Zdynamic Analysis of Offshore Structure In SAP 2000

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Abstract- *This paper details the results obtained from static and dynamic analysis, considering both operational and ultimate limit state performance characteristics, designed at high basin. The existing platforms at high region are affected by continuous sea state and operational state loading. So, it becomes a necessity that the structures are design for the purposes should be a reliable one. The Static load and dynamic load analysis done in this study, using the SAP2000 v14 software helps to analyze and predict the performance of structure at, when Subjected to various load cases which will help in future to design it properly and accurately. In this paper various loads such as wave load, wind load both for airy and stokes law and earthquake load have been used to calculate the displacement and bending moment for design.*

Keywords- Offshore, Sea wave, Nonlinear Analysis, Finite Element Analysis, Wave-Structure Interaction, wave loading

I. INTRODUCTION

1.1 General:-

The total number of offshore platform in various bays, gulf and oceans of the world is increasing year by year, most of which are of fixed jacket-type platforms located in 30 m to 200 m depth for oil and gas exploration purposes. Fixed offshore platforms are subjected to different environmental loads during their lifetime. These loads are imposed on platforms through natural phenomena such as wind, current, wave, earthquake, snow and earth movement. Among various types of environmental loading, wave forces loading is dominated loads. According to API-RP2A 1997 (2.2) [1-3], environmental loads, with the exception of earthquake, should be combined in a manner consistent with the probability of their simultaneous occurrence during the loading condition being considered. In addition DNV 1980 (5.2.4) suggests that loads due to earthquake normally need not be considered to act simultaneously with other environmental loads. It is necessary to design an offshore structure such that it can respond to moderate environmental loads without damage and is capable of resisting severe environmental loads without seriously endangering the occupants. The standard design of the structure is carried out using the allowable stress method. However, it is important to clarify the effects on nonlinear responses for an offshore structure under the severe wave conditions. Offshore structures may be analyzed using static or dynamic analysis methods. Static analysis methods are sufficient for structures, which are rigid enough to neglect the dynamic forces associated with the motion under the time-dependent environmental loadings. On the other hand, structures which are flexible due to their particular form and which are to be used in deep sea must be checked for dynamic loads. Dynamic analysis is particularly important for waves of moderate heights as they make the greatest contribution to fatigue damage and reliability of offshore structures. The dynamic response evaluation due to wave forces has significant roles on the reliable design of the offshore structure. In the design and analysis of fixed offshore structures many nonlinear physical quantities and mechanisms exist that are difficult to quantify and interpret in relation to hydrodynamic loading. The calculation of the wave loads on vertical tubular members is always of major concern to engineers, especially recently when such studies are motivated by the need to build solid offshore structures in connection with oil and natural gas productions. The effects of various wave patterns on offshore structures have been investigated by numerous researchers in the past [6 - 17].This research summarizes the nonlinear dynamic analysis of a 3-D model of a typical Jacket-Type platform, which is installed in Suez gulf, Red sea, 1988 and presents the numerical investigation on dynamic behavior of an offshore structure under wave loads. Wave loading is applied to a full jacket structure by Stokes 5th order wave methods with gravity loads also present. The analysis considers various nonlinearities produced due to change in the nonlinear hydrodynamic drag force. The wave forces on the elements of the offshore structure are calculated using Airy's wave theory and Morison's equation. Numerical results are presented for various combinations of typical sea states. Natural periods and mode shapes of the system are calculated. The results of these investigations highlight the importance of accurately simulating nonlinear effects in fixed offshore structures from the point of view of safe design and operation of such systems. When designing offshore platforms, there are a number of accidental loads that can occur. These include ship impacts, dropped objects, fire and explosions. Among these, explosions, as Illustrated on Figure

1-1, are particularly challenging, because there are many complexities Involved in the design process.

Fig 1: Gas explosion on an offshore platform [W1]

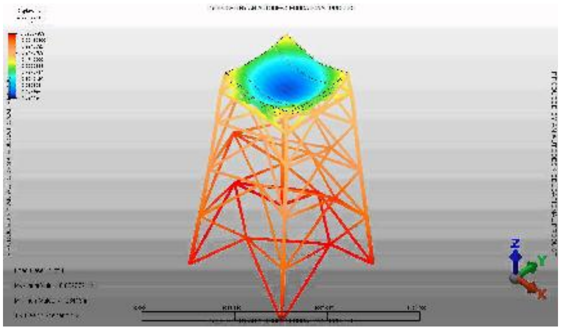


Fig 2: Model of a typical Offshore Module

1.3 PRIMARY EFFECTS

The primary effects focused on and investigated in this thesis, beside standard linear elasticity, are:

- Dynamic load and response
- Geometric nonlinearity
- Plasticity
- Strain rate effects

W.S. Zhu et al [1]: Based on an opening complex for a hydropower station in China, an equation considering four basic factors was fitted for prediction of displacement at the key point on the high sidewalls of the powerhouse, on the basis of a large number of numerical simulations. The basic factors include rock deformation modulus, overburden depth of caverns, height of the powerhouse and the lateral pressure coefficient of the initial stress.

Yang Yang et al [2]: Based on the characteristics of the dynamic interaction between an underground powerhouse concrete structure and its surrounding rock in a hydropower plant, an algorithm of dynamic contact force was proposed. This algorithm enables the simulation of three states of contact surface under dynamic loads, namely, cohesive contact, sliding contact, and separation. It is suitable for the numerical analysis of the dynamic response of the large and complex contact system consisting of underground

Youssef M.A. Hashash et al [6]: Underground facilities are an integral part of the infrastructure of modern society and are used for a wide range of applications, including subways and railways, highways, material storage, and sewage and water transport. Underground facilities built in areas subject to earthquake activity must withstand both seismic and static loading. Historically, underground facilities have experienced a lower rate of damage than surface structures..

A S Patil et al [7]: In the present paper study of nonlinear dynamic analysis of Ten storied RCC building considering different seismic intensities is carried out and seismic responses of such building are studied. The building under consideration is modeled with the help of SAP2000-15 software. Five different time histories have been used considering seismic intensities V, VI, VII, VIII, IX and X on Modified Mercalli's Intensity scale (MMI) for establishment of relationship between seismic intensities and seismic responses.

Dr. Martin Wieland et al [8]: Hydropower plants with underground powerhouse contain different types of underground structures namely: diversion tunnels, headrace and tailrace tunnels, access tunnels, caverns grouting and inspection galleries etc. As earthquake ground shaking affects all structures above and below ground at the same time and since some of them must remain operable after the strongest earthquake ground motion, they have to be designed and checked for different types of design earthquakes.

II. METHODOLOGY

In this paper offshore structures are studied in SAP 2000, the method of analysis is time history analysis. Time history analysis is nothing but calculating structure response of model subjected to specified ground motion. In this paper El-centro data is used for time history analysis it has total time period of 21 sec

III. PERFORMANCE ANALYSIS

The studied platform is a fixed Jacket-Type platform currently installed in the Suez gulf, Red sea, 1988 shown in Figure 3, The offshore structure is a four legs jacket platform, consists of a steel tubular-space frame. There are diagonal brace members in both vertical and horizontal planes in the units to enhance the structural stiffness. The Platform was originally designed as a 4-pile platform installed in 110 feet (110' =33.5 m) water depth.

- The Top side structure consists of Helideck 50'x50' at ELevation, EL. (+54') & Production deck50'x50' at EL. (+26'); Top of jacket at EL (+12.5').
- The Jacket consists of 4 legs with 33 inch Outer Diameter (33" O.D.) & 1 inch Wall Thickness(1"W.T.) between EL. (+10') and EL. (-23') and (33" O.D. x 0.5" W.T.) between EL. (-23') andEL. (-110').
- In the splash zone area that is assumed to extend from EL. (-6') to EL. (+6') LAT. (LowestAstronomical Tide).
- The jacket legs are horizontally braced with tubular members (8.625" O.D. x 0.322" W.T.) atelevations (+10'); (10.75" O.D. x 0.365" W.T.) at elevations (-23'); (12.75" O.D. x 0.375" W.T.) at elevations (-62’) and (14" O.D. X 0.375" W.T.) at elevations (-110’).
- In the vertical direction, the jacket is X-braced with tubular members (12.75" O.D. x 0.844" W.T.) from EL. (+10') to EL. (-23’) and (12.75” O.D. x 0.375” W.T.) from EL. (-23’) to EL. (-110’). The platform is supported by 4 piles (30" O.D. x 1.25" W.T.).)

Description of loading:

Density of various materials considered for design,
Concrete – 25kN/m3
Insulation – 1kN/m3
Structural steel – 78.5kN/m3
Live load – 5kN/m3

Wind load:

The following wind parameters are followed in accessing the wind loads on the structure
Basic wind speed – 55m/s
Terrain category -2
Class of structure – c
Risk coefficient k1 – 1
Topography factor k3– 1
K2 factor taken from Draft Code CED 38(7892):2013 (third revision of IS 4998(part 1):1992)

Earthquake force data:

Earthquake load for the chimney has been calculated as per IS 1893(par 4) : 2005
Zone factor – 0.16
Seismic zone – III
Importance factor (I) – 1.5
Reduction factor (R) – 3

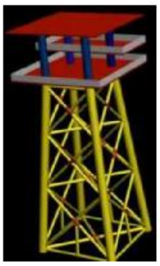
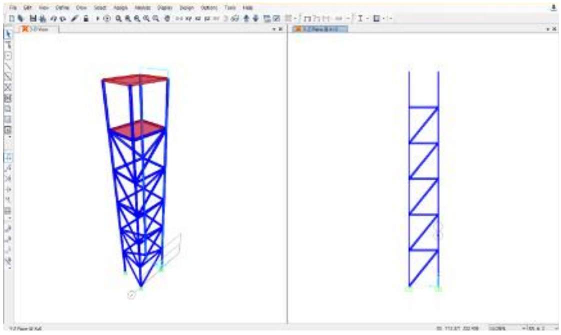
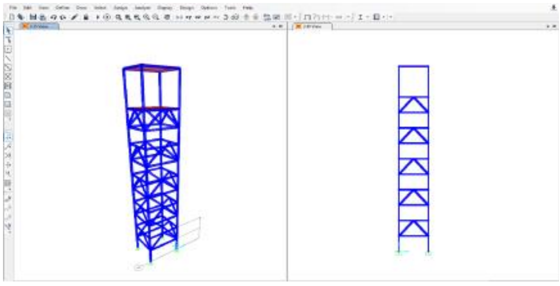



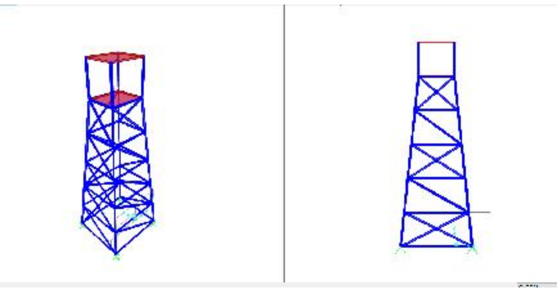
Fig 3: sketch map of platform model

Idealization of above problem statement is modeled in finite element analysis tool SAP 2000.Following models are prepared for comparative analysis of offshore steel structures

MODEL NO. 1	OFFSHORE PLATFOREM WITH SINGLE BRACING 0 DEGREE
MODEL NO. 2	OFFSHORE PLATFOREM WITH DOUBLE BRACING 0 DEGREE
MODEL NO. 3	OFFSHORE PLATFOREM WITH KNEE BRACING 0 DEGREE
MODEL NO. 4	OFFSHORE PLATFOREM WITH SINGLE BRACING 20 DEGREE
MODEL NO. 5	OFFSHORE PLATFOREM WITH DOUBLE BRACING 20 DEGREE
MODEL NO. 6	OFFSHORE PLATFOREM WITH KNEE BRACING 20 DEGREE
MODEL NO. 7	OFFSHORE PLATFOREM WITH SINGLE BRACING 30 DEGREE
MODEL NO. 8	OFFSHORE PLATFOREM WITH DOUBLE BRACING 30 DEGREE
MODEL NO. 9	OFFSHORE PLATFOREM WITH KNEE BRACING 30 DEGREE







IV. RESULT AND DISCUSSION

TIME PERIOD ZERO DEGREE:

Table 1: time period 0 degree

MODE SHAPE	SINGLE BRACING	DOUBLE BRACING	KNEE BRACING
0	0	0	0
1	4.24472	4.209284	5.963801
2	1.69657	1.479957	2.7825
3	1.470164	1.442051	2.436364
4	1.029211	0.932801	2.2706
5	0.528184	0.503503	0.947161
6	0.395103	0.260603	0.637286
7	0.206469	0.174174	0.560828
8	0.149784	0.122324	0.388552
9	0.131487	0.116064	0.301887
10	0.102495	0.09928	0.253771
11	0.087279	0.084492	0.24585
12	0.085654	0.084471	0.227683

In this graph maximum time period is in knee bracing is 6. The difference between knee and double bracing is 20%.

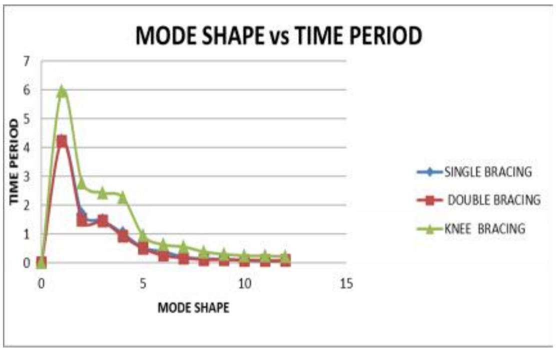


Fig 4 mode shear vs. time period

In this graph maximum base shear is in single bracing is 3.00e+03. The difference between double and single bracing is 5%.

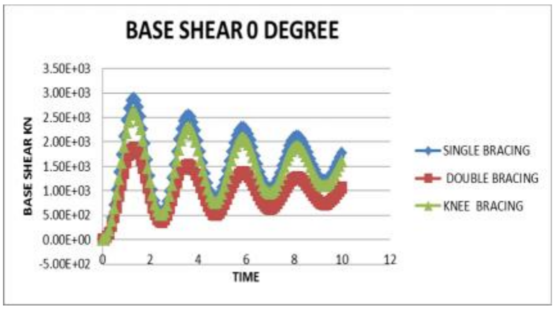


Fig 5 base shear vs time period

NATURAL FREQUENCY 0 DEGREE:

Table 2Natural Frequency 0 Degree

MODE SHAPE	SINGLE BRACING	DOUBLE BRACING	KNEE BRACING
0	0	0	0
1	0.23558	0.23757	0.16767
2	0.58942	0.67569	0.359388
3	0.68019	0.69345	0.410447
4	0.97161	1.0720	0.4404123
5	1.89328	1.986084	1.055786917
6	2.530987	3.837254	1.569155314
7	4.843332	5.741374	1.783079387
8	6.676275	8.175040	2.573659139
9	7.605296	8.615926	3.312497818
10	9.756563	10.07250	3.940567588
11	11.45757	11.83544	4.067520191
12	11.67481	11.83834	4.392075967

In this graph maximum natural frequency is in double bracing is 12. The difference between single and double bracing is 1%.

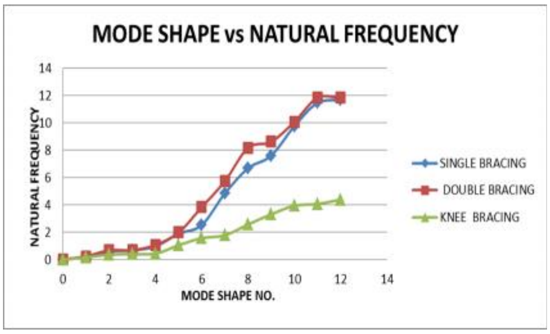


Fig 6 mode shear vs. natural frequency

DISPLACEMENT 0 DEGREE :

Table 3Displacement 0 Degree

DEFORMATION-Y 0 DEGREE mm		
SINGLE BRACING	DOUBLE BRACING	KNEE BRACING
0.544569	0.327	0.4279

In this graph maximum deformation-y 0 degree is in single bracing is 0.52. The difference between single and knee bracing is 25%.

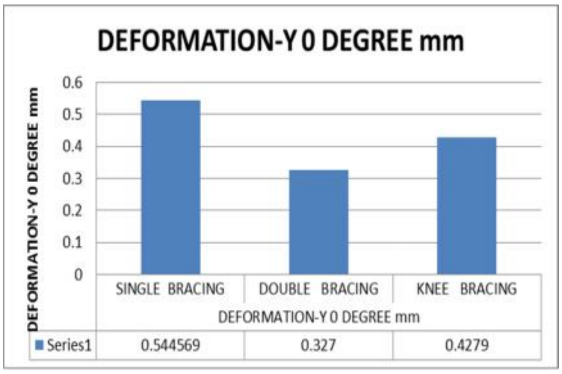


Fig 7Displacement 0 Degree

V. CONCLUSIONS

After finite element analysis in SAP models time history analysis is done for single bracings, double bracings and knee bracings following conclusions can be made for 0 degree, 20 degree and 30 degree

- Deformation in y direction is 25% less in double bracing and 15 % less in knee bracing. But deformation in X direction is observed more in knee bracings. In addition to this. For base shear it is observed that base shear is 15% more in single bracings than cross bracings and knee bracings
- Natural frequency is observed more in knee bracings which indicates less time period in knee bracings
- For angle variation 0 degree ,20degree and 30 degree the deformation observed less in 30 degree as compared to other so it can be included that steep slope increases the stability of structure

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