ISSN: 2277-3754
ISO 9001:2008 Certified
International Journal of Engineering and Innovative Technology (IJEIT)
Volume 4, Issue 2, August 2014

Dynamic Stability of Articulated Offshore Tower under Seismic Loading

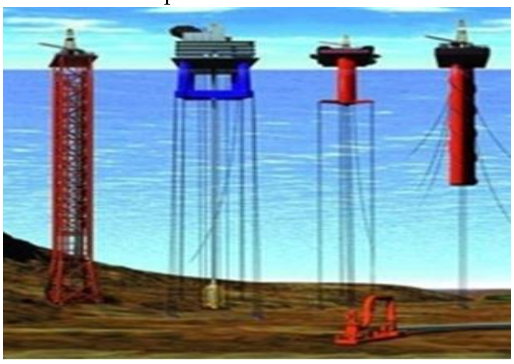
Prashant Atreya[1], Nazrul Islam[2], Mehtab Alam[1], Syed Danish Hasan[3]
[1]Department of Civil Engineering, Jamia Millia Islamia University, New Delhi, India
[2]Civil Engineering Department, Islamic University, Madina, KSA
[3]University Polytechnic, Aligarh Muslim University, Aligarh, India

Abstract - Offshore compliant structures such as articulated towers, tension leg platforms, guyed platforms etc. are economically attractive for deep-water conditions because of their reduced structural weight as compared to conventional platforms. These structures yield to the impact of wave and earthquake forces and the restoring moments are provided by large buoyant force. Compliant structures when subjected to stochastic natured sea waves, earthquake or wind loads or a combination of these loads, undergo large displacements depicting non-linear transient behaviour. These non-linearties are produced by non-linear excitation and restoring forces, damping non-linearity, etc. which may lead to complex response behaviour of articulated offshore tower. Non-linear restoring force is from the geometric non-linearity of the structure. Studies of Dynamic Stability phenomenon in such non-linear transient systems have always created interest amongst the researchers. Instant study investigates the transient behaviour of Single Hinged Articulated Tower (SHAT) under different categories of wave and earthquake loads followed by determination of its dynamic stability during various phases of loading. Minimum potential energy concept has been used to evaluate stability of the Tower followed by study of its dynamic stability pattern using two dimensional phase plots.

Keywords: **Single Hinged Articulated Tower, Earthquake, Dynamic Stability, Time History, and Phase Plot.**

I. INTRODUCTION

With the exhaustion of natural resources available on land, ocean has attracted human beings for minerals. More and more exploration activities are taking place in sea environment which have resulted in development of offshore platforms / base. The journey began with the erection of fixed type of offshore structure which in order to overcome large bending moments at base, required high strength base and large amount of material. In deep seas, these structures were found to be costly and their erection also provided challenges. This led to the development of compliant offshore structures. These structures are found to be suitable for deep sea water operations since they avoid unacceptably high hydro-dynamic loads by yielding to wave and current actions leading to economic designs. These structures float on sea water by virtue of buoyancy and are held at the sea base though latticed tower, teethers, mooring lines, chains etc. Fig.-1 shows the various types of compliant offshore structures. These structures have proved to be cost effective and offshore industry is now moving ahead with continued use of such structures. Due to their flexible nature, these structures have large displacements with inherent non-linearities, so prediction of behavior of these structures in oceanic environment is difficult and is met with many challenges. To get insight into the response behavior of these structures and to explore the possibility of their dynamic instability and chaotic motion, eefforts have been made to use simplified realistic mathematical models.



(Fig.-1 compliant offshore structures)

The presence of strong geometric non-linearity and non-linearity arising due to fluid structure interaction led to the possibility of dynamic instability of the systems. On account of these non-linearities, numerical investigations of compliant offshore structures have revealed complex behaviour involving sub-harmonic, super-harmonic and periodic solutions. Although, during last more than a decade, researchers have carried out seismic response of compliant offshore structures (Jain, A.K., and Datta, T.K.(1991)[1]; Lina, H., Youngang, T. and Cong, YI., (2006)[2]; Islam, N. and Ahmad, S.(2006)[3]; Chandrasekaran et al. 2008[4]; Islam, N., Zaheer, M.M. and Ahmad, S.(2009)[5]; Hasan, S. D., Islam, N. and Moin, K. 2011[6]), more efforts are required for stability analysis in non-linear environment. The stability analysis may consist of perturbating an approximate solution. Various methods of dynamic stability analysis of non-linear system in closed form by using analytical, semi-analytical and numerical techniques have been developed (Banik and others (2003-2009)[7,8 &9]; Friedmann et al. 1977[10]; Chua and Ushida, 1981[11]; Burton, 1982[12], Cai, G.Q. (1995)[13], Lin, Y.K. and Cai, G.Q., 1995[14]). Application of these techniques covers a wide range of application problems including standard problems of Van-Der-Pol oscillator, Duffing Oscillator, Double Pendulum etc. (Hamdan and Burton, (1993)[15]; Ravindra and Mallik, (1994)[16]; Blair et al. (1997)[17]; Yu and Bi, (1998)[18]; Mallik, A.K. and Bhattacharjee, J.K.(2005)[19]). The main focus of application problems was to study and investigate capabilities of the methods to bring out all possible instability phenomenon latent in the system. Islam Saiful, A.B.M (20130[19] has recently used two dimensional phase plots to determine dynamic stability phenomenon in SPAR

ISSN: 2277-3754
ISO 9001:2008 Certified
International Journal of Engineering and Innovative Technology (IJEIT)
Volume 4, Issue 2, August 2014

platforms. Single Hinged Articulated tower platform [SHAT] (Fig.- 2) is one of the widely used compliant offshore structure which is economically attractive especially as loading and mooring terminal in deep waters. In the instant study, single hinged articulated tower with geometrical and mechanical parameters mentioned in Table-I have been used. This platform is comparatively very light as compared to the conventional fixed platforms. The tower itself is a linear structure, flexibly connected to the sea bed through a cardon / universal joint and held vertically by the buoyancy force acting on it. The part of the tower emerging from the water i.e. deck supports the super structure designed to suit the particular application e.g. a tanker to be loaded, flaring of waste gases, etc. The buoyancy chamber just below the deck provides requisite amount of buoyancy to balance the tower, ballast and deck weight. As the connection to the sea bed is through the articulation, the structure is free to oscillate in any direction and does not transfer any bending moment to the base. The ballast chamber at the bottom lowers down the centre of gravity and provides stability to the Tower. With the impact of wind, wave or earthquake load, the tower deflects to one side thereby increasing the submerged volume of buoyancy chamber and hence uplifting force which brings back the tower to its mean position. The articulated tower which can be used at larger water depth may also have one or more number of joints at the intermediate level. Such towers having joints at the intermediate level are called multi hinged articulated tower. As the articulated tower is compliant in nature, it moves with the waves and thus the wave force and bending moment along the tower will be less as compared to a fixed structure. Usually the natural period of the towers is of the order of 40 to 90 seconds and hence dynamic amplification-factor is also small.

(Fig.-2 Amerada Hess Compliant Tower at Baldpate Garden Banks (First freestanding, non-guyed compliant tower installed in 1998)

II. METHODOLOGY

In the present work firstly a nonlinear dynamic analysis of the said structure under waves / earthquake has been carried out for its time domain responses using Langrangian approach which has the capability of equating kinetic and potential energies of the system to the rotational degrees of freedom. The random waves have been simulated by Monte-Carlo technique represented by Modified PM spectra. Modified Morison's equation has been used for estimation of hydro-dynamic loading. Water particle kinematics has been governed by Airy's linear wave theory. To incorporate variable submergence, Chakraborty's correction [20 & 21] has been applied. Seismic inputs have been applied using Northridge, Imperial valley CA, Duzce, Turkey spectra. Stability assessment has been carried out using concept of minimum potential energy and two dimensional phase plots.

A. *Assumptions and Structural Idealizations*

In the present study, the following assumptions and structural idealizations have been made for formulation of the problem in respect of single hinged articulated tower (Fig.-3):

1) Articulated tower is modelled as a stick with masses lumped at the nodes. The universal joint at base is modelled as mass-less rotational spring of zero stiffness.
2) Flexural deformations of the tower have been assumed to be negligible as compared to its displacements as a rigid body.
3) The entire tower has been discretized into 'np' number of elements of uniform length for the estimation of conservative and non-conservative forces, while diameter, mass and buoyancy may vary. The submerged elements of the tower have been subjected to time dependent hydro-dynamic loading. Due to nonlinear forces acting on the tower, the number of submerged elements shall also vary with respect to time.
4) Drag force is assumed to be proportional to the relative water particle velocity w.r.t. the structure, oscillating under wave and ground motion. The structural damping of the system is specified as a fraction of the critical damping corresponding to the un-deflected configuration of the tower.
5) Earthquake is assumed to be a broad band random stationary process described with the help of an accelogram.
6) The behaviour of the fluid surrounding the structure shall not be affected by the slow motion of the compliant tower.
7) Analysis due to earthquake excitation and due to wave forces are carried out independently, and therefore water particle kinematics is taken to be negligible for the seismic forces. Only two dimensional motion of the tower in the plane of the environment loading have been considered in the analysis.

III. ESTIMATION OF LOAD ON STRUCTURE

A. Wave Loads

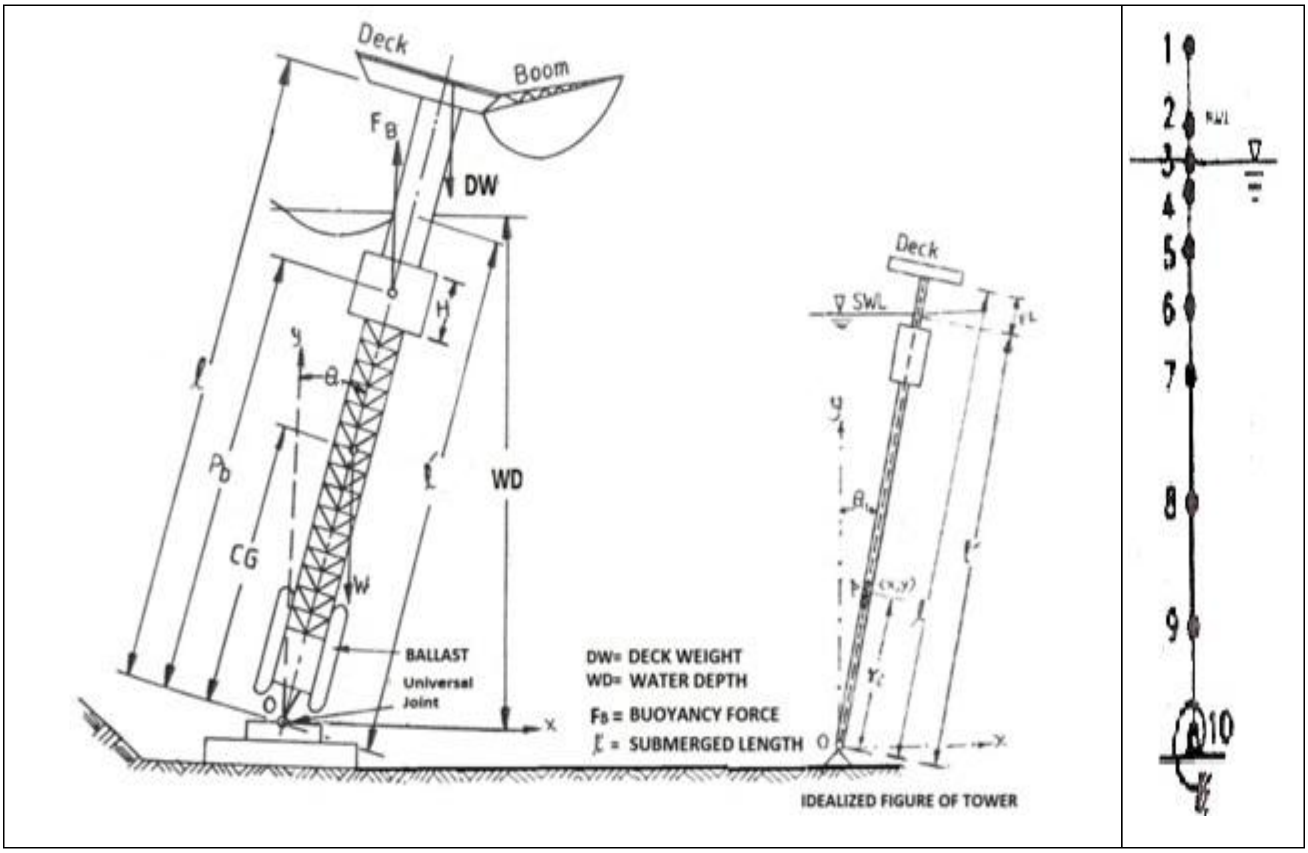
A variety regular waves as suggested by Hasan, S. D., Islam, N. and Moin, K.(2011)[6] and Jameel & Ahmad (2011)[22] have been considered. In order to investigate the combined effect of current and earthquake, current velocities of 1.0 m/s, uniform throughout the depth have also been considered. The influence of various parameters such as variable buoyancy, added mass, instantaneous towers orientation, variable submergence and effect of current, on the response of SHAT and its stability aspects

ISSN: 2277-3754
ISO 9001:2008 Certified
International Journal of Engineering and Innovative Technology (IJEIT)
Volume 4, Issue 2, August 2014

has been investigated in detail. To calculate the wave force on latticed articulated tower, the latticed tower has been replaced with cylindrical shaft of equivalent diameter. Wave forces on the submerged part of the latticed tower (cylindrical shaft) have been estimated by the modified Morison's equations, which duly takes into account the relative motion of the structure and water. The water particle velocities and accelerations have been stipulated by Airy's wave theory. To incorporate the effect of variable submergence, Chakrabarti's approach has been adopted. The transformation matrix has been used to compute the normal and tangential component of the hydrodynamic forces on each element of the tower corresponding to instantaneous deformed configuration of the tower. The updated mass-moment of inertia of the tower has been incorporated in the consistent mass and damping matrices. Newmark's Beta integration scheme has been deployed to solve the equation of motion taking into account all non-linearities involved in the system.

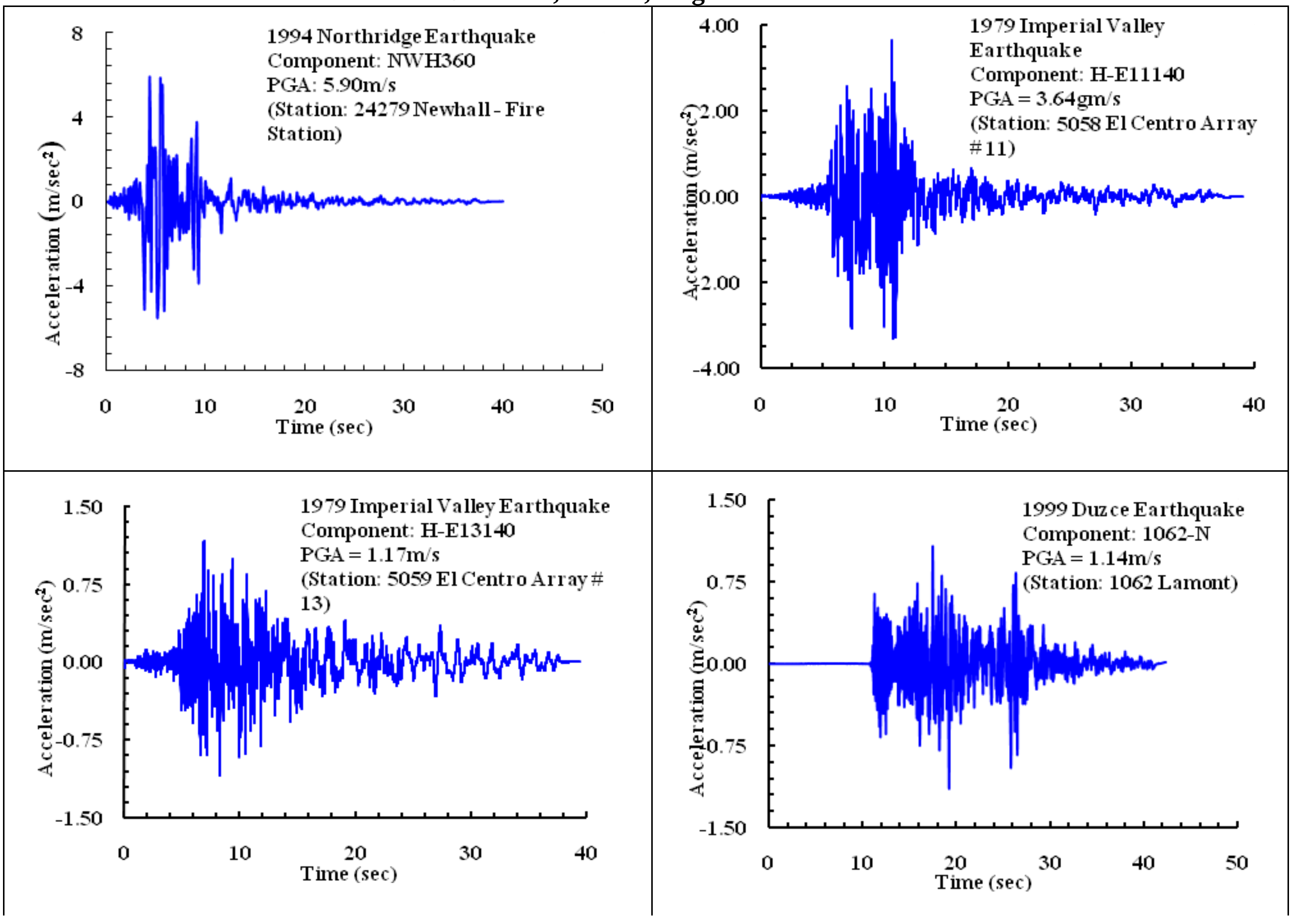
B. Seismic Loading

Accelograms (Northridge, Imperial Valley CA, and Duzce Turkey) have been used to provide input for ground acceleration time history (Fig.-4) for calculating seismic response of the tower by time history analysis. The wave loading are not correlated with seismic loadings. The two analysis are carried out independently. The analysis under earthquake alone is carried out using water particle kinematics as zero. To observe the behaviour due to the combined wave and earthquake forces, numerical studies are conducted to investigate the effects of initial conditions, current and wave on the seismic response of the tower. Without wave, the tower is assumed to have zero displacement and zero velocity at time t=0. When wave and earthquake are considered to act together, different initial conditions of the tower are assumed depending upon the oscillating state of the tower at the instant when the structure encounters the earthquake. Further, it is assumed that the earthquake forces act on the structure when it oscillates in a steady state under the regular sea. The responses induced due to earthquake are further compared with the responses due to strong sea-state / waves / waves in order to establish relative severity of the two independent events. The commencement of the earthquake has been considered such that the first prominent peak of accelogram matches with the crest of the wave in order to get the maximum impact of wave and earthquake load. The seismic responses of tower are further compared with the response due to strong sea states.



(Fig. 3- SHAT Model)

IJEIT
ISSN: 2277-3754
ISO 9001:2008 Certified
International Journal of Engineering and Innovative Technology (IJEIT)
Volume 4, Issue 2, August 2014



(Fig.-4(A) – Earthquake Accelograms)

Source of Earthquake Data-http://peer.berkeley.edu/smcat/earthquakes.html

IV. EQUATIONS FOR PROBLEM SOLUTION

A. Equation of Motion

The equation of Motion is derived using Lagrange's approach wherein kinetic and potential energy of the system are related in terms of rotational degrees of freedom.

$$\frac{d}{dt}\left[\frac{\partial KE}{\partial \dot{\theta}}\right] - \frac{\partial (KE)}{\partial \theta} + \frac{\partial (PE)}{\partial \theta} = M_\Theta \quad \textbf{(4.1.1)}$$

K.E. of Tower $= \frac{1}{2} I \, \dot{\theta}^2$ **(4.1.2)**

Where

$$I = \frac{1}{2} \sum_{i=1}^{nsp} m^*_i (r_i \dot{\theta})^2 + \frac{1}{2} \sum_{i=(nsp+1)}^{np} m_i (r_i \dot{\theta})^2 + I_d + m_d \, l_c^2$$

$$I = I^* + I_d + m_d \, l_c^2$$

Similarly Potential Energy of the tower (P.E. =∑ mgh) obtained as

$$P.E. = \{ \sum_{i=1}^{np} m_i r_i - \sum_{i=1}^{nsp} fb_i r_i \} g.\cos \Theta + m_d \, g. \, l_c \cos \Theta \quad \textbf{(4.1.3)}$$

$$\frac{(\partial KE)}{\partial \dot{\theta}} = \frac{1}{2} 2 I \dot{\theta} = I \dot{\theta} \quad \textbf{(4.1.4)}$$

$$\frac{d}{dt}\left[\frac{\partial KE}{\partial \dot{\theta}}\right] = I \ddot{\Theta} \quad and \quad \frac{\partial (KE)}{\partial \theta} = 0 \quad \textbf{(4.1.5)}$$

$$\frac{\partial (PE)}{\partial \theta} = -\{ \sum_{i=1}^{np} m_i r_i - \sum_{i=1}^{nsp} f b_i r_i \} g \sin \theta - m_d g l_c \sin \theta \quad \textbf{(4.1.6)}$$

Putting these values in equation (4.1.1), equation of motion is obtained as

$$I \ddot{\theta} - 0 + \{ \sum_{i=1}^{nsp} f b_i r_i - \sum_{i=1}^{np} m_i r_i - m_d \, l_c \} \, g \sin \theta = M_\Theta$$

or

$$[I] \, \ddot{\theta} + [\{ \sum_{i=1}^{nsp} f b_i r_i - \sum_{i=1}^{np} m_i r_i - m_d l_c \} \, g \, \frac{\sin\theta}{\theta}] \, \theta = M_\Theta$$

or **[M] { $\ddot{\Theta}$ } + [K] { Θ } = { M_Θ }** **(4.1.7)**

From above it is shown that [M] consists of mass moment of Inertias of all the elements including the deck, about the hinge and M_Θ is Moment due to Non conservative forces and

$$[K] = \{ \sum_{i=1}^{nsp} f b_i r_i - \sum_{i=1}^{np} m_i r_i - m_d l_c \} \, g \, \frac{\sin\theta}{\theta}$$

Referring Fig.-3, θ = angular displacement, $\dot{\theta}$ = angular velocity, $\ddot{\theta}$= angular acceleration, I = Total mass moment of Inertia of the Tower, fb_i is the buoyancy of ith element [$fb_i = \sum_{i=1}^{nsp} (\rho_w \frac{\pi}{4} (D_{EB})_i^2 \, ds_i$], D_{EB} = Equivalent diameter for buoyancy of the element, ρ_w = mass density of sea water (taken as 1025 kg/m^3 presently)position, m_i = Structural mass of the i_{th} element, m^*_i = Total mass ($m_i + a \, m_i$) of the submerged element[am_i = added mass of the i_{th} element $=(C_M-1)\rho_w\frac{\pi}{4}(D_{EI})^2{}_i ds_i]$, r_i = position vector of the number i_{th} element to the universal joint , ds_i = height of each element of tower, l = total height of tower, np = number of discretized parts, nsp = number of submerged elements at any instant of time, , $(C_M\text{-}1)$ = coefficient of added mass, C_M = coefficient of inertia (its value is taken as 2.0), $(D_{EI})_i$ = coefficient diameter of inertia for the i^{th} element, m_d = mass of deck, I_d = Moment of Inertia of deck about its centre, l_c = height of centre of mass of deck from hinge $[l_c = l + \rho_{cm}]$, ρ_{cm} = position of Centre of mass of deck above top of the tower, g = acceleration due to gravity.

B. Wave Velocities and loads

Wave Velocity

Airy's linear wave theory has been deployed for calculating wave velocities as stated:

Horizontal velocity:

$$u = \frac{\pi H}{T}\frac{\cos h\,(ks)}{\sin h(kd)} \cos (kx - \sigma t) \qquad \textbf{(4.2.1)}$$

Vertical velocity:

$$v = \frac{\pi H}{T}\frac{\sin\ h\,(ks)}{\sin h(kd)} \sin (kx - \sigma t) \qquad \textbf{(4.2.2)}$$

Corresponding water particle accelerations are obtained by time derivatives as:

Horizontal acceleration:

$$\dot{u} = \frac{\partial u}{\partial t} = \frac{2\pi^2 H}{T^2}\frac{\cos\ h\,(ks)}{\sin h(kd)} \sin (kx - \sigma t) \qquad \textbf{(4.2.3)}$$

Vertical acceleration:

$$\dot{v} = \frac{\partial v}{\partial t} = -\frac{2\pi^2 H}{T^2}\frac{\sin\ h\,(ks)}{\sin h(kd)} \cos (kx - \sigma t) \qquad \textbf{(4.2.4)}$$

Where, L= wave length, H = Wave height = Wave period, d= Water depth, $k = \frac{2\pi}{L}$ = Wave number, $\sigma = \frac{2\pi}{T}$ = wave frequency, s = vertical distance of the point under consideration, x = horizontal distance of the point measured in X- direction as the wave travel from the initial position of the tower, t = time at an instant To incorporate the effect of variable submergence, Chakrabarti's(1975) approach shall be adopted in which instantaneous sea surface elevation or fluctuating free surface effect can be incorporated as $(d + \eta)$ instead of water depth (d). The fluctuating free surface effect can be significant when the wave height is not small in comparison to the water depth.

Wave Load

The force $dF(t)$ due to wave on differential section of length ds_i of the cylinder is made up of two components namely inertia force component which is proportional to the normal component of the fluid particle acceleration and drag force which is proportional to the square of the normal component of the fluid particle velocity thus:

$$dF(t) = [\tfrac{\pi}{4} D^2\ \rho_w C_M\ (\ddot{u}) + \tfrac{1}{2}\rho_w D C_D\ \dot{u}\,|\ \dot{u}\ |]\ ds_i \qquad \textbf{(4.2.5)}$$

Where, D = Outer diameter of the cylinder, $\dot{u}$ = fluid particle velocity normal to the member, $\ddot{u}$ = fluid particle acceleration, C_M = Inertia co-efficient, C_D = Drag co-efficient.

When both wave and current forces are considered together, equation (3.2.5) shall be modified as follows:

$$dF(t) = [\ C_M\ \rho_w\ (\tfrac{\pi}{4} D^2\ )\ (\ddot{u}) + \tfrac{1}{2}\rho_w\ C_D\, D\, (v_c + \dot{u})\ |\ v_c + \ \dot{u}\ |\ ]\ ds \qquad \textbf{(4.2.6)}$$

where v_c is the water particle velocity normal to the member due to current only. Equation (4.2.5) can also be modified to account for relative velocity and acceleration between the fluid particle and the structure ($\dot{x}$ and $\ddot{x}$). The drag and inertia forces get modified as under:

$$dF(t) = [\ C_M\ \rho_w\ (\tfrac{\pi}{4} D^2\ )\ (\ddot{u} - \ddot{x}) + \tfrac{1}{2}\rho_w\ C_D\, D\ (\ \dot{u} - \dot{x})\ |\ \dot{u} - \dot{x}\ | + \tfrac{\pi}{4} D^2 \rho_w \ddot{x}]\ ds_i \qquad \textbf{(4.2.7)}$$

The above equations, through integration, shall be used to determine the instantaneous hydrodynamic loading along the submerged height of the shaft. Moments about the axes of rotation, due to these forces are determined by multiplying the differential force equation by the appropriate moment arms and then integrating over the length of the cylindrical shaft to obtain the total moment. As the total height of the shaft shall be divided into a finite number of elements for determination of the wave forces and moments, the total force is obtained by the summation of all elemental values.

C. Equation of motion under seismic forces

The equation of motion in terms of rotational degrees of freedom for combined earthquake and wave loading is written as:

$$[I]\{\ddot{\theta}\}+[C]\{\dot{\theta}\}+[K]\{\theta\}=\{M_\theta^h(t)\}+\{M_\theta^{eq}(t)\} \qquad \textbf{(4.3.1)}$$

Where, $\{M_\theta^h(t)\}$–Moment due to hydrodynamic loading including the effect of non-linearities and current, $\{M_\theta^{eq}(t$ }- Moment due to earthquake loads

The forcing functions due to wave and current only are expressed as:

$$F^h(t) = [M_e]\{\ \ddot{u} - \ddot{x}\} + [\ M]\ \{\ \ddot{x}\} + 0.5\rho_w C_D\ [A]\ \{(\ \dot{u} - \dot{x} + v_c)\ |\dot{u} - \dot{x} + v_c\ |\ \} \qquad \textbf{(4.3.2)}$$

Under the combined effect of wave and earthquake, the drag and inertia forces will be modified by replacing ($\dot{x}$) by ($\dot{x}_g$+ ẋ) and ($\ddot{x}$) by ($\ddot{x}_g$ + ẍ) leading to the following expressions:

ISSN: 2277-3754
ISO 9001:2008 Certified
International Journal of Engineering and Innovative Technology (IJEIT)
Volume 4, Issue 2, August 2014

$F^{eq}(t) = [-M_e]\{\ddot{x}_g\}+[M]\{\ddot{u}\}+0.5\rho_w C_D[A]\{(\dot{u}-(\dot{x}_g+\dot{x})+v_c)\,|\dot{u}-(\dot{x}_g+\dot{x})+v_c|\}$ **(4.3.3)**

For the consideration of earthquake alone the forcing function is modified by replacing ($\ddot{u}$) = 0 and ($\dot{u}$) = 0. Thus

F^{eq}(t) alone $= -[M_e]\{\ddot{x}_g\}+0.5\rho_w C_D[A]\{-(\dot{x}_g+\dot{x})\,|-(\dot{x}_g+\dot{x})|\}$ **(4.3.4)**

Where,

$M_e = [M]+[M_a]$ i.e. total mass matrix, $[M_a]$ = Added mass matrix $=(C_M - 1)\rho_w V$, V = lumped volume of the tower at a node, A = Projected Area, x, $\dot{x}$, $\ddot{x}$ = tower's linear displacement, linear velocity and linear acceleration respectively.

u, $\dot{u}$, $\ddot{u}$ = water particle linear displacement, linear velocity and linear acceleration respectively.

$\dot{x}_g$, $\ddot{x}_g$ = seismic ground velocity and ground acceleration respectively.

V. DETERMINATION OF STABILITY OF TOWER

A. Stability solutions Using Minimum Potential Energy Concept

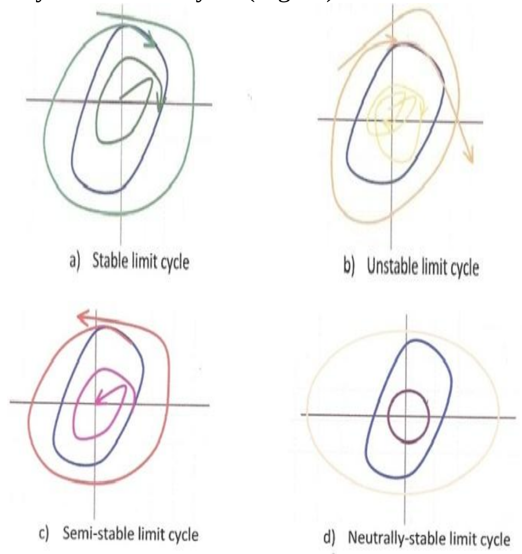
If the amplitude of vibration decreases with time, the system is said to be stable. If the transient increases indefinitely with time, the system is said to be unstable. For stable systems, the total energy in the system decreases with time. The loss of energy is usually dissipated as heat due to friction. Therefore damping for stable systems must be positive. For unstable systems, energy must have been kept adding to the system because there is continuous increase in amplitude of vibration. Work is therefore done on the system by the damping force. Hence damping for unstable systems is negative. For conservative systems, the principle of minimum potential energy (Seto, William W. (1989)[24])can be used to test the stability of a system. A system will be stable at an equilibrium position if the potential energy of the system is a minimum for that position, i.e.,

$$\frac{d(PE)}{dq} = 0 \text{ and } \frac{d^2(PE)}{dq^2} > 0 \qquad \textbf{(5.1.1)}$$

Where P.E. = Potential energy of the system, q = Generalised coordinates In the instant work, stability has been worked out by evaluating net impact of stabilizing forces on the tower. However, serviceability requirements which are determined on the basis of inclination of the tower from the mean position have also been kept in mind. In most of the cases, although the stabilizing forces may bring back the tower to its mean position, the towers have been declared as unserviceable due to larger inclinations on application of loads (viz. more than 2 degrees in case of drilling terminals, more than 4 degrees in case of mooring terminals and more than 5 degrees in case of flaring terminals).

B. Determination of Dynamic Stability using Two Dimensional Phase Plots

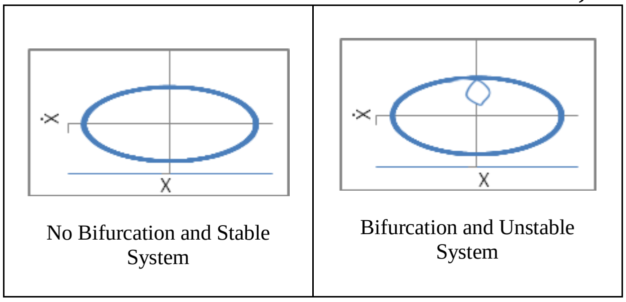
A phase portrait is a collection of trajectories that represent the solution of these equations in the phase space. To obtain phase plots, velocities are plotted on abscissa and displacement / rotation are plotted on x-axis. A trajectory is closed if and only if it corresponds to a periodic solution of a system. When a system approaches periodic behaviour and a closed curve in phase plane is observed, the closed path is called a limit cycle. A close trajectory of a dynamic system which has nearby open trajectories spiralling towards it both from inside and outside is called stable limit cycle(Fig-4a). If nearby open trajectories spiral away from closed path on both sides, the close trajectory is unstable limit cycle(Fig-4b). The thick line closed trajectory is limit cycle and other paths are neighbouring open trajectories. When the neighbouring trajectories spiral towards the limit cycle from one side and spiral away from the other side, it is semi-stable limit cycle (Fig-4c). If nearby trajectories neither approach nor recede from closed trajectory, it is neutrally-stable limit cycle (Fig-4d).



(Fig.-4(B) Limit Cycles)

In Phase Plots, instability phenomenon is shown in form of symmetry breaking bifurcations caused by nT sub-harmonic / super-harmonic oscillations and a periodic responses. In a dynamical system, a bifurcation occurs when a small smooth change made to the bifurcation parameter causes a sudden qualitative or topological alteration in structural behaviour(Fig.5). When the symmetry of a phase plot is disturbed, bifurcation is termed as symmetry breaking bifurcation. Sub-harmonic oscillations occur when the time period of subsequent cycle lessens by 1/n times than the previous time period. When the time period of subsequent cycle increases n times of previous time period, the oscillation is super-harmonic.

ISSN: 2277-3754
ISO 9001:2008 Certified
International Journal of Engineering and Innovative Technology (IJEIT)
Volume 4, Issue 2, August 2014



(Fig.5 Phase plots of stable / Unstable Systems)

VI. DISCUSSION OF RESULTS ON SEISMIC RESPONSE AND DYNAMIC STABILITY

In the Instant study, SHAT was subjected to a variety of regular wave load along with Northridge, Imperial valley CA, Duzce Turkey earthquake excitation at different starting time. The responses are obtained in terms of heel angle rotation, tip displacement; shear force, bending moment, axial force, net stabilizing moments and time required for dynamic stability etc. are given in Table-II. Further, response and stability parameters examined under four categories of earthquakes are given in Table III. Tower was also subjected to variation in dimensions of Buoyancy chamber height and diameter and its behaviour was examined. Table IV and Table V contains response and stability parameters when Tower was subjected to variation in Buoyant chamber height and diameter respectively. Stability of tower was checked using principles of minimum potential energy. Net positive stabilizing moments under various loading conditions bring the tower back to its mean position. Time to achieve dynamic stability status was determined using two dimensional phase plots.

Table 1 - Geometrical and mechanical Properties of SHAT Under Study

GEOMETRIC CHARACTERISTICS		Effective Diameter for added mass	4.5 m
Height of Tower (l)	400 m	Effective Diameter for Drag	13 m
Water depth (d)	350 m	Effective Diameter for Inertia	4.5m
Height of Ballast (H_{BL})	120 m	**MECHANICAL PROPERTIES**	
Height of Buoyancy chamber (H)	70 m	Deck Mass (M_D)	2.5 x 10^6 Kg
Position of Buoyancy chamber (P_{BC})	310 m	Structural Mass (SMT) of Tower	2.0 x 10^4 Kg/m
For Chamber		Mass of Ballast (MBT)	44840 Kg/m
Effective Diameter for Buoyancy (D_B)	20 m	**MECHANICAL OSCILLATIONS**	
Effective Diameter for Added Mass	7.5 m	Time period	29.47 sec.
Effective Diameter for Drag	14.5 m	**HYDRODYNAMIC SPECIFICATIONS**	
Effective Diameter for Inertia	7.5 m	Drag Coefficient (C_D)	0.6
For Tower shaft		Inertia Coefficient (C_M)	2.0
Effective Diameter for Buoyancy (D_B)	7.5 m	Mass Density of Sea Water	1024 Kg / cu-m

Table –II – Comparative study of responses for SHAT subjected to variety of regular waves and Northridge earthquake

Sl. No.	Load Case	Net Stabilizing Moment (Nm)	Abs. Max^m Heel Angle Rotation(Deg.)	Abs. Max^m Deck Disp. (m)	Abs. Max^m SF (N)	Abs. Max^m AF (N)	Abs. Max^m BM (N)	Dynamic Stabilization Period after Wave / EQ (sec)	Remark
1	Regular Wave Alone(H-2.15m, T-4.69 s)	5.49E+10	0.44	3.08	4.24E+06	2.26E+08	8.73E+10	610	S,S&US
2	Regular Wave Alone(H-11.15m, T-10.69 s)	5.48E+10	1.32	9.25	5.50E+06	2.26E+08	1.15E+11	350	S,S&US
3	Regular Wave Alone(H-17.15m, T-13.26 s)	5.47E+10	1.71	12.02	6.16E+06	2.26E+08	1.28E+11	250	S,S&US
4	Regular Wave Alone(H-17.15m, T-13.26 s, C-1 m/s)	5.48E+10	1.67	11.69	6.15E+06	2.26E+08	1.31E+11	230	S,S&US
5	Regular Wave (H-2.15m, T-4.69 s)with Northridge EQ at 0.0 sec	5.42E+10	4.19	29.34	1.03E+07	2.42E+08	2.15E+11	610	US,S&US
6	Regular Wave (H-2.15m, T-4.69 s)with Northridge EQ at 653.1 sec(After stabilization and at crest of wave)	5.40E+10	4.43	31.03	1.04E+07	2.42E+08	2.15E+11	610	US,S&US
7	Regular Wave with(H-17.15m, T-13.26 s) with Northridge EQ at 308.7 sec at wave crest	5.40E+10	5.00	35.03	1.14E+07	2.39E+08	1.88E+11	300	US,S&US

ISSN: 2277-3754
ISO 9001:2008 Certified
International Journal of Engineering and Innovative Technology (IJEIT)
Volume 4, Issue 2, August 2014

8	Regular Wave with(H-17.15m, T-13.26 s, C-1m/s with Northridge EQ at 308.7 sec.)	5.41E+10	4.92	34.51	1.09E+07	2.39E+08	1.80E+11	280	US,S&US
9	Strong large size Regular wave alone (H-30m T-15 s)	5.35E+10	2.52	17.66	1.28E+07	2.30E+08	2.77E+11	200	US,S&US
10	Only Northridge EQ in calm sea at 0 sec	4.64E+10	8.14	56.96	8.48E+07	2.77E+08	1.75E+12	1000	US,S&US

S,S&US - Serviceable, Safe & Unstable during Initial Wave / EQ US,S&US - Unserviceable, Safe & Unstable during Initial wave / EQ

Table-III – Comparative study of responses for SHAT subjected to Regular Wave & Variety of Earthquakes

Sl. No.	Load Case	Net Stabilizing Moment (Nm)	Abs. Max^m Heel Angle Rotation (Deg.)	Abs. Max^m Deck Disp. (m)	Abs. Max^m SF (N)	Abs. Max^m AF(N)	Abs. Max^m BM(N)	Dynamic Stabilization Period after Wave / EQ(sec.)	Remark
1	Regular Wave Alone	5.65E+10	0.64	4.47	2.60E+06	3.55E+07	5.42E+10	610	S&S
2	Regular Wave with Northridge NWH360 EQ(loading at Crest at 616 sec.)	5.62E+10	2.14	15.04	3.04E+06	2.34E+08	6.25E+10	640	S&US-EQ
3	Regular Wave with Northridge NWH360 EQ loading at Trough at 611.1 sec.)	5.62E+10	2.41	16.94	3.02E+06	2.35E+08	6.46E+10	640	S&US-EQ
4	Regular Wave with Imperial Valley, CA H-E11140 EQ (loading at Trough at 611.1 sec.)	5.62E+10	2.41	16.88	2.96E+06	2.28E+08	6.28E+10	640	S&US-EQ
5	Regular Wave with Imperial Valley, CA H-E13140 EQ (loading at Trough at 611.1 sec.)	5.63E+10	2.41	16.88	2.96E+06	2.22E+08	6.28E+10	640	S&US-EQ
6	Regular Wave with Duzce, Turkey 1062-N EQ (loading at Trough at 611.1 sec.)	5.63E+10	2.41	16.91	3.16E+06	2.22E+08	7.02E+10	640	S&US-EQ

S&S - Stable and Serviceable S&US-EQ - Stable and Unserviceable during EQ

Table-IV – Effect of change in Buoyancy Chamber Height on Stability

Buoyant Chamber Height(m)	Total Destabilizing Moment(Nm)	Net Stabilizing Moment(Nm)	Maxm Angular Disp.(Deg.)	Maxm Deck Displacement(m)	Remarks
70	3.13E+10	4.58E+10	3.19	22.38	S&U
90	3.16E+10	5.68E+10	2.90	20.36	S&U
130	3.21E+10	9.02E+10	2.26	15.83	S&U
170	3.26E+10	1.12E+11	2.08	14.59	S&S

Note:-Efforts have been made to bring the Heel Angle disp near to 2 deg. by varying Height of Buoyancy Chamber so that SHAT is within serviceable limits

S&S - Stable and Serviceable S&U - Stable and Unserviceable

Table-V – Effect of change in Buoyancy Chamber Diameter on Stability

Buoyant Chamber Diameter (m)	Total Desabilizing Moment(Nm)	Net Stabilizing Moment(Nm)	Maxm Angular Disp.(Deg.)	Maxm Deck Displacement(m)	Remarks
20	3.13E+10	4.58E+10	3.19	22.38	S&U
22	3.12E+10	5.83E+10	2.55	17.89	S&U
24	3.12E+10	7.19E+10	2.07	14.54	S&S

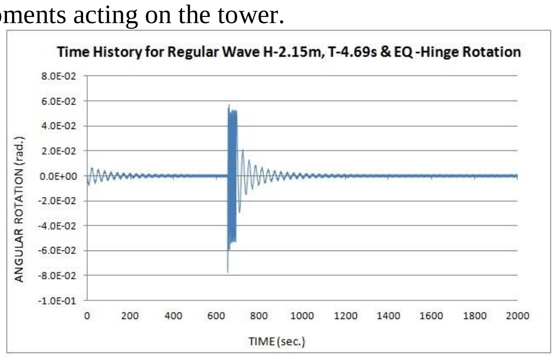
ISSN: 2277-3754
ISO 9001:2008 Certified
International Journal of Engineering and Innovative Technology (IJEIT)
Volume 4, Issue 2, August 2014

25	3.11E+10	7.93E+10	1.87	13.09	S&S

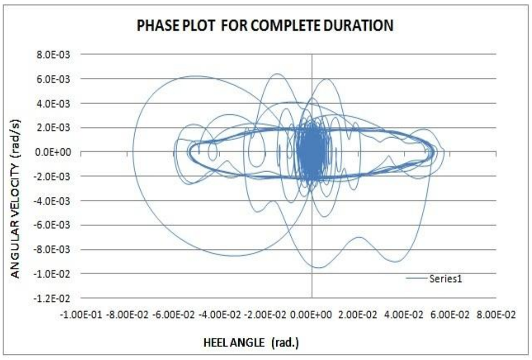
Note:-Efforts have been made to bring the Heel Angle disp near to 2 deg. by varying Dia of Buoyancy Chamber so that SHAT is within serviceable limits during EQ as well. Effect of change is Diameter size is much faster as buoyancy is proportional to square of Diameter.

S&S - Stable and Serviceable S&U - Stable and Unserviceable

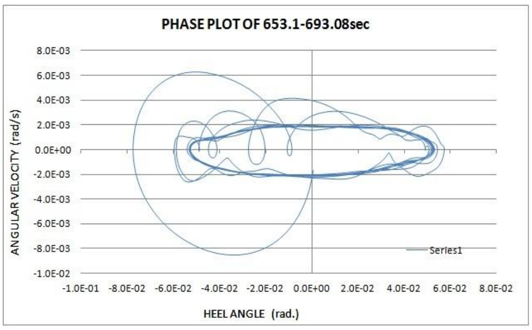
To illustrate the dynamic stability phenomenon during various phases of loading, one of the load case wherein SHAT has been subjected to regular wave load of H-2.15m and T-4.69s and Northridge earthquake at starting time of 653.1 seconds is demonstrated. Wave load was initially applied at 0 second. After initial wave load, hydrodynamic dampening took 610 seconds to bring the motion to dynamically stable state. Thereafter, Northridge Earthquake was applied at 653.1 second i.e. at the crest of wave. Time history plot for Hinge angle rotation, is given at Fig.6, which provides information about magnitude of hinge angle responses generated due to Wave and Earthquake loads. The phase plots as shown from Fig.7 to Fig.11 have been generated for the complete period of 2000 seconds, from 653.1 to 693.08 seconds, from 693.08 to 1000 seconds, from 1000 to 1300 seconds and from 1300 to 2000 seconds. From phase plots, it was observed that motion from 653.1 to 693.08 seconds was under earthquake impact and showed bifurcations and chaos. During Earthquake, absolute Maximum heel angle was 4.43 degrees, which was outside the permitted serviceable limits of the Tower. The earthquake of 39.98 seconds duration was applied at 653.1seconds, after the earthquake impact i.e. at 693.08 seconds the hydrodynamic dampening started affecting the motion. However, from 693.08 to 1000 seconds period, the motion has been non-harmonic, a periodic and asymmetric. It took another 300 seconds for trajectories to further settle down under the impact of hydrodynamic dampening forces and to start moving towards stable limit cycle. By 1300 seconds, i.e. after 650 seconds of removal of Earthquake impact, the responses became harmonic, symmetric and periodic and perfectly stable. From above discussion it is clear that with the onset of Northridge earthquake at 653.1 seconds, the tower becomes unserviceable as well as dynamically unstable for a period of approximately another 350 seconds. Then it showed the stable behaviour. During the entire motion, the tower has been safe due to net positive stabilizing moments acting on the tower.



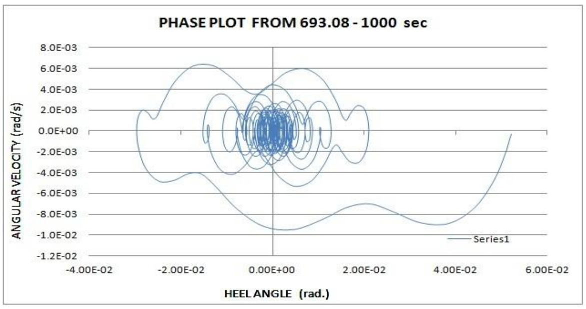
(Fig.6-Time history plot showing SHAT motion during entire duration of wave loading and EQ at 653.1 second)



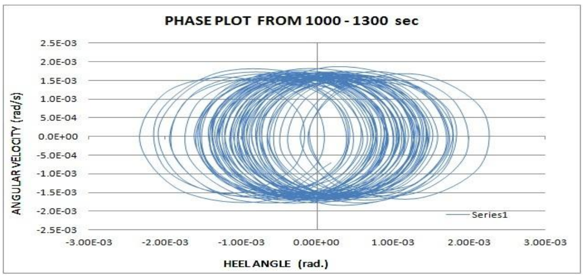
(Fig.7- Two dimensional Phase Plot for complete duration)



(Fig.8- Phase Plot during Northridge EQ loading phase of 39.98 seconds showing bifurcations in dynamically unstable motion)

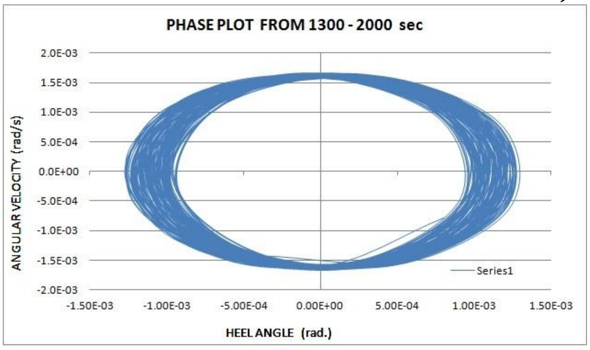


(Fig.9- Phase plot after EQ loading-still showing dynamically unstable motion with still existing bifurcations)



(Fig.10-Phase plot after EQ loading- showing trajectories moving towards stable limit cycle)

ISSN: 2277-3754
ISO 9001:2008 Certified
International Journal of Engineering and Innovative Technology (IJEIT)
Volume 4, Issue 2, August 2014



(Fig.11- Phase plot showing dynamically stable motion 650 seconds after removal of impact of Northridge EQ load)

VI. CONCLUSIONS

The observations in Table II – V were analyzed and following conclusions have been drawn:

A. Conclusive remarks on Seismic Response Analysis

1) The initial condition described by the instant of time of the steady state tower motion at which the earthquake strikes has significant effects on the tower response. Peak values differed upto 7-8% due to change in initial conditions i.e. when the earthquake is applied at axis or applied at crest of wave in regular sea. These values may further differ appreciably if earthquake occur at the time corresponding to trough or crest of the regular wave.
2) Even small duration earthquake gives big jolt to the structure leading to higher values of hinge angle rotation.
3) Larger size waves produce sufficient magnitude of hydrodynamic dampening so as to attenuate peaks in responses.
4) With the inclusion of the current alongwith regular sea wave, there is 2.58 - 2.82 % increase in Hinge angle response and also the Maximum value of Base shear reduces.
5) For different loadings, Base Axial Force values have common coefficient as 10^8. The changes in these values due to different combinations of loadings are very small.

B. Conclusive Remarks on Dynamic Stability Analysis

1) During the earthquake, the tower tends to vibrate at its own natural frequency while the steady state response again takes place in wave frequency when the earthquake is over.
2) The time required to achieve the steady state response after the duration of earthquake depends upon the sea environment at that time. High sea state dampens the seismic response quickly.
3) Although short duration intensive earthquake load gives a big jolt to the tower but it survives due to inherent restoring capacity. The responses due to short duration Earthquake die out quickly when the waves are also present. In the absence of earthquake and other environmental loads tower oscillations are checked by hydrodynamic dampening due to waves and due to tower oscillation and inherent tower buoyancy.
4) Subsequent to application of Wave / Earthquake loads, the time taken for Stabilization depends upon the nature / Size of the wave. The larger the size of wave, the smaller is the Stabilization period. For Small size wave (H-2.15, T-4.69s) with or without Earthquake, the time taken to achieve dynamic stability was 610 seconds. For middle sized wave (H-11.15, T-10.69s), the time taken for dynamic stability is 350 seconds. For another higher size wave (H-17.15, T-13.26s), the time taken for dynamic stability is 250 seconds. For Strong large size wave(H-30m, T-15s), the time taken for dynamic stability is 200 seconds.
5) When only Northridge Earthquake is applied in calm sea, the time taken for dynamic stability was 1000 sec. Waves / current provide hydrodynamic dampening to the structure, in the absence of dampening the structure takes more time to stabilize.
6) With the introduction of current in wave load with or without earthquake, the time taken for dynamic stability slightly reduces which shows that the current adds up to the hydrodynamic damping effects.
7) SHAT model subjected to small regular wave (H-4.8m, T-10.4s) and 4 varieties of Earthquake loadings (1994 Northridge-NWH360, 1979 Imperial Valley H-E11140, 1979 Imperial Valley H-E13140 & 1999 Duzce, Turkey -1062-N) showed that absolute Maximum values of responses arising out of impact of all the four types of Earthquakes are having similar magnitude(except for some minor variation). Northridge Earthquake is found to be severest of all four. The stabilization period after each earthquake is 640 sec., which once again emphasize the fact that stabilization period is a property of nature / size of wave. Nature of Earthquake, however governed the shape of Time History / Phase plot of responses.
8) In all the regular wave cases analysed during study, it was observed that during the initial period pertaining to onset of waves or period pertaining to Earthquake, the dynamic instability is visible in Phase plots. The motion is non-harmonic, a periodic and asymmetric. Bifurcations are easily visible in the phase plots confirming dynamic instability. With the passage of time, the hydrodynamic dampening effects reduce responses. The trajectories start moving towards the stable limit cycle and the motion gradually becomes harmonic, periodic and symmetric. No bifurcations are visible on the phase plots after longer duration loadings and the structure show dynamic stability.

ISSN: 2277-3754
ISO 9001:2008 Certified
International Journal of Engineering and Innovative Technology (IJEIT)
Volume 4, Issue 2, August 2014

REFERENCES

[1] Jain, A.K., and Datta, T.K.,"Non-linear behavior of articulated tower in random sea." *J. Eng. Ind.* 1991, *pp*.238-240.

[2] Lina, H., Youngang, T. and Cong, YI. "Seismic response analysis of articulated tower platform in the still water". China Offshore Platform, period 03, 2006.

[3] Islam, N. and Ahmad, S., "Earthquake Response of Articulated offshore tower." European Earthquake Engineering, 1, 2006, 48-58.

[4] Chandrasekaran and Gaurav, S., "Offshore Triangular TLP earthquake motion analysis under distinctly high sea waves". International Journal of Ships and Offshore Structures (Taylor & Francis), Vol. 3, No. 3, 2008, 173-184.

[5] Islam, N., Zaheer, M.M. and Ahmad, S."Double Hinged Articulated tower interaction with wind and waves." Journal of Wind Eng. Ind. Aerodyn.2009, 97(5-6), 287-297.

[6] Hasan, S. D., Islam, N. and Moin, K., "Multi-hinged Articulated Offshore Tower under Vertical Ground Excitation". ASCE Journal of Structural Engineering, Vol. 137, No. 4, 2011, pp. 469-480, DOI: 10.1061/(ASCE)ST.1943-541X.0000284.

[7] Banik, A.K.and Datta T.K., "Stochastic response and Stability of Single leg Articulated Tower", Proceeding of International Conference on Offshore Mechanics and Arctic Engineering (OMAE), June 8-13, 2003Cancun, Mexico, pp. 431-438.

[8] Banik, A.K. and Datta, T.K., "Stability Analysis of an Articulated Loading Platform in Regular Sea" Journal of Computational Nonlinear Dynamics, ASME, Vol. 3/011013-1, 2009, Issue doi:10.1115/1.2815332.

[9] Banik, A.K., "Dynamic Stability Analysis of Compliant Offshore Structures", IIT, Delhi, 2004.

[10] Friedmann, P., Hammond, C.E., and Woo, Tze-hsin, "Efficient Numerical Treatment of Periodic Systems with Application to Stability Problems." International Journal for Numerical Methods in Engineering, Vol.11, 1977, pp.1117-1136.

[11] Chua, L.O. and Ushida, A., "Algorithms for Comuting Almost Periodic Steady-State Response of Nonlinear Systems to Multiple Input Frequencies" IEEE Transactions on Circuits and Systems, Vol.CAS-28, 1981, pp.953-971.

[12] Burton, T.D., "Nonlinear Oscillator Limit Cycle Analysis using a Finite Transformation Approach." International Journal of Nonlinear Mechanics, Vol. 17, 1982, pp.7-19.

[13] Cai, G.Q., "Random Vibration of Nonlinear Systems under Non-White Excitations." Journal of Engineering Mechanics, ASCE, Vol.121 (5), 1995, pp.633-639.

[14] Lin, Y.K. and Cai, G.Q., Probabilistic Structural Dynamics" Advanced Theory and Mechanics, McGraw-Hill, New York, 1995.

[15] Hamdan, M.N. and Burton, T.D., "On the Steady State Response and Stability of Non-Linear Oscillators using Harmonic balance." Journal of Sound and Vibration, Vol.166 (2), 1993, pp.255-266.

[16] Rabindra, B., and Mallik, A.K., "Stability Analysis of Non-linearly Damped Duffing Oscillator," Journal of Sound and Vibration, Vol.171 (5), 1994, pp.708-716.

[17] Blair, K.B., Krousgrill, C.M., and Farris, T.N., "Harmonic Balance and Continuation Techniques in the Dynamic Analysis of the Duffing's Equation." Journal of Sound and Vibration, Vol.202 (5), 1997, pp.717-731.

[18] Yu, P., and Bi. Q., "Analysis of Nonlinear Dynamics and Bifurcations of a Double Pendulum." Journal of Sound and Vibration, Vol. 217(4), 1998, pp.691-736.

[19] Mallik, A.K. and Bhattacharjee, J.K., "Stability Problems in Applied Mechanics". Narosa Publishing House, 2005 Chapter2-3.

[20] Islam Saiful, A.B.M, "Nonlinear Dynamic Behavior of Fully Coupled SPAR Platform", PhD. Thesis, Deptt. Of Civil Engg., University of Malaya, Kuala Lumpur,2013.

[21] Chakrabarti S.K., "Stability Analysis of Interaction of an articulated tower with waves", Proceedings of Fluid interaction, Heikidiki, Greece. Vol.1, 2001, pp.281-292.

[22] Chakrabarti, S. And Cotter, D., "Motion Analysis of Articulated Tower", Journal of Waterway, Port, Coastal and Ocean Division, ASCE, Vol.105, 1979, pp.281-292.

[23] Jameel, M., & Ahmad, S., "Fatigue Reliability Assessment of Coupled Spar-Mooring System". Paper presented at the ASME 30th International Conference on Ocean, Offshore and Arctic Engineering (OMAE 2011-49687), Rotterdam, Netherlands, 2011.

[24] Seto, William W."Schaum's outline series, Theory and problems of Mechanical Vibrations" McGraw Hill, International Edition., 1989.