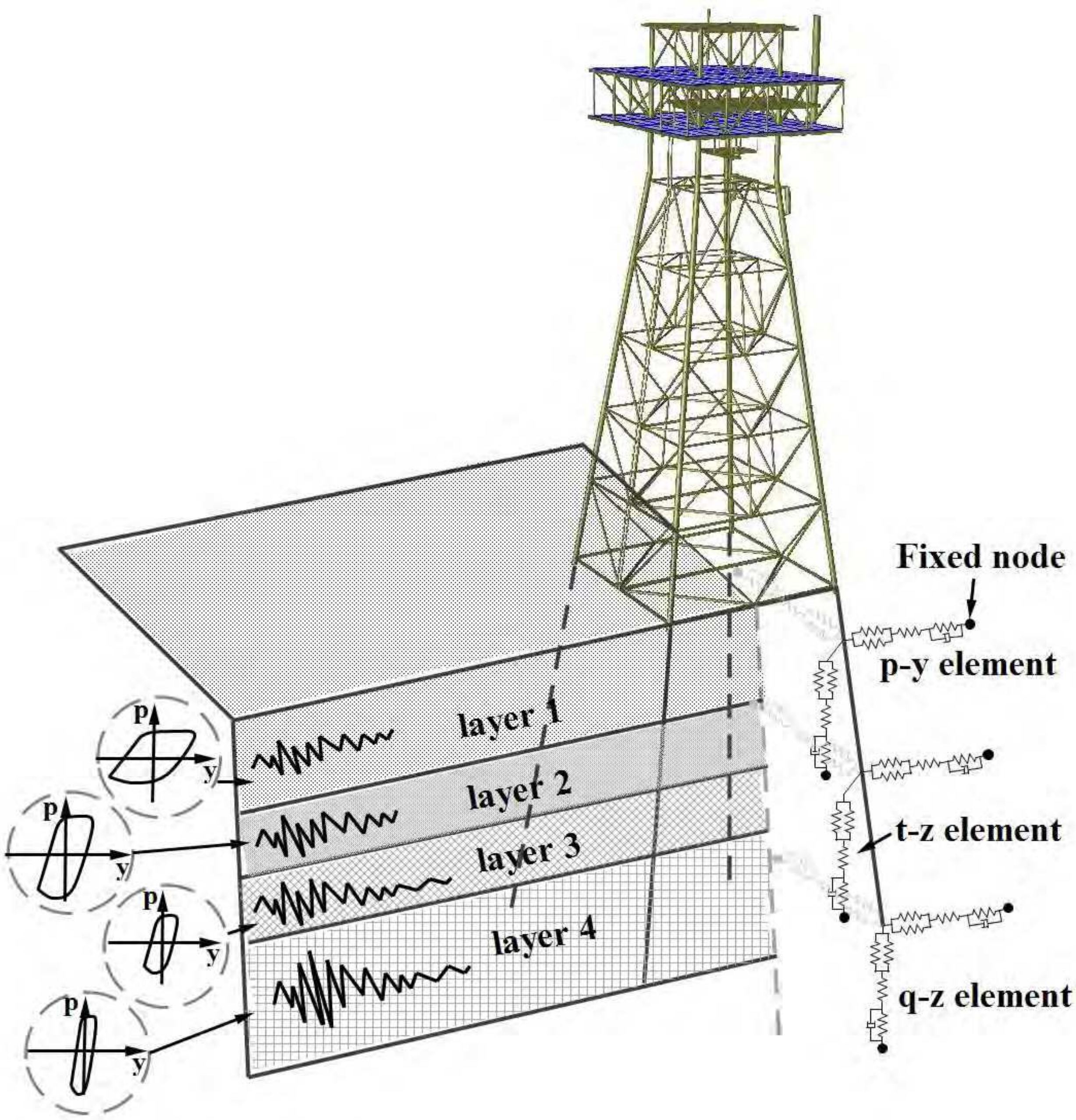


Fig. 4. Modeling of soil-pile-structure interaction.

2.5.4 Numerical study

A uniform distribution of load is applied for the pushover analysis. The backbone curve of the deformation-control action of members is needed for an assessment of the structure based on the FEMA procedure. Fig. 5 shows the backbone curve of the axial action for strut-2 of the platform and FEMA criteria. Table 6 shows dimensions and assessment parameters of jacket struts. In this Table Δ_C and Δ_T are the axial deformations at the expected buckling load and at the expected tensile yielding load, respectively. Fig. 6 shows the pushover curve of the structure. This Figure shows an instantaneous loss of strength at a deck displacement equal to 0.29 m. It can be seen that after the point with a deck displacement of 1.15m the load-deformation curve has a negative slope. This figure also shows that for a deck displacement equal to 2.0 m, the 1st and 2nd platform levels remain elastic.

The typical platform in Persian Gulf was assessed using responses obtained from a series of nonlinear time history analyses using three best fit records for each hazard level.

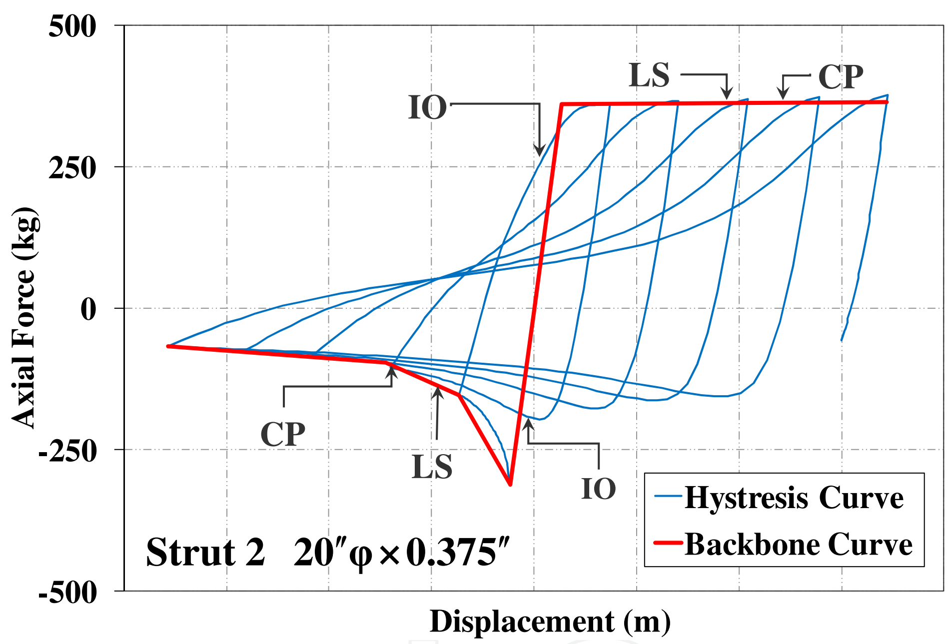


Fig. 5. Hysteretic behavior of Strut-2 and its Backbone curve.

2.5.5 Structural assessment

This section provides processes of assessment of the typical jacket platform based on the API and FEMA-356. The demands are obtained from mean values of the time history analysis.

2.5.5.1 API standard

The collapse of the structure is defined by its lack of ability to withstand to the load. The collapse load is defined as the maximum load the structure can withstand, before the load-displacement curve starts a negative trend. The deck displacement at the collapse point of the jacket is 1.15 m.

2.5.5.2 FEMA-356 document

Leg behavior. Compression action in leg elements is a force control action, but the action type for flexure in these elements depends on the axial force of legs.

Brace behavior. Braces are members with deformation controlled action and mainly determine the performance level of jackets. Fig. 6 shows the performance of the jacket for each hazard level. This figure shows that the jacket platform is highly sensitive to input motion.

2.5.6 Directivity effects

One of the primary factors affecting motion in the near-fault region is the directivity in which rupture progresses from the hypocenter along the zone of rupture. "Directivity" refers to the direction of rupture propagation as opposed to the direction of ground displacement (Abrahamson, 1998). A site may be classified after an earthquake as demonstrating forward, reverse, or neutral directivity effects. If the rupture propagates toward the site and the angle between the fault and the direction from the hypocenter to the site is reasonably small, the site is likely to demonstrate forward directivity. If rupture propagates away from the site, it will likely demonstrate reverse directivity (Abrahamson, 1998). If the site is more or less perpendicular to the fault from the hypocenter it will likely demonstrate neutral directivity. The phrase "directivity effects" usually refers to "forward directivity effects", as these case results in ground motions that are more critical to engineered structures.

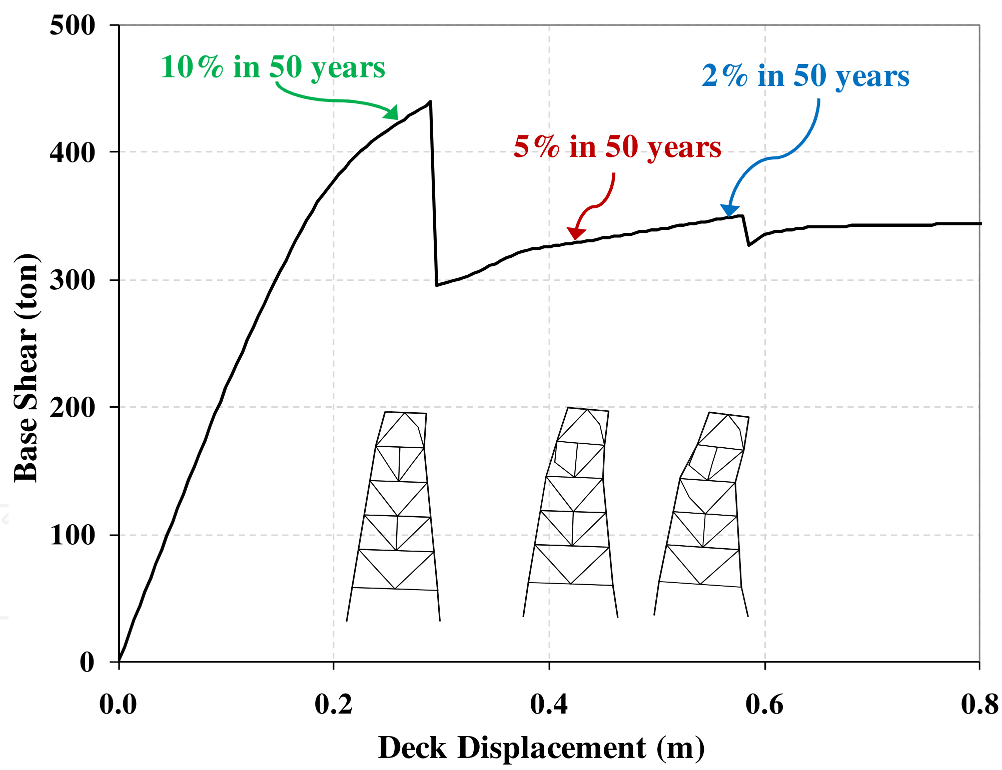


Fig. 6. Performance of the jacket for various levels of hazard.

A large velocity directivity pulse occurs when the conditions of forward directivity are met. These conditions include:

1. The earthquake is sufficiently large (moment magnitude greater than 6);
2. The site is located sufficiently close to the fault rupture (within 10 km); and
3. The rupture propagates toward the site.

This large velocity directivity pulse will be evident in the fault-normal direction. It is typically located toward the front of the time history and consists of, on average, one cycle of motion. Peak velocities usually are between 30 cm/sec to 200 cm/s with a mean value about 100 cm/sec. The period of the pulse can range from 0.5 sec to 5 sec with a mean value about 2.5 sec. Tabeshpour presented a conceptual discussion on the effect of near field earthquakes on various structures (2009).

2.5.6.1 Pulse-type excitation

For consideration of directivity effects, the analytical velocity pulse model proposed by He (2003) is expressed as:

$$\dot{u}_p = Ct^n e^{-at} \sin \omega_p t \tag{1}$$

Where C is the amplitude scaling factor, a is the decay factor, n is the shape parameter of the envelope, and ω_p is the pulse frequency in rad/s. Differentiating above equation, the acceleration $\ddot{u}_p$ of the pulse can be obtained as:

$$\ddot{u}_p = Ct^n e^{-at}\left[(\frac{n}{t} - a)\sin \omega_p t + \omega_p \cos \omega_p t \right] \tag{2}$$

The acceleration $\ddot{u}_p$ in above Equation is considered as ground acceleration for numerical simulations in this chapter. To illustrate the performance of structure during the near-fault excitations, the parameter $\beta = T_p / T$ is used, where T and T_p are the fundamental period of jacket platform and pulse period, respectively. Fig. 7 shows time-history plots of acceleration for a velocity pulse with parameters $a = 2.51$, $C = 7.17$, $n = 1$ for three T_p.

2.5.6.2 Results and discussions

Base shear of the structure vs. deck displacement (hysteresis loops) is presented in Fig. 8. A global view of the nonlinear behavior of the structure is seen clearly at the first pulse of excitation. In is seen that all energy dissipation by hysteretic behavior of the elements is occurred just in the one loop. Maximum displacements are shown by points A, B and C in horizontal axis.

3. Rehabilitation

Fig. 9 provides rehabilitation process of jacket offshore platforms. Rehabilitation of existing jacket consists of two phases of assessment and rehabilitation. Many researches have been carried out in this matter that is shown in this figure. With respect to type of loads, many types of control systems can be used for rehabilitation of jacket offshore platforms. Fig. 10 shows types of control systems usable for jacket offshore platforms.

3.1 Tuned mass damper (TMD), wind and wave protection

TMD is suitable passive control device for narrow band loads such as wind and wave loads. The efficiency of this device should be investigated for environmental loads. Fig. 11 shows jacket platform equipped with TMD and equal single degree of freedom. Fig. 12 shows the effect of TMD on the displacement response of jacket platform under the harmonic wave load with wave period equal to the fundamental period of the structure (T) and wave height H=0.212 m. A clearly decrease is observed in structural response.

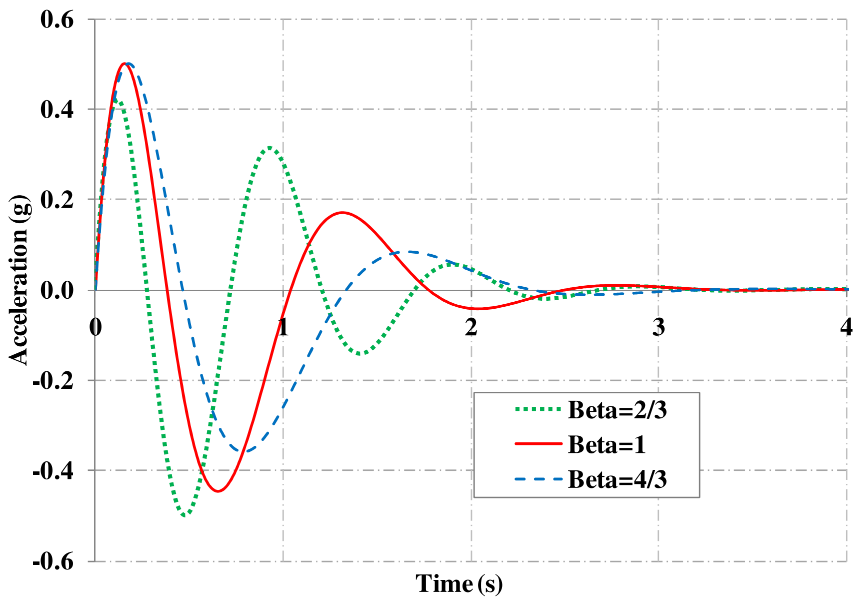


Fig. 7. Time-history plots of acceleration for $\beta = 2/3, 1, 4/3$.

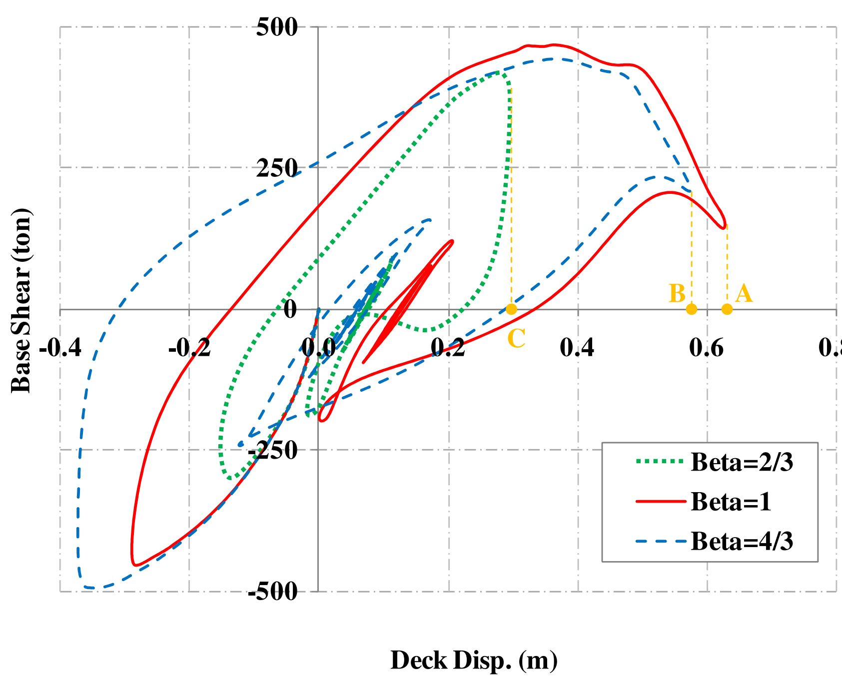


Fig. 8. Hysteresis loops of the structure for β = 2/3,1,4/3.

- Assessment
  - High Cycle Fatigue (Wave and Wind)
    - Bea and Mortazavi(1996)-Bea et al. (1994,1995,1996)
    - Kawamoto et al.(1982)-Craig et al.(1980)
    - Krieger et al.(1994)-Petrauskas et al.(1994)-Ersdal(2005)
    - Gebara et al.(2000)-Puskar and Verret(2007)
  - Low Cycle Fatigue (Earthquake)
    - Bea et al. (1994,1995,1996)-Bea and Mortazavi(1996)
    - Bea & Stear(1996)-Golafshani et al. (2006)

Fig. 9. Rehabilitation studies process for jacket offshore platforms.

- Control Systems
  - Passive: Abdel-Rohman(1996); Lee (1997); Suhardjo & Kareem(2001)
    - Friction Damper Devices:
      - Komachi et al.(2009, 2011)-Patil & Janjid (2005)
      - Qu et al.(2001)-Golafshani & Gholizad(2009)
    - Viscous Damper: Ding (2001)-Patil & Janjid (2005)
    - Viscoelastic Damper: Ou et al. (1999)-Patil & Janjid (2005)
    - Damping Isolation Systems: Ou et al. (2006)
    - Tuned Liquid Damper :
      - Vandiver & Mitome (1979)
      - Chen et al. (1999)-Jin et al. (2007)
    - Tuned Mass Damper :
      - Yoe et al. (2009)- Golafshani & Gholizad(2009)
      - Tabeshpour et al. (2010, 2011)
    - Tuned Liquid Column Damper: Lee et al. (2006)
  - Active: Abdel-Rohman (1996); Suneja & Datta (1998); Suhardjo & Kareem (2001)
    - Active Tuned Mass Damper:
      - Kawano & Venkataramana (1992)
      - Kawano (1993)-Mahadik & Jangid (2003)
    - Active Mass Damper: Vincenzo & Roger (1999)

Fig. 10. Types of control systems used for jacket offshore platforms.

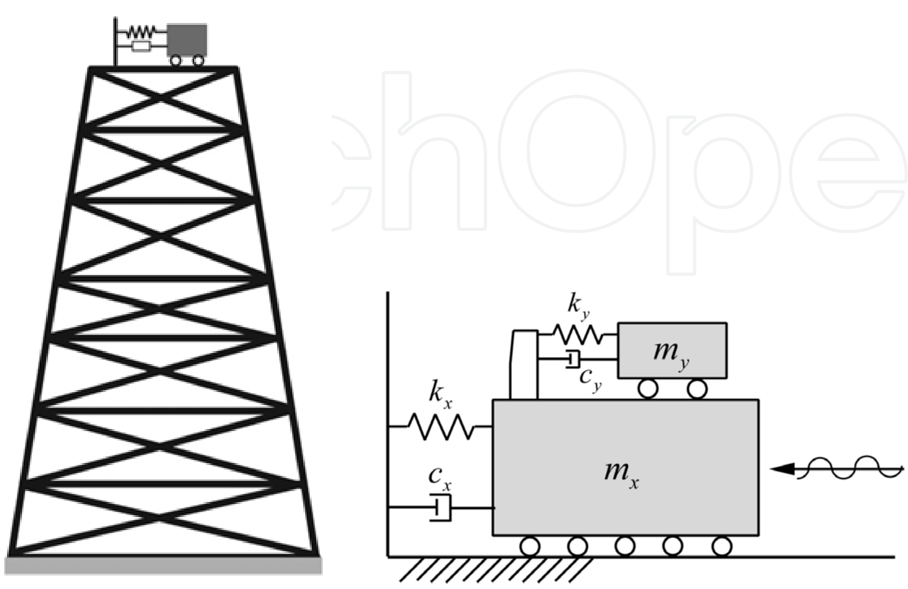


Fig. 11. Steel jacket platform utilized with a TMD and its equivalent SDOF system.

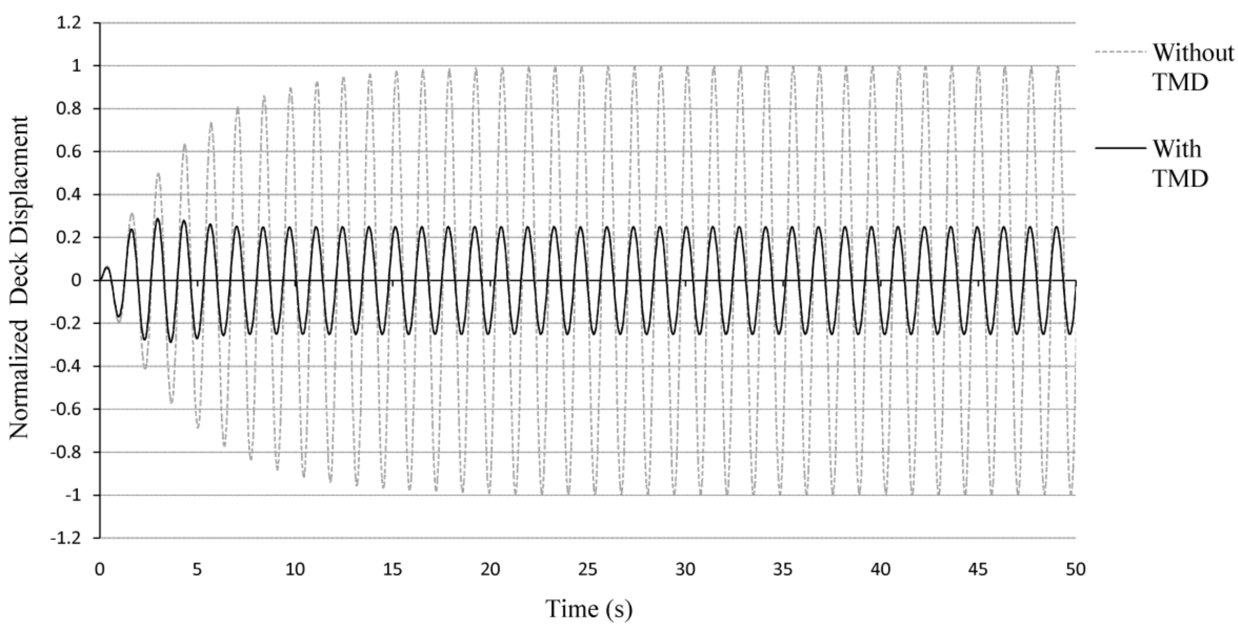


Fig. 12. Time history of deck displacement for harmonic load; W/O & W TMD (H=0.212 m, T).

Fig. 13 also shows a clearly decrease in structural response for the harmonic wave load with wave period equal to three times of the fundamental period of the structure (3T) and wave height H=1.91 m. Time history of deck acceleration for this loading has been shown in Fig. 14. It can be seen that acceleration reduces highly.

Fig. 15 shows the effect of TMD on the response of structure opposed to wind load. Mean velocity of wind with return period of 50 years at 10 m height from sea surface has been assumed to be equal to 22.5 m/s. This figure shows that TMD is effective for wind load cases as well.

3.2 Friction damper devices (FDD), seismic protection

In this research, a novel friction damper device (FDD), Mualla and Belev (2002), which is economical, can be easily manufactured and quickly installed, is used. The damper main parts are the central (vertical) plate, two side (horizontal) plates and two circular friction pad discs placed in between the steel (Fig. 16). The hinge connection is meant to increase the amount of relative rotation between the central and side plates, which in turn enhances the energy dissipation in the system. The ends of the two side plates are connected to the members of inverted V-brace at a distance r from the FDD centre. The bracing makes use of pretensioned bars in order to avoid compression stresses and subsequent buckling. The bracing bars are pin-connected at both ends to the damper and to the column bases. The combination of two side plates and one central plate increases the frictional surface area and provides symmetry needed for obtaining plane action of the device. Zero-length element of program used for modeling of the frictional hinge. In order to verify modeling assumptions, model related to Mualla article evaluated. Tabeshpour and Ebrahimian presented a simple procedure for design of friction damper (2007, 2009). Komachi et al. investigated the efficiency of FDD for rehabilitation of jacket platforms (2011).

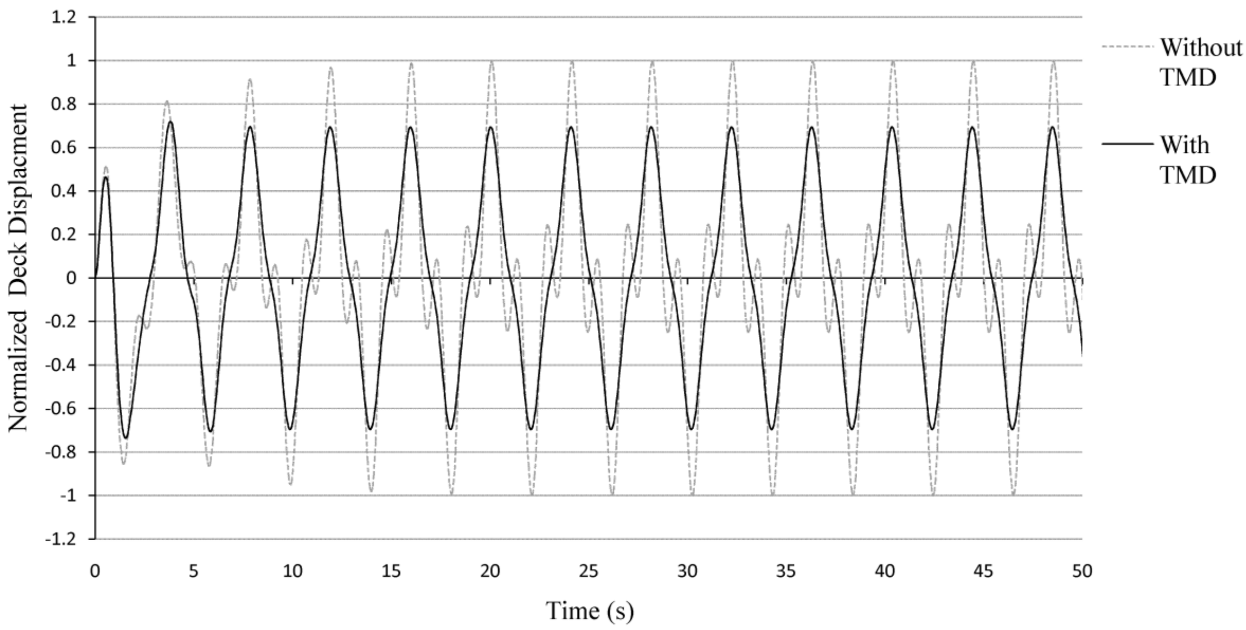


Fig. 13. Time history of deck displacement for harmonic load; W/O & W TMD (H=1.91 m, 3T).

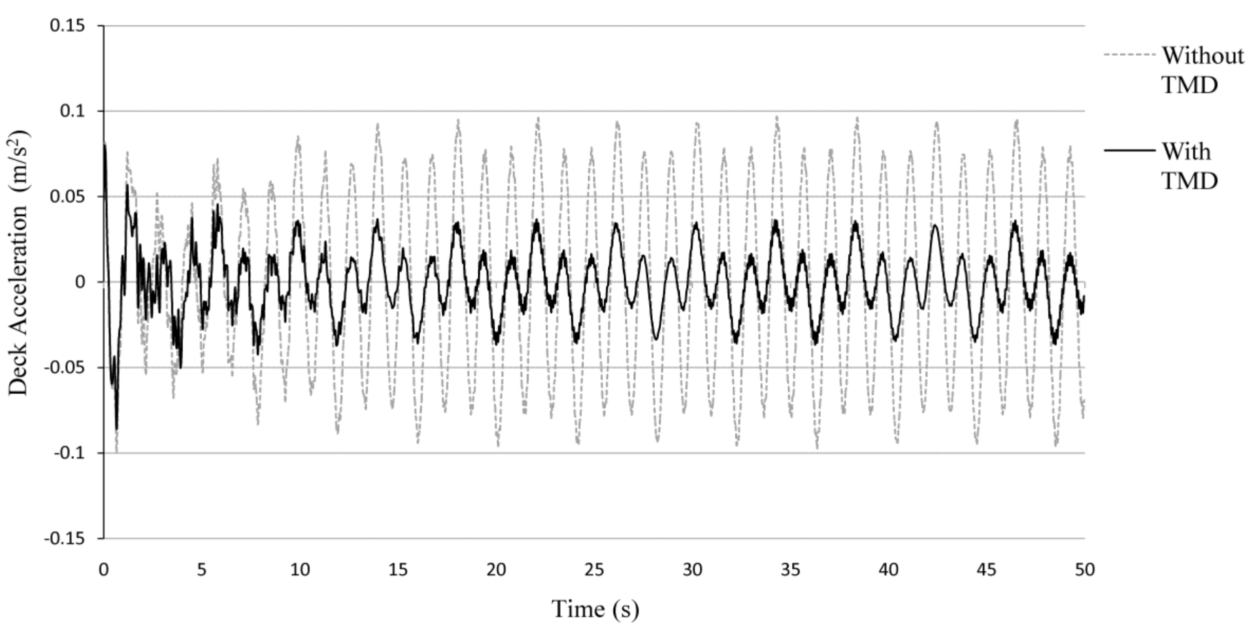


Fig. 14. Time history of deck acceleration for harmonic load; W/O & W TMD (H=1.91 m).

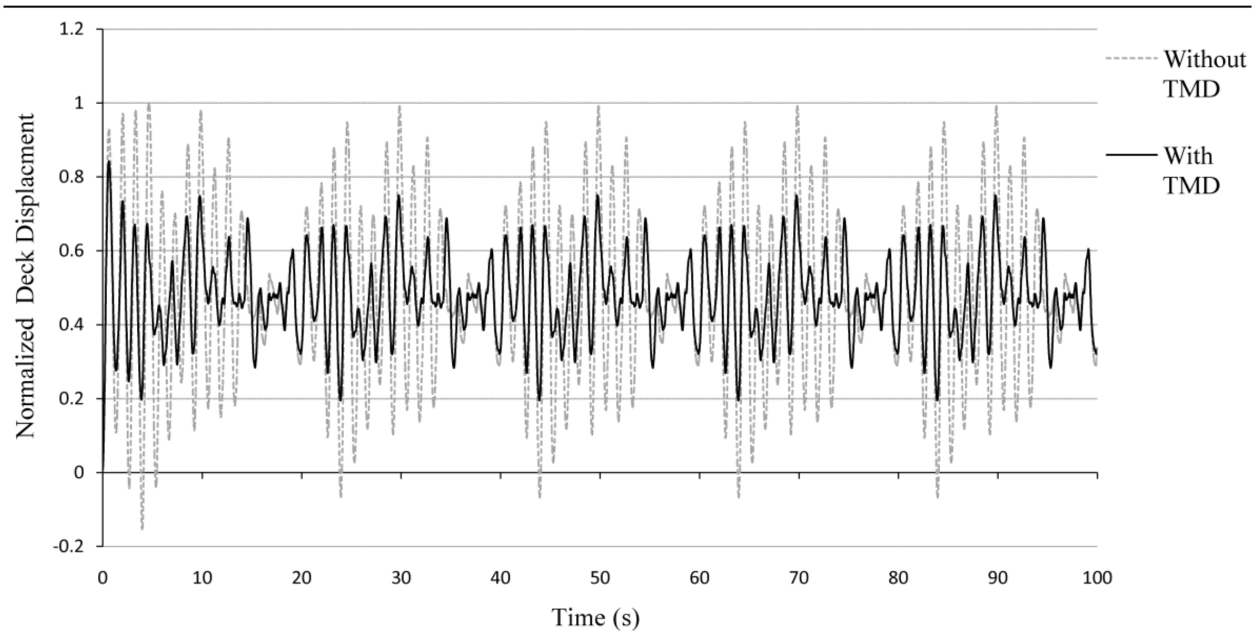
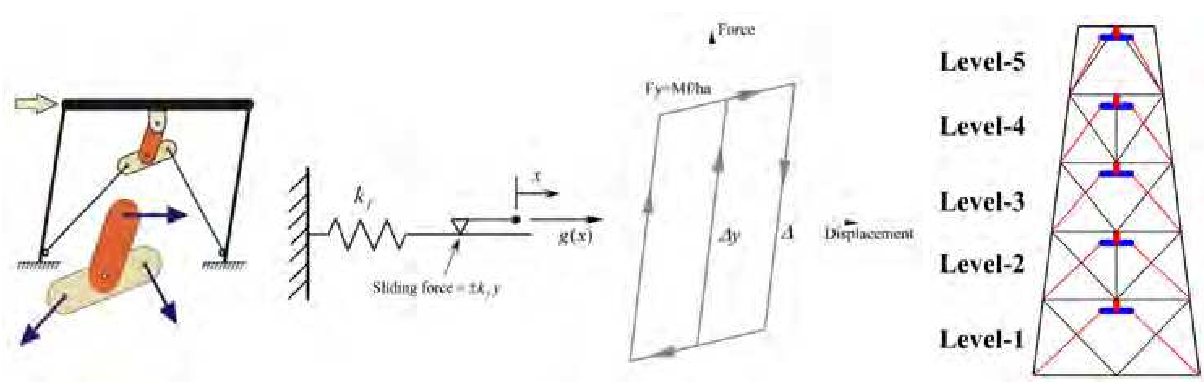


Fig. 15. Time history of deck displacement for wind load; W/O & W TMD.



a b c d

Fig. 16. a) Configuration of friction damper, b) Mathematical model, c)Hystresis behavior, d) Arrangement of FDD.

The equivalent viscous damping of this system is obtained by:

$$\beta_{eff} = \frac{2}{\pi}\frac{FR(SR-FR)}{\left(SR+FR^2\right)}, \quad \frac{FR}{SR} \prec 1 \tag{3}$$

That FR and SR are the damper properties in terms of the structure properties are defined as follows:

$$\begin{cases} FR = \dfrac{F_y}{F_e}: \ \text{the ratio of damper stiffness to total structure stiffness} \\ SR = \dfrac{K_{bd}}{K_e}: \ \text{the ratio of damper yield force to total structure force} \end{cases}$$

Fig. 17 shows the comparison of pushover curves of the jacket with and without the damper. It can be seen that the damper improves the performance of the jacket especially at the nonlinear region. Base shear of the structure vs. deck displacement (hysteresis loops) for CHY101W record is presented in Fig. 18 for cases of with and without FDD.

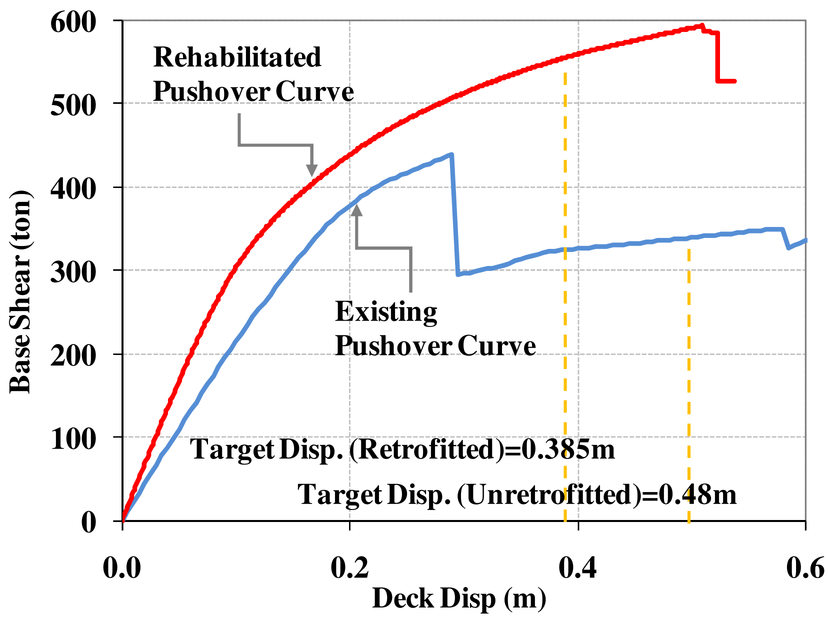


Fig. 17. Pushover curve of the jacket for rehabilitated and existing cases.

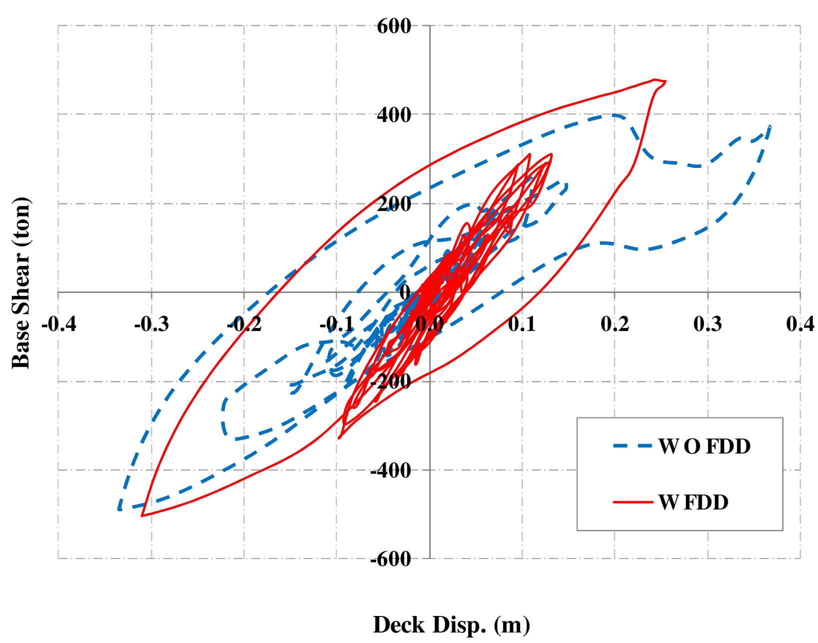


Fig. 18. Hysteresis loops of jacket for CHY101W record.

Fig. 19 shows time history of frictional hinge rotation of dampers at various levels of jacket for CHY101W record. Fig. 20 shows the time history of deck displacement for CHY101W record. This figure shows that deck displacement reduces highly (about 60%) and base shear of structure reduce about 10% too.

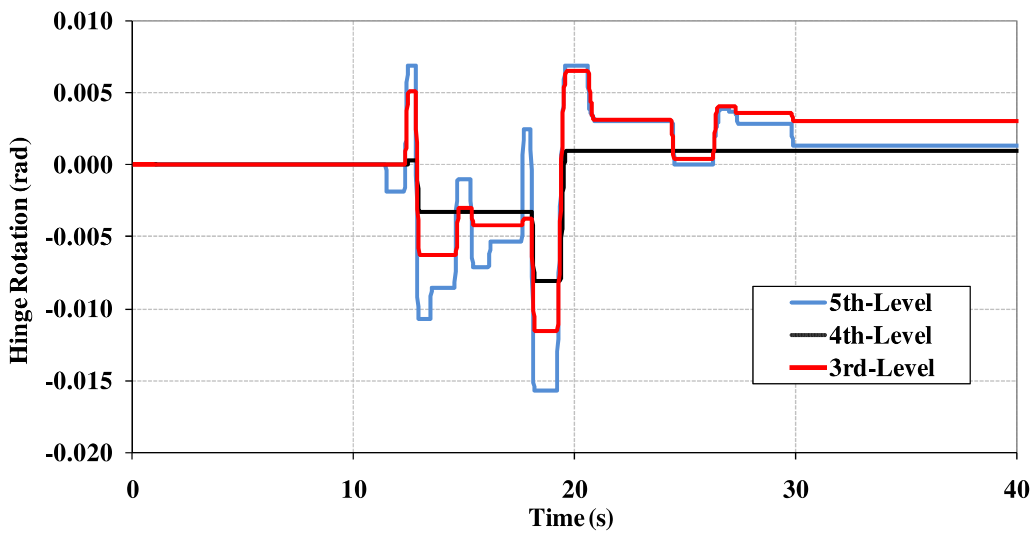


Fig. 19. Time history of friction hinge rotation for CHY101W record.

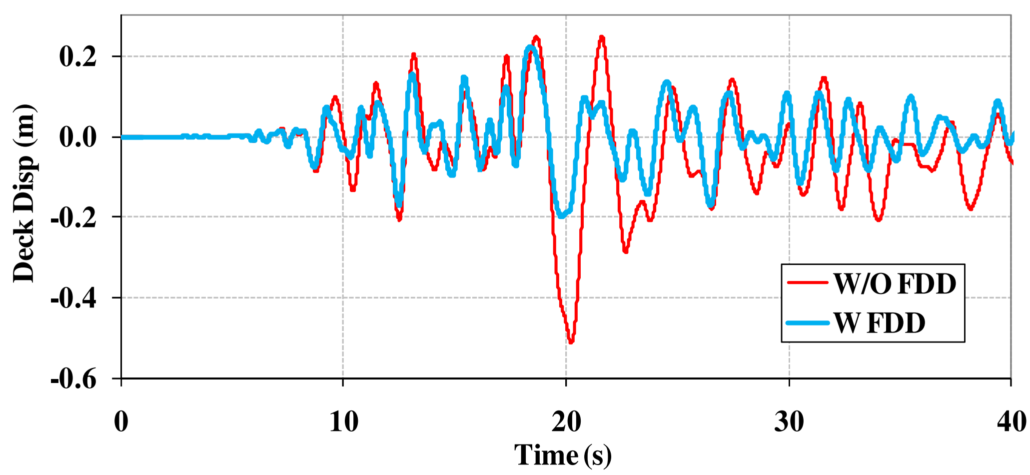


Fig. 20. Time history of deck displacement of structure for cases with and without damper for CHY101W record.

4. Conclusion

This chapter presented process of assessment and rehabilitation of jacket platform offshore structures. A seismic assessment of API standard and FEMA-356 documents was compared to each other. As an example an existing 4 legged service platform was assessed with API and FEMA-356 with respect to various earthquake hazard levels. Nonlinear static and dynamic analyses were used to determine the response of the structure. This study shows that:

1. Comparative building redundancy of a jacket platform structure is low and failure of one member in these structures can lead to immediate reduction of strength and afterward collapse of the structure.
2. However global criteria for seismic assessment of jacket platforms are presented in the API standard but there is no numerical or specific criterion in order to assess the structure. It is observed that building documents can be used to develop numerical and

applicable criteria for seismic assessment of such structures. Assessment with respect to member's criteria has some advantages such as better decision making for rehabilitation and detecting the full capacity of members.

3. Return periods related to collapse prevention in API and FEMA are 1000 and 2500 years respectively. The return period of API should probably be reviewed because the expected mean life time of the jacket is greater than design mean life time. However the approach and methodology presented in building structure documents (such as FEMA-356) is very appropriate and efficient in the seismic assessment of jacket platform structures.

The effects of near-fault earthquakes on the behavior of steel jacket platforms has been presented. Pulse type excitation has been used for investigation of structural behavior. It is shown that the maximum response of structure occurs when $T = T_p$ and also that input pulse with $\beta > 1.0$ gives a higher amplitude rather than $\beta < 1.0$, that denotes importance of ratio of period of directivity pulse to structure period. The increasing in dynamic amplitude can be more than two times than that of both static and far field responses.

Rehabilitation process using TMD and FDD for these structures presented. Effect of TMD on the response of jacket offshore platform under the wave and wind loads presented. It was shown that TMD is very effective for reduction of jacket responses under the these loads.

FDD was used on a steel jacket platform located in Iranian waters of the Persian Gulf. Results were shown that responses of jacket reduce dramatically. A numerical study was performed using pushover and nonlinear dynamic analysis. Pushover analysis results were shown that use of FDD system reduce target displacement of the structure and also was shown that a sudden decrease of jacket strength does not occur when this system is installed on the structure. Due to the low redundancy of jacket platform structures, the strength of these structures can decrease suddenly and the use of FDD systems can be extremely useful. Analysis results were shown that friction damper greatly reduces deck displacement. It was observed that for large record accelerations structure behavior becomes highly nonlinear and the performance of the friction damper for response reduction increases (for example up to 65% deck displacement reductions). Numerical studies clearly exhibit that these control systems represent a practical alternative for rehabilitation of existing jacket platforms.

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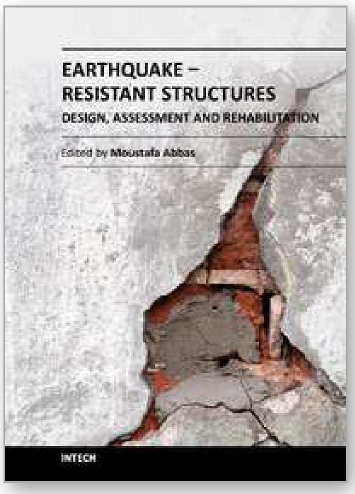

Earthquake-Resistant Structures - Design, Assessment and Rehabilitation
Edited by Prof. Abbas Moustafa

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This book deals with earthquake-resistant structures, such as, buildings, bridges and liquid storage tanks. It contains twenty chapters covering several interesting research topics written by researchers and experts in the field of earthquake engineering. The book covers seismic-resistance design of masonry and reinforced concrete structures to be constructed as well as safety assessment, strengthening and rehabilitation of existing structures against earthquake loads. It also includes three chapters on electromagnetic sensing techniques for health assessment of structures, post earthquake assessment of steel buildings in fire environment and response of underground pipes to blast loads. The book provides the state-of-the-art on recent progress in earthquake-resistant structures. It should be useful to graduate students, researchers and practicing structural engineers.

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