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Dynamic Behavior of the Cable Structures and Towers of the Arecibo Radio-Observatory

Juan C. Morales, PhD, P.E.
Chairperson, Department of Mechanical Engineering, Turabo University, Gurabo, Puerto Rico
jcmorales@suagm.edu or jcmorales1564@yahoo.com

Luis E. Suárez, PhD
Professor, Department of Civil Engineering and Surveying, University of Puerto Rico, Mayagüez
Puerto Rico, lsuarez@uprm.edu

Abstract

This paper presents an overview of the results of a comprehensive study to determine whether the cable structures and towers that support the suspended platform of the Arecibo Radio-Observatory are capable of surviving a maximum-credible earthquake. The investigation was requested by the staff of the Arecibo Observatory to provide an understanding of the response of this unique structure during a strong seismic event. The Arecibo Observatory, part of the National Astronomy and Ionosphere Center (NAIC), is one of the most important research centers in the world for astronomic and ionospheric studies and thus maintaining its structural integrity is a top priority. In addition, the many unique components that form the observatory pose an unusual and challenging problem for its dynamic analysis. The structures of the observatory were designed and built in 1963 before modern seismic provisions became available and enforced in building codes. This contingency, along with the fact that the Arecibo Observatory is likely to be exposed to high seismic activity because Puerto Rico is surrounded by seismic faults as well as faults within the island itself, calls for a seismic assessment of its structures. This paper describes the creation of the finite element models and the modal analysis used to determine the vibration modes and natural frequencies of this unique structure. A detailed 3-dimensional model of the three reinforced concrete supporting towers, the main cables, the tie-down, auxiliary and backstay cables, the suspended platform and Gregorian dome was created in the program SAP2000. Preliminary results of the response of the structure to seismic loading are also presented.

Keywords

Arecibo Observatory, Cables, Seismic, Modal, Dynamics

1. Introduction

This paper presents an overview of the results of a comprehensive study to determine whether the cable structures and towers that support the suspended platform of the Arecibo Radio-Observatory are capable of surviving a maximum-credible earthquake. The investigation was requested by the staff of the Arecibo Observatory to provide an understanding of the response of this unique structure during a strong seismic event. The Arecibo Observatory, part of the National Astronomy and Ionosphere Center (NAIC), is one of the most important research centers in the world for astronomic and ionospheric studies and thus maintaining its structural integrity is a top priority. In addition, the many unique components that form the observatory pose an unusual and challenging problem for its dynamic analysis. The structures of the observatory were designed and built in 1963 before modern seismic provisions became available and enforced in building codes. This contingency, along with the fact that the Arecibo Observatory is likely to be exposed to high seismic activity because Puerto Rico is surrounded by seismic faults as well as faults within the island itself, calls for a seismic assessment of its structures.

Figure 1 shows the primary components of the structure. These include the three reinforced concrete towers, the suspended platform and its supporting cables. The primary reflector is not included in the study as it is an independently supported structure.

Figure 1: Overall view of the Arecibo Observatory (courtesy of NAIC)

The 900 ton platform, suspended by 18 cables at a height of 450 feet above the reflector, supports an azimuth arm, the Gregorian dome and the line-feed antenna (see Figure 2). The azimuth arm is capable of rotating about a vertical axis through its centroid. The dome and the antenna are capable of sliding along the arm to a maximum 20-degree angle.

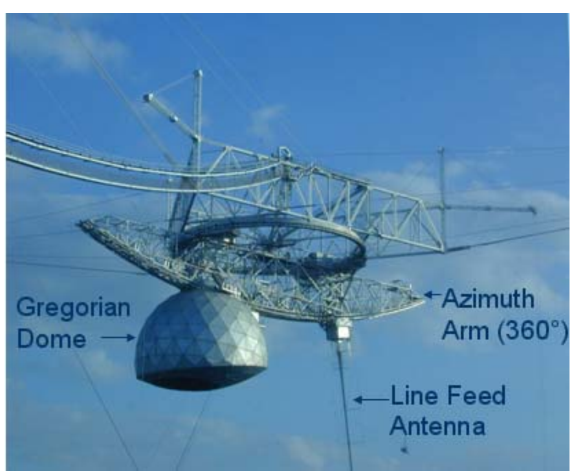


Figure 2: Suspended platform and its main components (courtesy of NAIC)

Each tower has a set of six platform cables and seven backstay cables as shown in Figure 3. The backstays originate in massive reinforced-concrete anchors. Towers T4 and T12 are 265 ft high while tower T8 is 365 ft high. All three tower tops are at the same elevation. The combined volume of reinforced concrete in the three towers is 9,100 yd^3 which represents a combined weight of 17,800 tons.

Figure 3: Tower T8, the tallest (365 ft) of the three towers. (courtesy of NAIC)

As shown in Figure 4, the towers are of cruciform shape which gives them doubly-strong axes, one of them aligned with the cables. The dimensions vary approximately every 60 feet as the towers are stepped in by approximately 3 feet on all sides. The thickness is maintained constant at 6 feet.

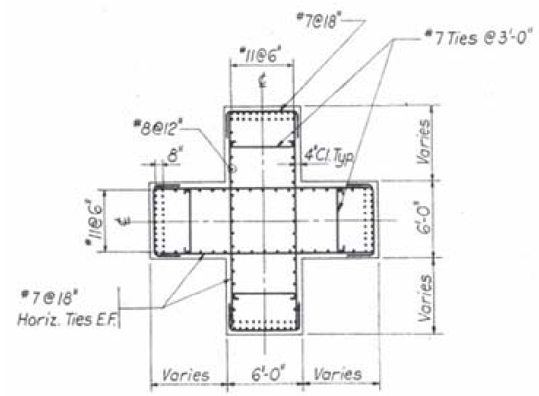


Figure 4: Cruciform shape of tower cross-section (courtesy of NAIC)

The six platform cables per tower are distributed into four, 3.0 inch diameter, main cables which are attached to the platform corners, and two, 3.25 inch diameter, auxiliary cables which are attached at approximately the 2/3 points of the platform (see Figure 5).

Figure 5. Attachment points of cables to platform. T4 cables highlighted. (photo by author)

Two, 1.5 inch diamter, vertical tiedown cables run from each platform-corner extension, as shown in Figure 6, to mechanical jacks below the primary reflector. The tiedown-cables/jack system permits adjustment of each corner of the platform with millimeter precision to keep the platform leveled.

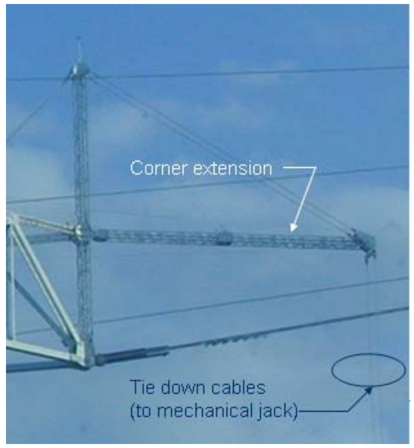


Figure 6: Tiedown cables and platform attachment points (courtesy of NAIC)

2. Literature Survey

Most of the relevant publications were found in the American Society of Civil Engineering (ASCE) Journal of Bridge Engineering, and were related to suspension bridges and cable-stayed bridges. Bridges are the most closely-related structures to the Arecibo Observatory. The Arecibo Observatory is a one-of-a-kind structure and thus there is not any study dealing with this unique and complex structural system. The literature was consulted to determine the most appropriate methodology to conduct the study. From the work of Xu (1997), Chang (2001), Zhang (2001), Cunha (2001), and Astaneh-Asl (2001) it was determined that commercial finite element software, such as SAP2000, was more appropriate for this ample-breadth problem. Fixed supports were assumed in all the bridge studies cited above. It was also found that this type of cable structure behaves in an essentially linear fashion (Ren, 1999a) on account of the very high tensions generated in the cables. The sag effect of the cables is taken into account with the linearized Ernst's equivalent tangent modulus of elasticity, which is valid while the tensions in the cables do not vary significantly. It is also reported in Ren (1999a) that the results from using large displacements are nearly identical to a linear (small displacement) model. In all cases where field measurements are used to confirm the finite element models (Xu, 1997 and Chang, 2001) these were obtained from separate studies. The tensions in the cables of the Arecibo Observatory have been measured by the Arecibo Observatory staff and there is excellent agreement with the values specified in the drawings.

3. Finite Element Model Creation

The computational experiments on the structures of the Arecibo Observatory were conducted with three finite element models of increasing complexity using SAP2000 software (Version 9). Model A (similar to Figure 7 but without tiedown cables) models the platform as two rigid equilateral triangles and assumes the platform weight is uniformly distributed in the triangles; Model B, shown in Figure 7, is equal to Model A but includes the tiedown cables; Model C (Figure 8) explicitly models the azimuth arm, the Gregorian dome and the line feed antenna, which are modeled as rigid bodies with the appropriate mass assigned to them. Model C is analyzed in a worst-case condition: the azimuth arm points at tower T8; the

Gregorian dome is at the extreme end (20˚ position) of the azimuth arm; and the line feed antenna is at stow position.

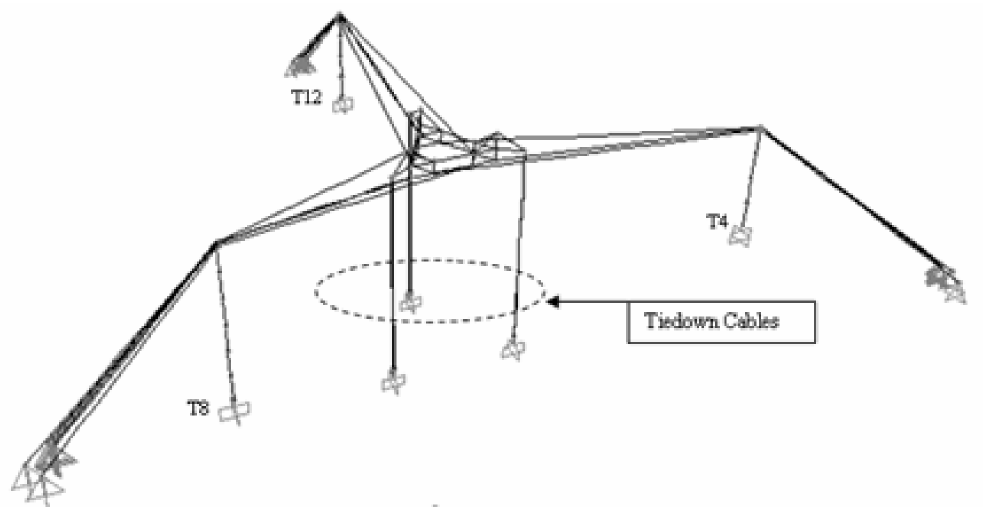


Figure 7: Finite element Model B

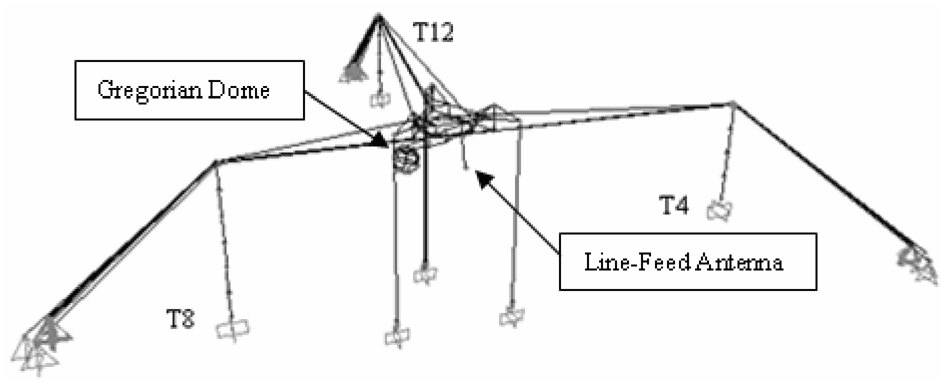


Figure 8: Finite element Model C

Drawings supplied by the Arecibo Observatory were used to generate the models. The towers were modeled with frame elements and were fixed at their bases. The local coordinate systems of the towers were rotated so the "2" direction was oriented radially outwards, which corrresponds to the general direction of the cables. The unit weight of concrete was taken as 150 pcf while the modulus of elasticity was taken as 3122 ksi (based on a compressive strength of 3000 psi). The total area of the section, moment of inertia, shear area and torsional constant were calculated for each segment of the tower and were input as section properties. The platform components were modeled with frame elements with very high stiffness. The platform mass was uniformly distributed in the triangular chords in Models A and B. In Model C, the mass of the azimuth arm, the Gregorian dome and the line-feed antenna, was assigned separately to each of these components while the remaining mass was uniformly distributed in the triangular chords. The cables were modeled with cable elements, and their geometry is specified in the "undeformed geometry" state. The specific weight of the cables was taken as 490 pcf. The effective area of the cables, which is used for weight and mass calculations, was taken as 77% of the nominal area of the cable, a consequence of the voids between cable strands. The linearized modulus of elasticity was calculated using Ernst's equivalent tangent modulus, and it was shown that the asymptotic value of 24,000 ksi had essentially been reached. The cable anchors were modeled as pinned restraints.

4. Deformed Equilibrium State Due to Dead Loads

A key requirement for the finite element analysis of cable structures is to start from the deformed equilibrium state due to dead loads, to include the geometric stiffness matrix of the cable elements. Consideration of the geometric stiffness matrix is accomplished in SAP2000 by running a p-delta analysis. P-delta effects, usually ignored in many typical structural applications, are of primary importance in cable structures. The reason is that cables elements derive practically all their lateral stiffness from their state of tension (defined by the 'geometric' stiffness matrix) rather than from their physical properties (defined by the 'mechanical' stiffness matrix). Tension in the cables is generated during the p-delta analysis as the self-weight is gradually applied to the model. The cables tighten as a reaction to the applied weight. The 'structural' or 'total' stiffness matrix of the structure is calculated at the end of the p-delta analysis and is defined as the sum of the 'mechanical' and the 'geometric' stiffness components. The software provides an option to run the p-delta analysis with large displacements. In this option the nodal coordinates are updated after each step to account for their displacement. The results obtained with and without the large displacement option are nearly identical. This same result was observed in Ren (1999b) the case of a cable-stayed bridge.

The desired deformed equilibrium state due to dead loads is defined by the following three criteria:

1. The elevation of the platform at the end of the P-delta analysis should match the elevation specified in the drawings. The platform descends during the p-delta analysis as cables stretch due to the applied self-weight.

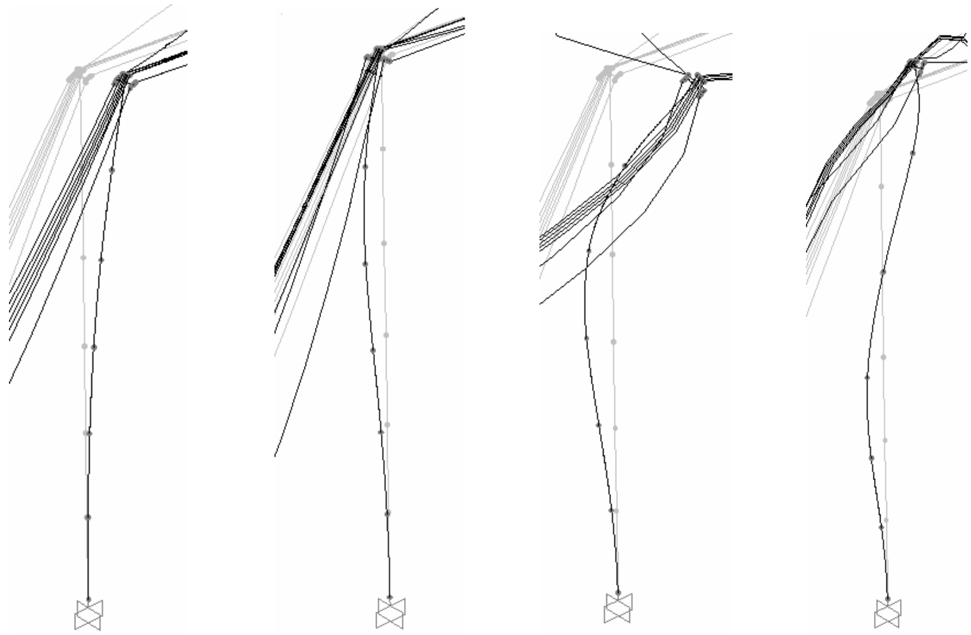
2. The three towers should be vertical, i.e., the x and y displacements at the top of the towers should be ~ zero. This requires preloading of the backstay cables, otherwise, the three towers would flex radially inward due to the applied platform weight. Cable preload is achieved by applying negative temperatures to the cables. The resulting negative thermal strains simulate the preload applied to backstays with hydraulic jacks, at their concrete anchors.

3. The magnitude of the tension in each cable should be equal to the tension values specified in the drawings.

Fifteen iterations were required to achieve the first two criteria. The third criteria is achieved when the three service cables were included in the model. Without the service cables, the tension values in the backstays are approximately 10% below the values specified in the drawings. However, the service cables were eventually ignored as it was determined that the natural periods of the towers and the platform, as well as the seismic response, are insensitive to a 10% variation in tension, i.e., nearly identical results are obtained if the service cables are included as well as if they are not included.

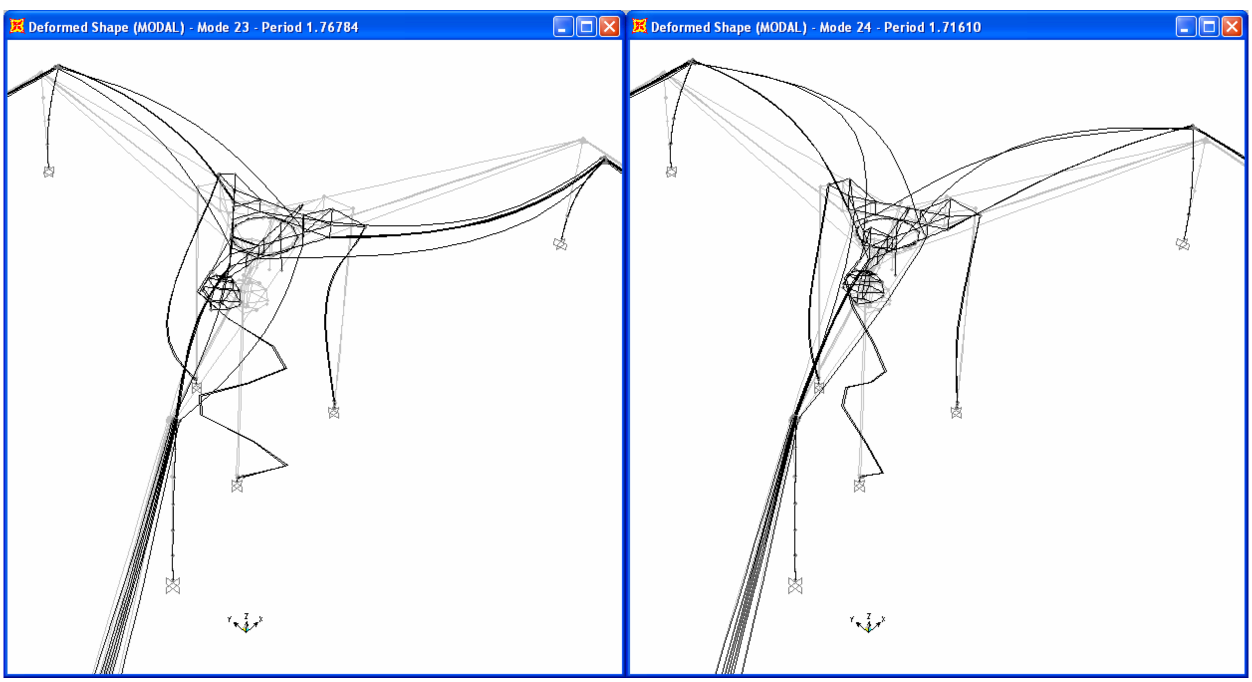
5. Modal Analysis Results

The three models (A, B, and C) were analyzed to observe the natural response of the structure. In addition, several sensitivity studies were performed to obtain a clearer picture of the modal response. All three models revealed the same three different types of mode shapes (platform modes, tower modes and independent cable modes), in addition to interactions between them. The strongest interactions occur at periods between 1.8 and 0.7 seconds. The first four tower mode shapes of tower T8 are shown in Figure 9. Two of the strongest interaction modes are displayed in Figure 10, which shows towers T4 and T12 in fundamental mode, while in-phase (Figure 10.a) and out-of-phase (Figure 10.b). When the two towers are in-phase with each other, the platform exhibits very strong rotation about its vertical axis. When the two towers are out-of-phase the platform exhibits very strong rotation about a N-S axis and the Gregorian dome displaces up and down. All rotational modes of the platform participate in interactions. On the other hand, the vertical displacement mode of the platform barely participates in interactions.



1st mode T= 2.26 sec | 2nd mode T = 1.1 sec | 3rd mode T = 0.70 sec | 4th mode T = 0.45 sec

Figure 9: First four flexural modes of tower T8



(a) Towers T4 and T12 in-phase (1.7 sec)

(b) Towers T4 and T12 out-of-phase (1.2 sec)

Figure 10: Strong interaction modes. Towers T4 and T12 in-phase, and out-of-phase

The modal analysis results for Models A, B, and C are shown in Table 1. They are essentially identical except for two instances. In the first instance, the periods of the platform modes (the first three modes in the table) are ~50% lower for models B and C than for Model A. This is due to the presence of tiedown cables which add considerable stiffness to the platform. In the second instance, the periods of the tiedown cables (last two rows of the table) exhibit different periods in Model C due to the unbalanced Gregorian dome location within the platform. The T8 tiedown is almost slack so its period is very high. On the other hand, the T4 and T12 tiedowns are much tighter, to counterbalance the weight of the Gregorian dome, so their periods are lower.

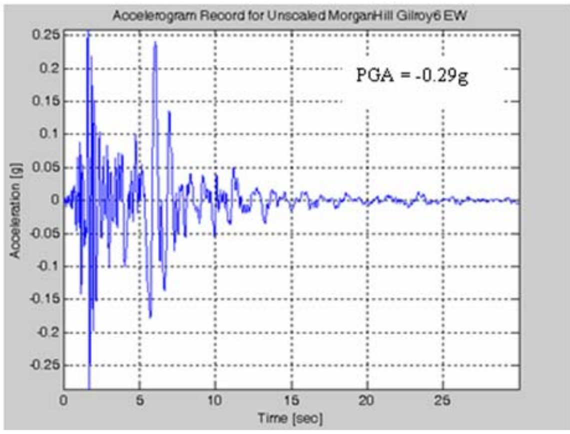
Table 1: Natural periods obtained for Models A, B, and C

		Model A (No Tiedowns)		Model B (With Tiedowns)		Model C (With Tiedowns)		
ID	Mode Description	Mode Number	Period [sec]	Mode Number	Period [sec]	Mode Number	Period [sec]	Max % Diff.
A	Platform Vertical Displacement	1	4.683	14	2.146	22	2.141	-54.3%
B	Platform Rotation about East-West Axis	2	3.529	17	1.628	31	1.530	-56.6%
C	Platform Rotation about North-South Axis	3	3.525	18	1.587	30	1.578	-55.2%
D	1st Mode Tower T8	4	2.264	13	2.260	9	2.264	0.2%
E	1st Mode Towers T4 and T12 - In Phase (Platform rotates about vertical axis)	5	1.788	15	1.764	23	1.768	-1.3%
F	1st Mode Towers T4 and T12 - Out of Phase	6	1.722	16	1.723	24	1.716	-0.4%
G	2nd Mode Tower T8	68	1.100	82	1.148	116	1.085	-5.5%
H	2nd Mode Towers T4 and T12 - In Phase	101	0.752	137	0.739	153	0.747	-1.7%
I	2nd Mode Towers T4 and T12 - Out of Phase (Cradle Motion of Platform)	102	0.733	138	0.714	162	0.727	-2.6%
J	3rd Mode Tower T8	107	0.698	139	0.695	163	0.695	-0.4%
K	3rd Mode Tower T4	186	0.451	258	0.448	264	0.448	-0.7%
L	3rd Mode Tower T12	188	0.447	260	0.444	265	0.444	-0.7%
M	4th Mode Tower T8	187	0.450	259	0.447	263	0.448	-0.7%
N	4th Mode Tower T4	301	0.327	385	0.326	378	0.327	-0.3%
O	4th Mode Tower T12	302	0.322	410	0.321	403	0.321	-0.3%
P	5th Mode Tower T8	315	0.305	423	0.303	424	0.303	-0.7%
Q	1st Mode Auxiliary Cables	11	1.529	20	1.517	32	1.519	-0.8%
R	1st Mode Main Cables	35	1.284	57	1.283	62	1.284	-0.1%
S	1st Mode T12 Auxiliary Backstays	61	1.181	83	1.146	91	1.146	-3.0%
T	1st Mode T8 Auxiliary Backstays	65	1.151	87	1.127	95	1.126	-2.2%
U	1st Mode T4 Auxiliary Backstays	22	1.398	32	1.381	45	1.380	-1.3%
V	1st Mode T12 Main Backstays	69	1.109	91	1.089	107	1.090	-1.8%
W	1st Mode T8 Main Backstays	79	1.065	101	1.057	118	1.057	-0.8%
X	1st Mode T4 Main Backstays	26	1.300	48	1.29	53	1.291	-0.8%
Y	1st Mode Tiedowns T8	N/A	N/A	1	2.633	1	6.575	60.0%
Z	1st Mode Tiedowns T4 & T12	N/A	N/A	1	2.633	10	2.21	-19.1%

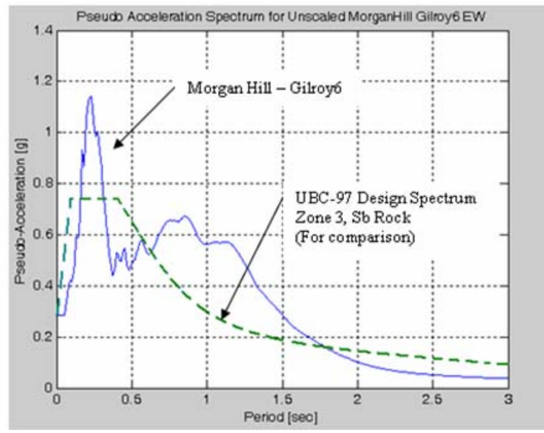
6. Seismic Response

Five different earthquakes were considered in the final seismic studies. The two maximum-credible records were the 1984 Morgan Hill earthquake recorded at the Gilroy #6 station, and an artificial earthquake generated with program SIMQKE, which is compatible with the UBC-97 design spectrum for seismic zone 3 and rock type S_b. The 1984 Morgan Hill record represents the seismic hazard for the vicinity of the Arecibo Observatory, as determined by Llop (2002). A third input was the weaker 1966 Parkfield earthquake recorded at station 097 which, according to Irizarry (1999), represents the expected seismic hazard for the city of San Juan. The last two inputs were the 1986 San Salvador earthquake recorded at CIG station, and the 1994 Northridge earthquake recorded at Castaic station. Both of these records are very strong and, according to Irizarry (1999), these represent the seismic hazard for the cities of Ponce and Mayagüez. The five earthquakes were considered to study the performance of the Arecibo

Observatory structures under a wide range of conditions. The accelerogram of the Morgan Hill EW record, and its accompanying response spectrum, are shown in Figure 11. A plot of the deformed geometry of the model, 6.2 seconds into the Morgan Hill earthquake, is shown in Figure 12. At 6.2 seconds, very high bending moments are recorded in the towers. Table 2 presents the maximum bending moments recorded at the bottom of each of the tower segments for the Parkfield, Artificial, and Morgan Hill earthquakes. TSEC1 represents the top of the towers while TSEC6 represents the base.



(a) Accelerogram



(b) Response spectrum

Figure 11: Accelerogram and response spectrum of the EW Morgan Hill-Gilroy#6 EW record

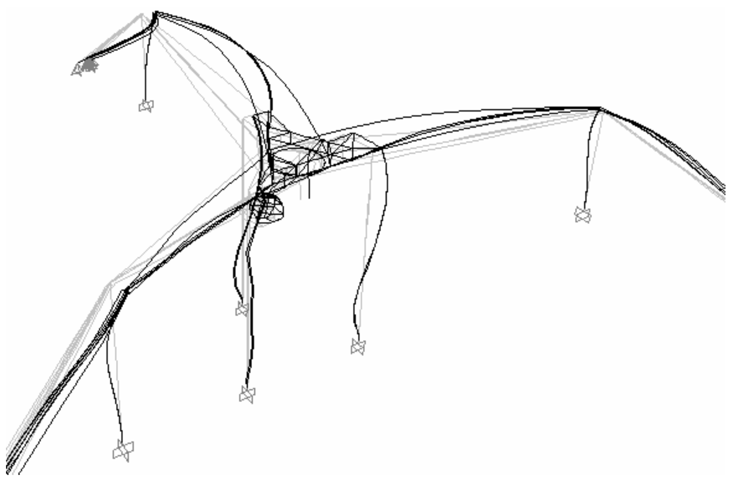


Figure 12: Deformed geometry (100x magnification), 6.2 sec into the Morgan Hill earthquake

Table 2: Maximum bending moments at the bottom of the tower segments

	Parkfield (Station 097)	Artificial (SIMQKE)	Morgan Hill (Gilroy #6)
Section	M_max [kip-ft]	M_max [kip-ft]	M_max [kip-ft]
TSEC1	14440	23970	32900
TSEC2	20440	39690	38240
TSEC3	26000	57700	50010
TSEC4	56400	96380	103400
TSEC5	53340	133000	165000
TSEC6	97100	227000	275900

7. Conclusions

The investigation revealed three different types of mode shapes (platform modes, tower modes and independent cable modes), in addition to interactions between them. The strongest interactions occur at periods between 1.8 and 0.7 seconds. Differences in the natural periods of vibration (between the three models considered in this study) are due to the presence of the tiedown cables which stiffen the platform in the vertical direction. Partial results of the seismic response of the structure were provided. The maximum bending moments at the base of each of the tower segments were included for the Parkfield, Artificial (UBC-97 compatible), and Morgan Hill. Further research is required to determine if the towers are capable of resisting the maximum bending moments in the elastic regime.

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