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STRUCTURAL RESPONSE OF A SELF-SUPPORTING TOWER UNDER SEISMIC LOADS

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الملخص العربي :

تعتبر أبراج الاتصالات من المنشآت الضرورية والهامة أثناء وعقب الكوارث الزلزالية لذلك الحفاظ علي سلامتها من الأولويات القصوي. يقدم البحث دراسة إنشائية ديناميكية لأحد الأبراج الجمالونية للاتصالات بارتفاع ٨٠ متر الموجودة من ضمن شبكات الاتصالات بساحل البحر الاحمر وهي منطقة معلومة بنشاطها الزلزالي عبر العقود الماضية. تم استخدام نموذج رياضي للبرج تم ضبطه ليحاكي القياسات الديناميكة علي البرج في الموقع . التحليل الانشائي للبرج تحت تأثير الأحمال الزلزالية تم بعدة طرق منها الحمل الاستاتيكي المكافئ وطريقة التجاوب الطيفي وطريقة التحليل الزمني الديناميكي تحت تأثير زلزال العقبة ١٩٥٥ وكل الطرق المستخدمة طبقا للمواصفة ANSI/TIA-222-G-2005 والمختصة بأبراج الاتصالات بما يتوافق مع الكود المصري للأحمال فيما يخص الخواص الزلزالية لمنطقة البحر الأحمر التي يقع فيها البرج تحت الدراسة.

Abstract

Telecommunication towers are very vulnerable structures in that way, and essential for communication and post-disaster networks and their preservation in the case of a severe earthquake is crucial. A number of Steel lattice towers currently constitute a part of large telecommunication network along the Egyptian red sea coast. A zone known for its seismic activity for centuries, thus the assessment of their seismic performance is highly demanded. This paper discusses a full dynamic investigation on the seismic performance of an existing 80.0 m tall self-supporting telecommunication tower located in Egypt. The structural response under seismic loads was thoroughly investigated using the equivalent method, response spectrum and time history dynamic analyses based on ANSI/TIA-222-G-2005 provisions. The simulation of the full-scale tower using finite element modeling was validated by experimental measurements of the vibration response of the tower on-site (previous research work of authors). The experimental modal analysis gives higher values of natural frequency of the tower than the standard formula, which is directly influential on the structural dynamic behavior. The resulting numerical predictions are in close agreement with the captured dynamic properties. The reliably simulated 3D finite element model was used for detailed dynamic study under seismic loads obtained from 1955 AQABA earthquake acceleration records for the time history analysis. Significant results were obtained as the analyses are established on validation via experimental testing results. The results based on this analysis are presented; useful conclusions are introduced regarding the performance of the structural members of the steel lattice tower under investigation.

Keywords: self-supporting tower, finite element, modal analysis, seismic response, equivalent static method, response spectrum, time history analysis.

1. Introduction

Steel lattice towers have become vital part of current infrastructures particularly in wireless telecommunication networks. Elevated antenna for radio and television broadcasting, telecommunication systems are essentially supported and sustained by such tall towers. Data transmitted by means of telecommunication towers are critical in times of catastrophic situations. For that reason, their full serviceability is of main concern in the case of a disaster. Telecommunication towers are categorized as slender-tall structures that are vulnerable to excessive vibration from dynamic environments generated by wind, earthquake, blast and explosions. Such Vibrations cover an ample of spectrum of frequencies, which affect the towers in different way, ranging from serviceability problems to fatigue or collapse [1].

McClure, [2] quotes a recent survey of earthquake performance of communication structures which summarizes documented reports of only 16 instances of structural damage related to seven important earthquakes in the past 50 years. It was concluded from this survey that broadcast structures and large building-supported microwave towers are the most vulnerable types [3]. It is established that owing to their indispensable role, the protection of these telecommunication structures during a natural disaster such as an earthquake is of principal priority and that's why their dynamic analysis should be accurately evaluated [4]. Dynamic analysis of telecommunication towers have been mostly counting on the analytical methods in regard to the complexity of applying full-scale testing. Konno and Kimura [5] presented one of the first studies on the effects of earthquake loads on lattice telecommunication towers at top of buildings. Simulation of a stick model of the tower using lumped masses and a viscous damping ratio of 1% was used in their studies. It was observed that in some of the members, the forces due to earthquake were greater than those due to wind. Mikus [6] investigated the seismic response of six 3-legged self-supporting telecommunication towers with different heights. The selected towers were numerically simulated as bare towers without considering the antennas and other accessories attached to them. Three earthquake records were considered as input in the analysis. It was concluded that modal superposition with the lowest four modes of vibration would ensure sufficient accuracy. Lefort [7] investigated the effects of wind and earthquake loads on the self-supporting antenna towers and it is reported that for towers, seismically induced member forces may exceed forces obtained from service and wind load calculations. Amiri and Booston [8] studied the dynamic response of, self-supporting steel telecommunication towers with different heights. Wind and seismic loading were considered. The study implies the necessity of considering earthquake loads in tower analysis and design. Some analytical investigations of seismic response of self-supporting towers suggested simplified modelling and analysis techniques, where the tower was modelled as a simple cantilever. Equivalent static load methods were also introduced to model the seismic response of the self-supporting towers [9; 10]. Khedr and McClure [11] proposed a new approximate static analysis method, however performing a detailed seismic analysis is suggested for such types of structures when most of the leg members and diagonal members are controlled by seismic loading.

Lattice structures are characterized by significant stiffness and low weight, and they are often used as telecommunication towers [12,13]. Telecommunication towers are characterized by a considerable slenderness ratio, which is why lattice structures have to be checked for sensitivity to dynamic seismic loads. The seismic sensitivity of communication structures is influenced by the coincidence between its dominant natural frequencies and the frequency content of the excitation, in this case the ground motion. Past earthquake records have typical dominant frequencies in the range of 0.1 to 10 Hz, with a concentration in the 0.3 to 3 Hz for horizontal motion while vertical motion involves the higher frequency band. The first step in the assessment of structure sensitivity to earthquakes is thus the evaluation of its dominant natural frequencies. Detailed dynamic analysis should be preceded by a frequency analysis where the dynamic characteristics are fully determined [2]. Despite of all the numerical studies which have been done with care and expert knowledge, little validation with physical tests or measurements has been reported to evaluate the level of accuracy of these computational studies. Hence the degree of uncertainty of these modeling predictions has not been determined up to now even in controlled laboratory conditions [14].

In this paper, the seismic analysis was applied using an updated finite element model of the 4-legged angular self-supporting telecommunication tower. The finite element model was validated based on field ambient vibration testing [15]. A detailed 3D finite element model was built in ANSYS to simulate the measured dynamic properties in the range 0-25 Hz [15]. Detailed dynamic analysis was conducted to investigate the structural response of the tower under seismic loads based on the ANSI/TIA-222-G-2005 provisions [16]. The equivalent static method, response spectrum and time history methods were adopted to perform the structural assessment. 1955 Aqaba earthquake was selected to perform the detailed time history analysis. Study provides a thorough investigation of the dynamic behavior of existing self–supporting towers which its numerical model is successfully vibration based validated. Accordingly making it one of the few studies where numerical model can be reliably utilized for detailed seismic response analyses and structural assessment. The overview of the results revealed that calculation of the fundamental natural frequency according to the formula in ANSI/TIA-222-G-2005 is lower than the measured properties from field dynamic testing. The structural response of the tower under seismic loading is illustrated in terms of resulting maximum stresses, support reactions and top lateral displacements.

2 .The self-supporting tower

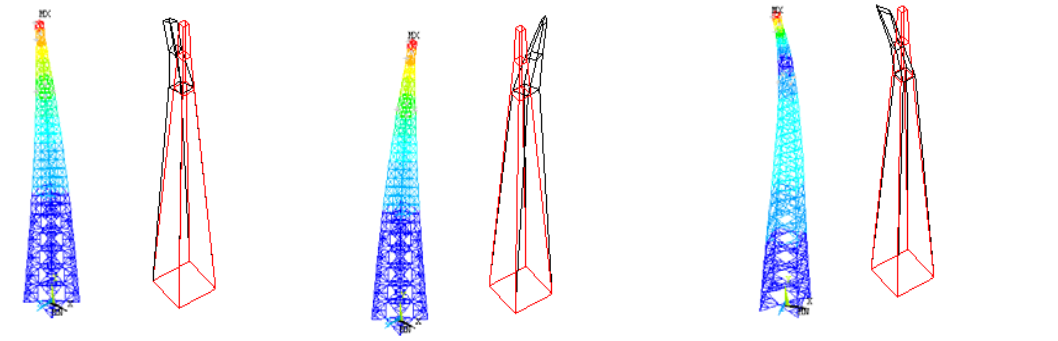
The tower is 80.55 meter, 4-sided, self-supporting tower, and 8.85 m width at base and 1.35 m width at top (Fig.1). The leg sections are angle sections ranges from L 200 x20 at the lower sections to L90X9 at the top. Diagonals are angle sections ranges from L 100x10 to L 50x5. For this tower, there are 8 separate sections. The sections are labeled T1 to T8 with T1 at the bottom being 12.50 m and T8 at the top 8.05 m tall and the remaining sections being only 10 m. Four DB 224 antennas were placed on the tower at each leg at height 64 m. Three other antennas, one 90 cm diameter high performance dish antenna is attached at height 73.75 m, other two 30 cm microwave dish antenna were installed at 70.30 m and 80.1 m.

3. Validated finite element model using field ambient vibration testing

The field ambient vibration testing techniques were used to identify the modal properties of the full-scale tower in the range 0-25 Hz [15]. Highly sensitive accelerometers were used to measure the ambient response of the tower. The dynamic response of the tower was investigated through measuring 12 DOF's signals at 80, 70 m, 60m levels. The measurements were taken for the upper 30 meters of the tower for accessibility limitations available at site. The reference channels were chosen in the main orthogonal directions at the top level of the tower where all the structural modes are present in the measured spectra. Stochastic subspace identification technique was selected for the modal identification analysis. It is proven to be powerful in identifying the modal properties in an efficient way for such slender structures usually characterized by very closely spaced modes [15]. Detailed three-dimensional full-scale numerical simulation using the finite element method in ANSYS has been carried out. The tower is made of steel with Young’s modulus of E = 2.0 * 1011 Pa and Poisson’s ratio of ν = 0.3, the unit weight of the steel material is 7850 kg/m3. The analyzed structure composed of rods which are equal-sided angle sections joined with bolts. Beam element was selected for main tower legs and link element was chosen to represent the diagonals and horizontals. Components such as antenna and ancillary such as ladders, platforms, feeder cables were represented as point masses along the tower height. Consideration of masses of all ancillaries is important since mass of such items could contribute significantly for seismic response of a tower under an earthquake, as the weight of ancillaries including antennas comprises considerable portion of overall self-weight of an actual tower [17]. The initial finite element model in Ansys was successfully updated based on the vibration testing results. Good agreement was obtained in comparison to experimental results, more details about the field testing and the model updating of the tower under study can be found in [15]. Fig.2 shows the final results of both the experimental and numerical analyses

Self –Supporting tower **Mounted antenna** **Tower foundation**

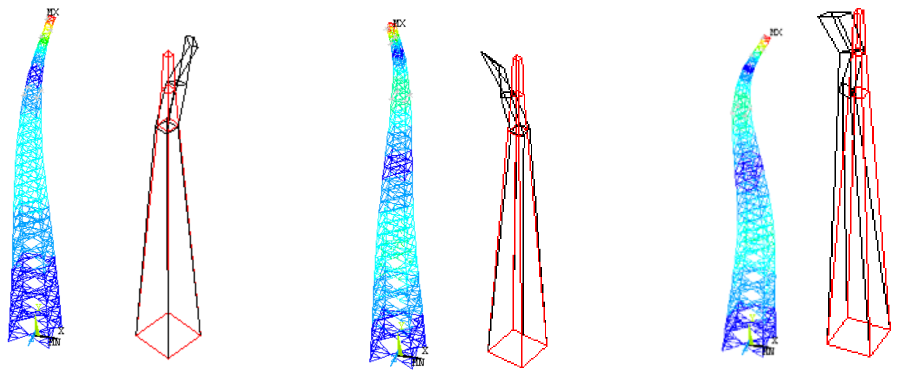
Fig 1: Self –supporting tower , foundation and mounted antenna



First bending X
Exp: Freq=1.248Hz d=1.883%
Num: Freq= 1.26 Hz

First bending Y
Exp: Freq=1.254Hz d=1.191 %
Num: Freq= 1.26 Hz

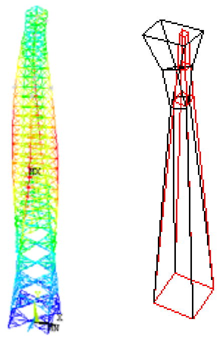
Second bending X
Exp: Freq =3.323 Hz d=1.182%
Num: Freq= 3.45 Hz



Second bending Y
Exp: Freq=3.34 Hz d=1.465 %
Num: Freq= 3.48 Hz

Third bending X
Exp: Freq=7.136Hz d=1.326 %
Num: Freq= 7.339 Hz

Third bending Y
Exp: Freq =7.737 Hz d=1.864%
Num: Freq= 7.63 Hz



First torsion Exp:
req=9.839 Hz d=2.03 %
Num: Freq= 9.655 Hz

Fig 2: Experimental and simulated numerical modal analysis results of the tower

4. Seismic loads

For the estimation of seismic loads on tower, four methods are provided in the ANSI/TIA-222-G-2005 [16] as follows:

1. Equivalent lateral force, method 1
2. Equivalent Modal analysis, method 2
3. Modal analysis, method 3
4. Time history analysis, method 4

The first two methods are direct simplified equivalent static methods and the other two are more detailed dynamic analysis procedures.

4.1 Equivalent static method

Equivalent static methods are the simplest analytical procedures that can be applied for seismic response analysis. For the selection of the suitable equivalent static method for the analysis as given in ANSI/TIA-222-G -2005[16], criteria have been illustrated in the code and accordingly for the 80.55 m tower, method 2 was implied for the analysis as follows:

$F_{sz}= S_{az}\ W_z\ I/R$

Where;

F_{sz} = Lateral seismic force at level z under consideration

S_{az} = Acceleration coefficient at height z

$= \{a\ (S_A)^2 \ +b\ (S_{DS})\ ^2\}/\{\ (S_A)^2 \ +c(S_{DS})\ ^2\}^{0.5}$

a,b,c= Acceleration coefficients

S_A = S_{D1} f_1 when f_1 <= S_{DS} /S_{D1}, otherwise S_A = S_{DS}

f_1 = Fundamental frequency of the tower (calculated according to section 2.7.11)

S_{DS} = Design spectral response acceleration at short period

S_{D1} = Design spectral response acceleration at period of 1.0 second

W_z = Portion of total gravity load assigned to level under consideration

I = Importance factor

R = Response modification coefficient equal to 3.0 for lattice self-supporting towers

For the computation of seismic shear, maximum considered earthquake spectral response acceleration at short period (SDS) and maximum considered earthquake spectral response acceleration at 1.0 second (SD1) are required. These are site specific acceleration coefficients and these values are currently given for USA only in ANSI/TIA-222-G-2005 [16]. Recommended seismic acceleration parameters are locally available, since code of practice for Egyptian design loads [18] is available in Egypt. The recommended SDS and SD1 values for Marsa alam (Fig.3.) zone are 0.27g and 0.068g respectively.

The calculations counting on the provided formula for this method revealed that the fundamental frequency is 1.15 Hz with -8% difference from the corresponding measured one at 1.25 Hz [15]. It is important to note that the computed value was used in this part of the analysis.

4.2. Response spectrum analysis

Procedures for Developing Seismic Response Spectra for dynamic structural analysis may be developed following procedures outlined in the Egyptian loads code (2012) [18]. The simplified Egyptian spectrum (acceleration, m/sec2, versus period, seconds) is defined by a linearly increasing portion up to control period TB, followed by flat response up to control period TC, followed by a decaying response to larger periods with intermediate value at TD as shown in Fig.4. Design response spectrums have to be developed by considering local seismological parameters and structural characteristics of the tower. Equations given to develop response spectrum are as follows;

For $0 \leq T \leq TB$: $Sd(T) = ag\,\gamma 1\, S\,[(2/3)+(T/TB)((2.5/R)-(2/3))]$

$TB \leq T \leq TC$: $Sd(T) = (2.5\, ag\,\gamma 1\, S\,\eta)/R$

$TC \leq T \leq TD$: $Sd(T) = (2.5\, ag\,\gamma 1\, S\,\eta\,[TC / T])/R \geq 0.2\, ag\,\gamma 1$

$TD \leq T \leq 4.0$ sec : $Sd(T) = (2.5\, ag\,\gamma 1\, S\,\eta\,[TC\; TD / T2])/R \geq 0.2\, ag\,\gamma 1$

Where;

Sd (T) = Design spectral response acceleration at period T

T = Period corresponding to the fundamental natural frequency of the mode under consideration

ag = Design base acceleration for standard return period

TB , TC , TD = Control periods specifying the response spectrum curve

$\gamma 1$ =Importance factor of the structure

η =Corrective Damping value for the horizontal response spectrum spectra

S = Soil coefficient

R = Response modification coefficient equal to 3.0 for lattice self-supporting towers

Selection of the seismic coefficients is based on the Egyptian code seismic zone for the site under consideration and the near-surface geotechnical (soil/rock) properties. There are currently five seismic zones specified in the code. Marsa alam zone (Fig.3) falls with the seismic zone 3 with design ground acceleration ag = 0.15. Soil types are defined based on measured or estimated shear wave velocity (Vs) standard penetration test blow count (N), or undrained shear strength values. Soil type C is defined as dense granular soil, very stiff cohesive soil. Accordingly the seismic coefficients were specified as follows:

Subsoil Class	S	T_B	T_C	T_D
C	1.50	0.1	0.25	1.2

$\gamma 1$ is taken as 1.40 represents the importance of structure that is designated as essential facilities to work post-earthquake events to preserve safety and emergency communication. η is taken as 1.05 as assigned for steel bolted structures. For the above formula (sec 4.2) response spectrum curve was developed as shown in Fig. 4.

Fig.3: Location map of Marsa Alam city

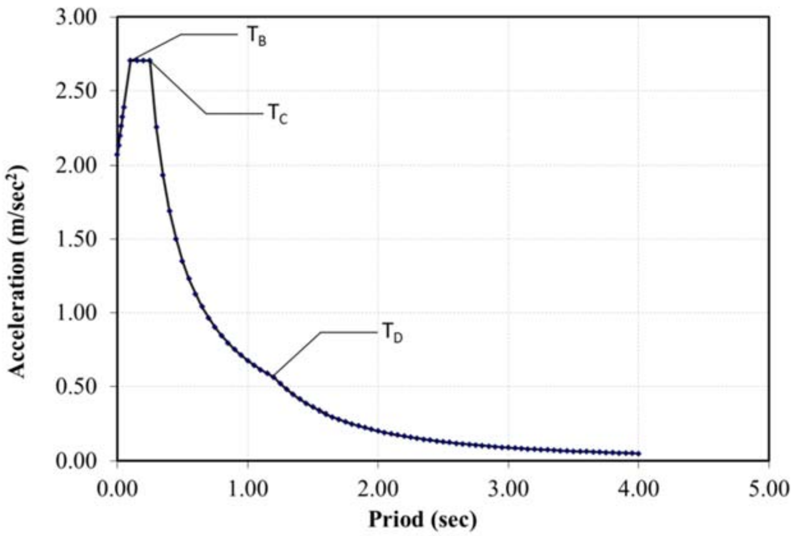


Fig.4: Developed response spectrum according to the Egyptian loads code 2012

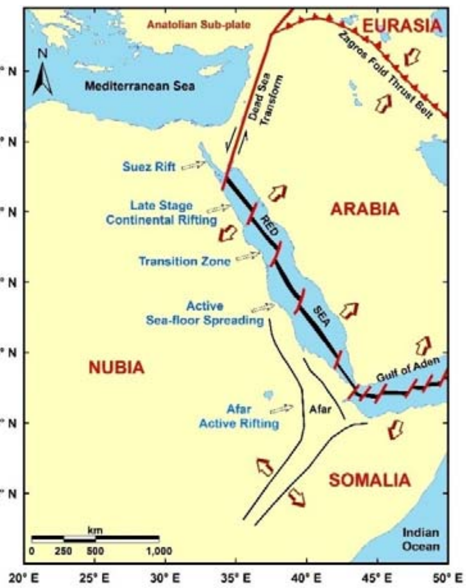
To analyze the tower in ANSYS, at first a validated modal analysis was performed. The calculated natural modes of vibration were correlated with their measured counterparts [16]. Next, the tower undergoes a spectral analysis under the standard design spectrum in the main horizontal direction X. As given in ANSI/TIA- 222-G [16], more than 85% modal mass participation was ensured in the spectrum analysis by considering appropriate number of modes. Accordingly, 45 modes were considered for the tower, the damping ratio used in all modes is equal to 3 percent as suggested by IASS [19].

Table 1: Response spectrum analysis data in ANSYS

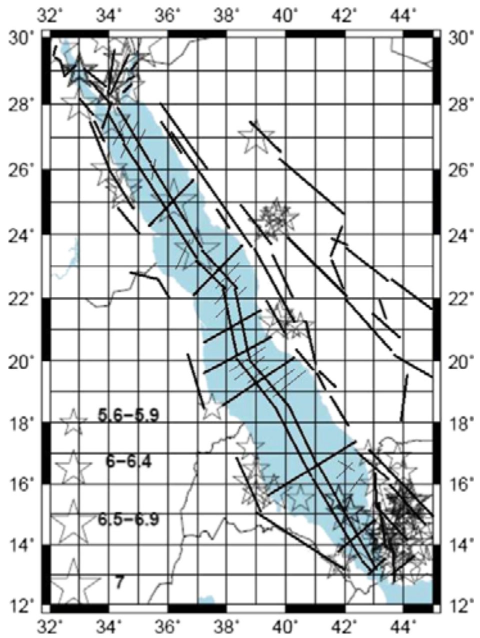
Mode No.	Freq. (Hz)	Spectrum Value (m/sec2)	Participation Factor	Mode coefficient	M.C ratio	Effective mass	Mass Fraction
2	1.276	0.86496	97.7	1.315	1	9546.05	0.355096
6	4.042	2.7039	-76.12	-0.3191	0.242706	5793.55	0.570599
7	4.043	2.7039	16.65	6.98E-02	0.05307	277.153	0.580908
10	7.339	2.7039	7.886	1.00E-02	0.007628	62.1828	0.583221
11	7.63	2.7039	-2.549	-3.00E-03	0.002281	6.49741	0.583463
12	7.65	2.7039	70.79	8.28E-02	0.063011	5010.74	0.769847
25	10.91	2.7039	6.494	3.74E-03	0.002841	42.1682	0.771427
30	11.2	2.7039	4.795	2.62E-03	0.00199	22.9908	0.772428
31	11.34	2.7039	-0.3889	-2.07E-04	0.000157	0.151237	0.772434
32	11.45	2.7039	13.8	7.21E-03	0.00548	190.378	0.779516
33	11.49	2.7039	-3.834	-1.99E-03	0.001514	14.6961	0.780062
34	11.52	2.7039	38.14	1.97E-02	0.014984	1454.78	0.834175
37	11.68	2.7039	21.13	1.06E-02	0.008074	446.687	0.85081
38	11.73	2.7039	1.841	9.16E-04	0.000697	3.38992	0.850936
40	11.93	2.7039	1.659	7.99E-04	0.000608	2.75101	0.851039
41	11.93	2.7039	27.67	1.33E-02	0.010118	765.354	0.879508
43	11.94	2.7039	-7.759	-3.73E-03	0.002834	60.1972	0.881747
45	12.27	2.7039	1.609	7.32E-04	0.000557	2.58922	0.881843

4.3.1 Seismicity of red sea region

The self–supporting tower exists by the red sea region in Marsa Alam city Fig. 3. The Red Sea Rift is a spreading center between two tectonic plates, the African Plate and the Arabian Plate Fig.5a. It extends from the Dead Sea Transform fault system, and ends at an intersection with the Aden Ridge and the East African Rift, forming the Afar Triple Junction in the Afar Depression of the Horn of Africa [20]. Search in a number of seismological sources revealed that in the period 1913–86 some 135 earthquakes (3 ⩽ M ⩽ 6.9) occurred in the Red Sea and western Arabia between latitudes 14° and 27.2° N. In the same period, 49 earthquakes (M ⩽ 6.9) and 247 earthquakes (M ⩽ 4.9) are reported to have occurred in the Gulfs of Suez and Aqaba respectively. The seismicity data indicate that both northern and southern parts of the Red Sea are seismically active .The central part of the Red Sea appears to be of relatively low activity, particularly the area bound by latitudes 21 and 23° where its northern half has no historic nor instrumental epicentres. Nevertheless, the seismicity of the different parts of the Red Sea appears to have varied and fluctuated with time ass shown in Fig.5b. [21; 22]. The Aqaba 1955 earthquake was chosen as an input to perform the time history analysis its record is shown in Fig.6.



a. Tectonic framework of the Red Sea region (redrawn after Ghebreab [20])



b. Epicentral distribution of historical earthquakes of red sea region (Redrawn after Z. H. El-Isa and A. Al Shanti [21])

Fig. 5: seismicity of the red sea region where the self-supporting tower is located in Marsa Alam City

4.3.2 Time history analysis

Linear response history analysis, known as time history analysis, is a numerical technique in which the response of a structural model to a specific earthquake ground motion accelerogram is determined through a process of a numerical integration of the eqs. of motion. The main advantages of the time history analysis is that it provides a time dependent history of the response of the structure to a specific ground motion. On the other hand this method provides information about the stress and deformation state of the structure throughout the period of response [23].

The basic equation of motion solved by a time history analysis is

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{F(t)\}$$

Where:

[M] = mass matrix
[C] = damping matrix
[K] = stiffness matrix
$\{\ddot{u}\}$= nodal acceleration vector
$\{\dot{u}\}$= nodal velocity vector
{u} = nodal displacement vector
{F(t)} = load vector

At any given time, t, these equations can be thought of as a set of "static" equilibrium equations that also take into account inertia forces ([M]$\{\ddot{u}\}$) and damping forces ([C]$\{\dot{u}\}$). The ANSYS program [24] uses the New mark time integration method to solve these equations at discrete time points. The time increment between successive time points is called the integration time step.

Time-history response of the tower model under Aqaba earthquake record is simulated. The strong ground motion record is input along the tower horizontal direction. The assumption of linear dynamic behavior, largely verified experimentally for self-supporting lattice towers, presumes the existence of normal modes of vibration [1]. Thus a linear dynamic approach has been employed for tower analysis. For this purpose, time history acceleration record is translated to ANSYS V11 [24] format. Using the first fifty modes of vibration, the modal mass participation of the tower in structural modes reaches to more than 90 percent. The damping ratio used in all modes is equal to 3 percent as suggested by IASS [19] for towers with regular bolt/nut connections. The result of time history analysis for the axial stresses generated in the main leg members under seismic loading in one orthogonal direction (X) is given in Fig.7. The result of the time history for displacement at the top of the tower is shown in Fig.8. Antenna-supporting towers must meet strict serviceability criteria. Seismic amplification may affect the top part of the tower where the antennas are attached and it should not result in any local permanent deformation after the earthquake. The lateral displacements along the tower in earthquake direction are within the order of 3% of the tower’s height as resulting from time history analysis (Fig.7). The results of structural assessment for the tower under the different dynamic analysis methods are summarized in Table 2.

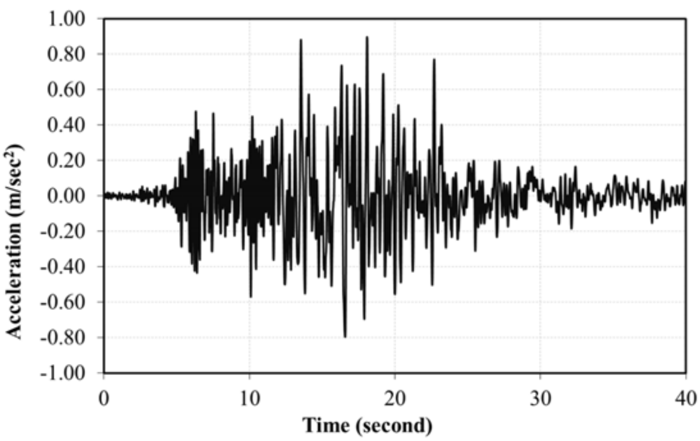


Fig 6. The 1995 Gulf of Aqaba earthquake (also known as Nuweiba earthquake)

All the results reported are those of the dynamic analyses including the static response due to self-weight. Maximum stresses in leg and bracing members are shown in Table 2. The structural response of the tower members is considerably low and all the stresses are within the allowable capacity of the structural elements. The actual fundamental frequency of tower is 1.25 Hz. Most of the main structural modes of the tower lie in the frequency range 0-25 Hz which may interpret the low considerably low generated stresses in main structural elements. As it can be noticed, the response of tower on the scale of the response spectrum curve for Marsa Alam city is considered moderate as the dynamic characteristics of the tower lie in the range greater than 1.25 Hz which exhibits moderate to low response.

Other staining actions as maximum support reactions and top lateral displacement at top of tower for different dynamic techniques are summarized in Table 3.

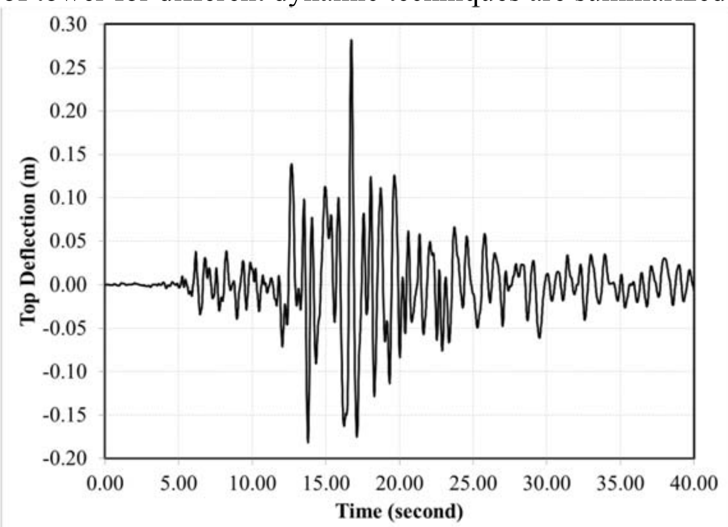


Fig. 7: Time history of tower displacement in direction of earthquake at top level

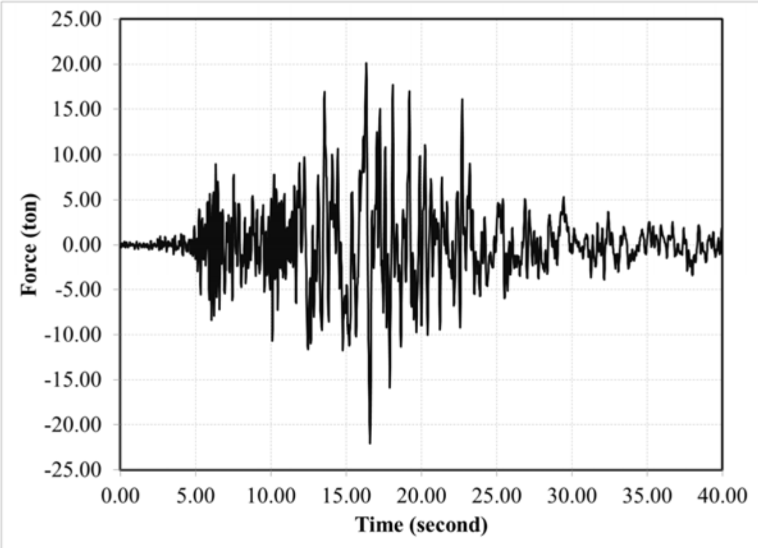


Fig. 8: Time history of normal stresses at the main tower leg

Table 2: Maximum stresses in structural elements under seismic loading using different techniques

No. of segment	Height (m)	Member	element no. on Ansys file	section	Applied stress due to O.W (t/cm^2)	applied stress calculated from (equivalent static) (t/cm^2)	applied stress calculated from (response spectrum) (t/cm^2)	applied stress calculated from (time history Aqaba) (t/cm^2)	allowable stress for member (capacity of section) (t/cm^2)
1	12.57	Leg	41	EA200X20	0.069	0.279	0.216	0.448	1.382
		bracing	53	EA100X10	0.011	0.018	0.383	0.225	1.112
		sec. bracing	81	EA50X5	0.007	0.022	0.136	0.027	0.116
2	22.7	Leg	231	EA150X15	0.078	0.443	0.382	0.652	1.387
		bracing	239	EA80X8	0.007	0.019	0.302	0.201	0.829
		sec. bracing	255	EA50X5	0.009	0.039	0.212	0.063	0.193
3	35.21	Leg	550	EA120X12	0.089	0.649	0.441	0.862	1.367
		bracing	437	EA70X7	0.003	0.022	0.505	0.203	0.663
		sec. bracing	453	EA50X5	0.007	0.028	0.252	0.06	0.155
4	62.67	Leg	741	EA120X12	0.086	0.626	0.441	0.745	1.266
		bracing	744	EA70X7	0.007	0.022	0.372	0.149	0.305
		sec.	758	EA50X5	0.001	0.001	0.089	0.009	0.270

		bracing							
5	80.56	Leg	1024	EA90X9	0.025	0.743	0.494	0.437	0.978
		bracing	1037	EA50X5	0.003	0.112	0.964	0.165	0.204
		sec. bracing	1045	EA50X5	0.000	0.001	0.021	0.006	0.691

Table 3: Maximum support reactions and top displacements

Analysis Type	**Equivalent static method**	**Response spectrum**	**Time History**
Maximum support reaction (ton)	35.347	25.460	22.094
Maximum top lateral displacement (mm)	272	236	282

7. Conclusion

The research presents useful comprehensive full-scale investigations of the dynamic analysis of an existing self-supporting telecommunication tower located in south region of Egypt within the red sea province. An accurate validated numerical model of the tower was utilized to reliably perform the dynamic analyses.

The study is one of other few research work that used experimental dynamic testing of large full-scale telecommunication tower to create a reliable representative numerical model of the existing tower. Accordingly more accurate results are expected from the structural dynamic analysis. The tower was thoroughly analysed using different analytical techniques in ANSI/TIA-222-G-2005 to evaluate its structural response under seismic loading. Structural actions (member forces, support reactions and deflections) generated in the analysis under seismic loads relevant to Egypt are considerably low. All the stresses are within the allowable capacity of the structural members. Hence, it can be expected that other existing towers in this height range among the large telecommunication wireless network will withstand without any problem under a minor to moderate earthquake, which is the most probable type of earthquake that can be expected in the southern region of red sea.

The equivalent static method for seismic analysis produces conservative results when compared with results of response spectrum analysis. As observed in this study the difference about 38% and 93% in support reaction estimation and stresses in main leg respectively.

The formula provided in ANSI/TIA-222-G-2005 for estimating the fundamental natural frequency of the self-supporting lattice tower produced a value with -8% of its measured counterpart from field dynamic testing. For strict following the methods provided in the standard, only the equivalent static analysis was based on this formula to perform the seismic analysis. For both the response spectrum and time history methods, the measured properties were the basis for validating the numerical model to complete the seismic analysis procedures in ANSYS.

In Ansys, it requires the first fifty modes of the 80.55 m tower, including both the lateral and torsional modes to be included in the response spectrum and time history analyses so that the condition of 90% of the modal mass of the tower can be achieved. In context of serviceability criteria for the function of the mounted antennas, the resulting displacement under the seismic loading is found to satisfy the required criteria.

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