STRONG MOTION INSTRUMENTATION IN THE BRITISH VIRGIN ISLANDS

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ABSTRACT:

This paper describes recent strong motion instrumentation activities carried out within a collaborative agreement between The Puerto Rico Strong Motion Program (PRSMP) and the Department of Disaster Mitigation of the Government of British Virgin Islands (BVI). The instrumentation activities involved installation of four free field strong motion stations and the instrumentation of two structures [2]. The paper also presents an overview of the seismic setting of the BVI. The instrumentation efforts hope to improve the capability of BVI to respond to a natural hazard such as an earthquake. This article also presents details of the seismic analysis carried out in the design of the instrumentation scheme for one of the instrumented structures. This analysis included dynamic analysis with models calibrated with ambient vibration modeling. The analytical work was used to select the location of the instruments and to assess the seismic vulnerability of the structure.

KEYWORDS: Seismic Instrumentation, Strong Motion, Structure Instrumentation

1. INTRODUCTION

The British Virgin Islands (BVI) are comprised of more than 50 islands and cays located in the lesser Antilles as shown in Figure 1. The main island of the BVI include Tórtola, Beef Island and Virgin Gorda. The population of BVI is about 25,000 inhabitants in an area of about 59 square miles. The seismicity of the BVI is considered high due to its location at the northeastern corner of the Puerto Rico platelet which is part of the Caribbean Tectonic Plate. Historic records show evidence of multiple damaging earthquake. Examples of large seismic events include: July 11, 1785 with a Modified Mercally Intensity (MMI) of VIII, February 8 1843 (MMI=V) and November 18, 1867 (MMI = VII). The main source of tectonic activity affecting the BVI are the Anegada Trough, which runs from the Southwest, and the Puerto Rico Trench located to the North of BVI. The Maximum Credible Earthquake for the Anegada Trough has been established as 7.5 and for the Puerto Rico Trench as 8.0 [1, 4]. The expected Peak Horizontal Ground Acceleration in the BVI varies from 0.20g to 0.25g with a recurrence of 475 years [1, 4] for a zero attenuation variability.

From this brief description of the seismic setting of the BVI it is clear that efforts towards establishing strong motion instrumentation in the BVI region is warranted. This paper describes instrumentation efforts carried out by the PRSMP within a collaborative agreement with the Government of the BVI. Specifically, the instrumentation work involved installation of 4 free-field strong motion stations (2 in Tórtola, 1 in Beef Island and 1 in Virgin Gorda) and seismic instrumentation of the Control Tower of the International Lettsome Airport at Beef Island. In the near future, the Government Central Administration Building of BVI will be instrumented. For the sake of brevity, this article will primarily describe the free-field instrumentation and the instrumentation of the Control Tower of the Beef Island Airport.

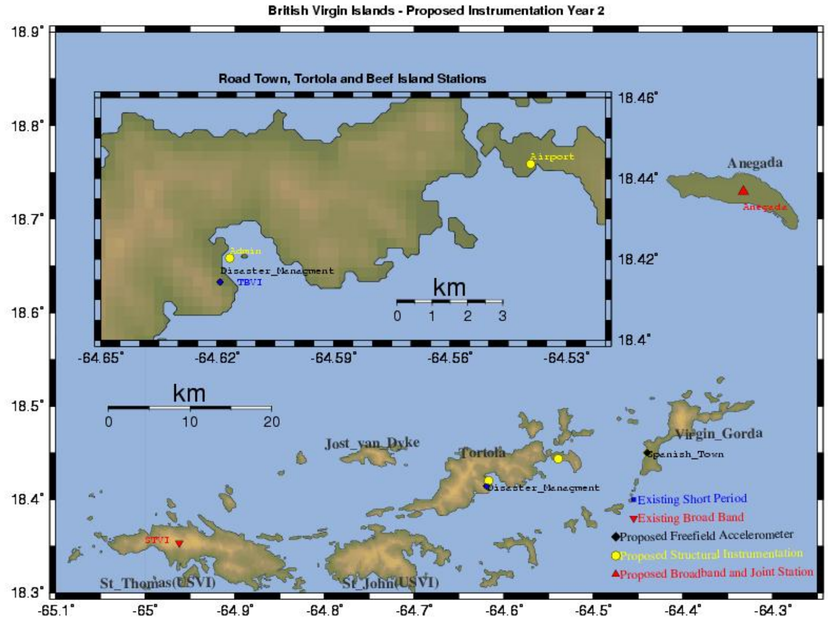

Figure 1 Summary of Seismic Instrumentation at British Virgin Islands

2. FREE FIELD STRONG MOTION STATIONS

Four strong motion free field stations were installed in BVI; three during the summer of 2006 and the last one during the summer of 2007. All the accelerographs are ETNAs, and are set to record accelerations up to ±2g with a sampling rate of 200 sps. The trigger level of the sensors in both horizontal directions was set at 1%g, while in the vertical direction it was set at 0.2%g. The pre-event memory, post event memory, and minimum run time were set for 20 sec, 40 sec, and 60 sec, respectively. The instruments includes a 12 volts internal battery which will allow the data logger to record any earthquake even during a black out. The communication with the instruments is via Internet. The first strong motion station in BVI was installed at the backyard of the Government Central Administration Building (Latitude 18.470278º N, Longitude 64.616944º W) as can be seen in Photo 1A. The GPS antenna was installed on the southwestern shear wall of the structure. The station code was established as TRT1.

The second free field station was installed at the courtyard of the Control Tower Building at Beef Island (Latitude 18.443989º N, Longitude 64.538246º W) as can be seen in Photo 1B. The station was named as BFI1. The third station was installed at the yard of the Department of Disaster Management Building (Latitude 18.415142º N, Longitude 64.618469º W) at Tórtola as can be seen in Photo 1C. The station was named as TRT2. The last station was installed at the yard of the Government Administration Building in Spanish Town at the Island of Virgin Gorda (Latitude 18.448736º N, Longitude 64.434250º W) as can be seen in Photo 1D. The station was named as VRG1.

3. INSTRUMENTATION OF LETTSOME AIRPORT CONTROL TOWER

3.1 General Description of the Structure

During the year 2007 the Lettsome Airport Control Tower at Beef Island was seismically instrumented. It is a six-story RC structure connected to the administration building which is a one-story metal deck roof structure. See Figure 2. There is a construction joint between the one-story structure and the tower making each one an independent structure of the other. On the top of the RC tower there is a metal structure where the control cabin is located.

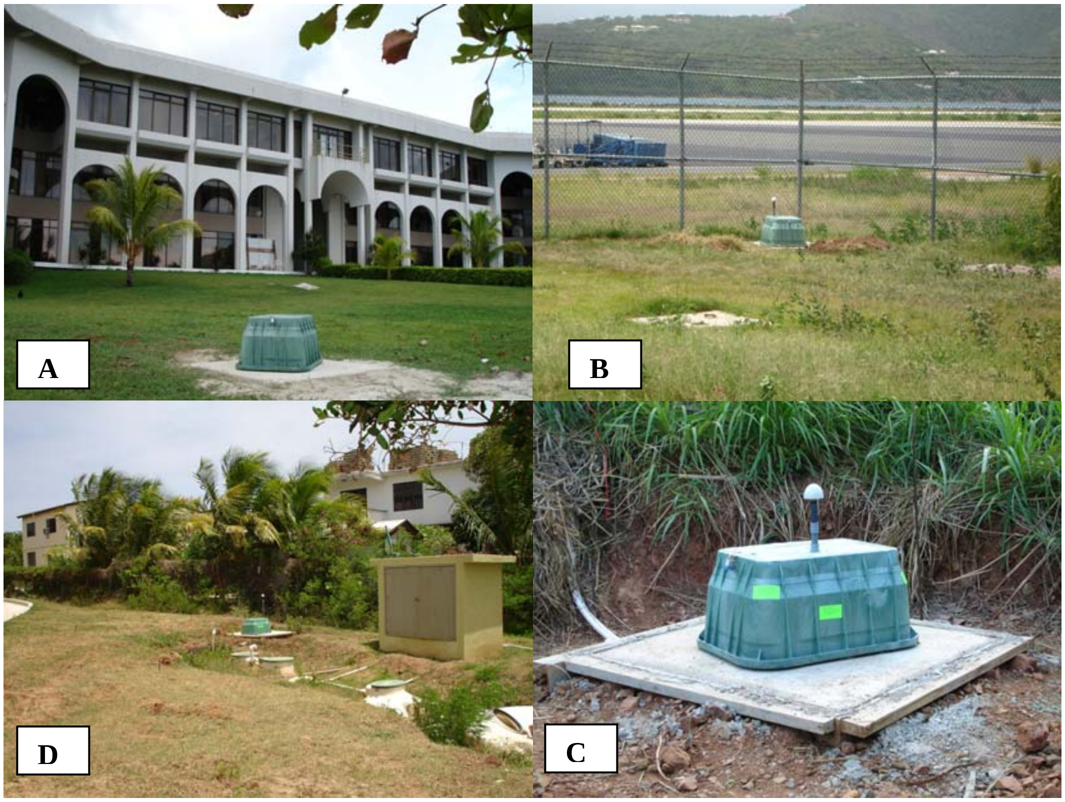

Photo 1 Strong motion free field stations at BVI. A) Government Central Administration Building in Road Town, Tórtola. B) At the Lettsome Airport in Beef Island. C) At the Department of Disaster Management in Road Town, Tórtola. D) At the courtyard of the Government Building in Spanish Town, Virgin Gorda.

The structure is founded on a raft foundation. In the sixth floor, the structure has an overhang. In the overhang, the computer room is located. The tower has a trapezoidal shape section in the first five stories as shown in Figure 3B. The trapezoidal shape section is divided with a central RC wall in the longitudinal direction. On one side of the central wall is the elevator shaft; while on the other side is a metal stair. The peripheral wall of the trapezoidal section is 12" thick, while the central wall is 10" thick. The six-story floor and roof has an 8" thick RC slab that extends the whole overhang. The total height of the RC structure is 75.2'.

3.2 Structural Dynamic Characteristics

The RC tower was modeled in the computer program ETABS using shell elements. The accumulated height and weight of the structure are depicted in Table 1. The cracked moment of inertia of the RC elements were considered according to the ACI 318-05 [5]. In particular, the cracked moment of inertia of the RC walls was used as 0.70 the gross moment of inertia for every story.

Table 2 shows the first 21 modes of vibrations, their respective period, direction according the coordinates shown on Figure 4, and its modal participation component obtained from the ETABS model. As can be notice, with the modes 2 and 6, the 81% of the modal participation in the x-direction can be capture, while with the modes 1 and 4 about 83% of the modal participation in the y-direction is observed.

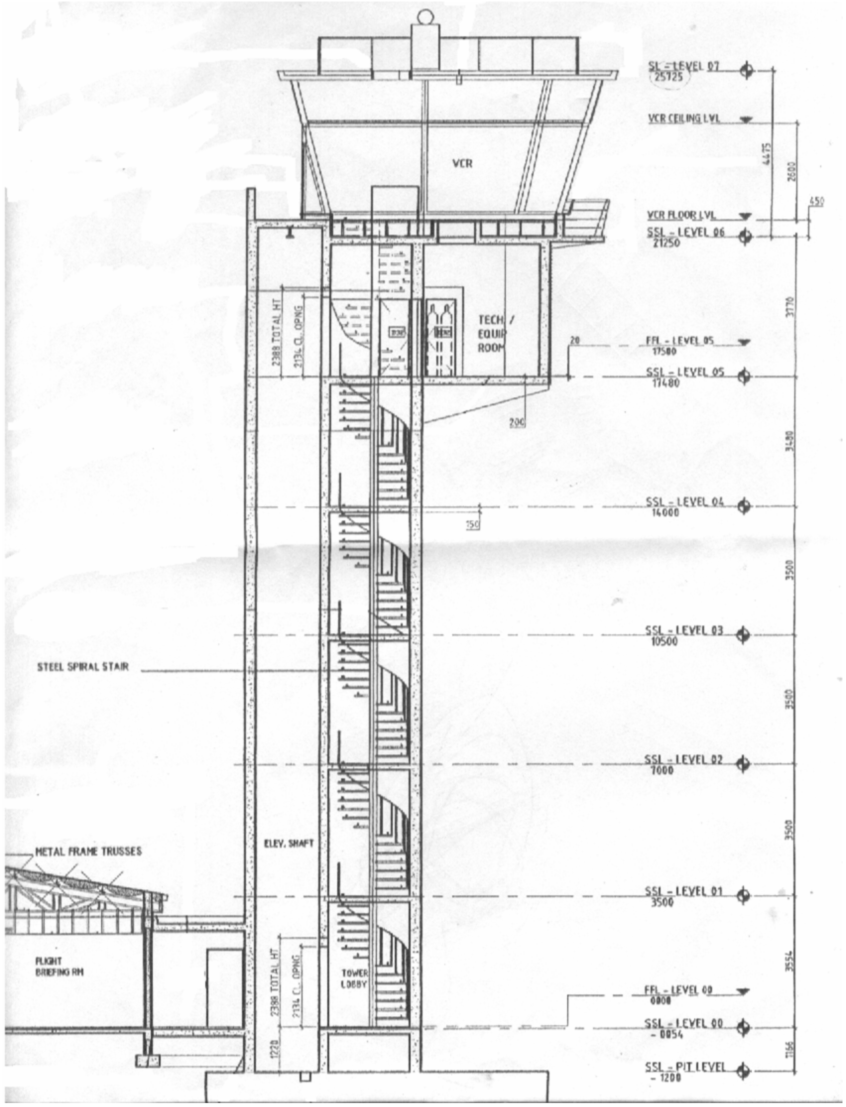

Figure 2 Control Tower with Part of the One-Story Administration Building.

In order to calibrate the model an ETNA strong motion accelerograph was placed on the overhang of the sixth story. Ambient vibration data was colleted for about 22 hours for the three orthogonal sensors at 100 sps. After cleaning up the data eliminating frequencies from different machines, the first nine frequencies of the structure were obtained. These ambient vibration frequencies are compared with the ones obtained in the computer model in Table 3. As can be seen from the table, seven out of the nine frequencies shows that the computer model is slightly stiffer than what the ambient vibration shows. The biggest different is about 20% in the second mode. The only two frequencies that show a more flexible structure in the computer model are the one related to the rotational mode around Z-axis. This difference might be partially explained by the absence of the metal stair in the computer model.

If the stiffness of the first bottom third part of the structure is reduced by considering the cracked moment of inertia of the RC wall as one-half its gross moment of inertia, the correlation between the two frequency sets is greatly improved as can be seen in the same Table 3.

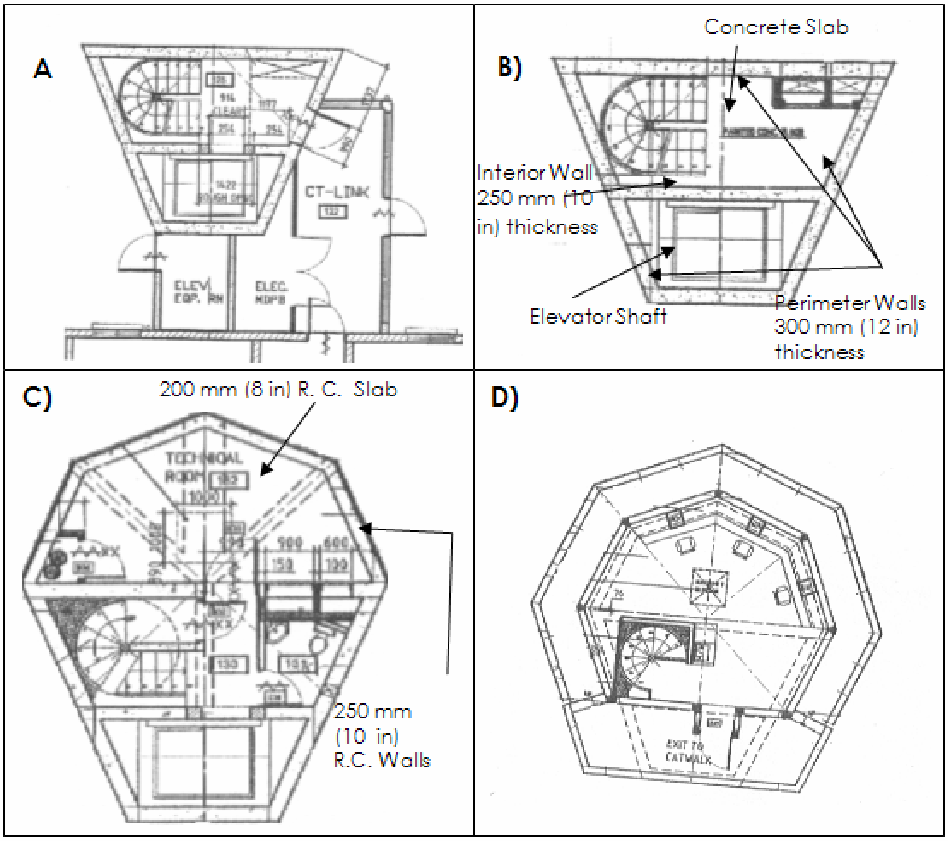

Figure 3 A) First story section, B) Second to fifth story section, C) Sixth story section and, D) Section of the steel cabin on the top of the tower.

Table 1 Control Tower accumulated height and weight.

Level	Height	Accumulate Height	Weight	Accumulate Weight
	feet		kips	
7	14.68	88.38	25.0	25.0
6	12.37	73.70	277.1	302.1
5	11.41	61.34	163.4	465.5
4	11.48	49.92	129.8	595.3
3	11.48	38.44	129.8	725.1
2	11.48	26.96	129.8	854.9
1	11.66	15.48	131.7	986.6
0	3.82	3.82	51.8	1,038.4

3.3 Sensors Location

Given the fact that a plastic hinge at the bottom of the RC trapezoidal-shape section wall will create and incipient collapse of the tower, it is of utmost importance to properly record the acceleration input of the earthquake.

Considering the raft foundation as a rigid slab, with six sensors properly located, the entire movement of the ground slab can be recorded.

Table 2 Modes of vibration, frequencies and modal participation.

Mode	Period (Seconds)	Frecuency (Hertz)	UX %	UY %	UZ %	RX %	RY %	RZ %
1	0.28926	3.46	0.00	64.90	0.14	97.77	0.00	0.00
2	0.24710	4.05	64.20	0.00	0.00	0.00	97.32	1.87
3	0.09208	10.86	0.42	0.00	0.00	0.00	0.41	81.77
4	0.06198	16.13	0.00	18.48	4.37	1.85	0.00	0.01
5	0.05352	18.69	0.00	0.01	0.02	0.00	0.00	0.00
6	0.04898	20.42	17.18	0.00	0.03	0.00	1.70	0.22
7	0.04519	22.13	0.00	0.07	0.61	0.01	0.00	0.00
8	0.04295	23.29	0.00	2.01	24.64	0.05	0.00	0.00
9	0.04209	23.76	2.99	0.00	0.02	0.00	0.25	0.00
10	0.03725	26.85	0.00	0.06	50.22	0.02	0.00	0.00
11	0.02783	35.93	0.00	4.85	2.29	0.17	0.00	0.21
12	0.02696	37.10	0.09	0.18	0.07	0.01	0.00	5.69
13	0.02141	46.71	6.05	0.00	0.00	0.00	0.18	0.99
14	0.01848	54.10	0.01	2.61	0.56	0.02	0.00	0.00
15	0.01363	73.38	0.00	1.11	2.36	0.01	0.00	0.00
16	0.01251	79.96	0.13	0.34	5.32	0.00	0.00	0.00
17	0.01080	92.60	6.64	0.00	0.27	0.00	0.01	0.00
18	0.00908	110.12	0.10	0.26	0.53	0.00	0.00	0.00
19	0.00812	123.14	0.01	5.06	0.00	0.00	0.00	0.00
20	0.00622	160.80	0.04	0.01	6.16	0.00	0.00	0.06
21	0.00038	2,638.52	0.03	0.01	0.11	0.00	0.00	0.04

Table 3 Correlation between the periods by the computer model and with ambient vibration.

			Computer Model (Etabs)			
Mode	Description	Ambient Vibration	Icr = 0.7 Igross	Difference (%)	Icr = 0.5 Igross (Only first and second level)	Difference (%)
1	First mode in Y direction	0.3349	0.289257	-13.63	0.331167	-1.11
2	First mode in X direction	0.3092	0.247102	-20.08	0.283084	-8.45
3	First rotational mode around Z	0.0843	0.092075	9.22	0.101927	20.91
4	Second mode in Y direction	0.0684	0.061983	-9.38	0.066748	-2.42
6	Second mode in X direction	0.0600	0.048980	-18.37	0.052507	-12.49
8	First mode in Z direction	0.0452	0.042946	-4.99	0.045120	-0.18
11	Third mode in Y direction	0.0326	0.027834	-14.62	0.029453	-9.65
12	Second rotational mode around Z	0.0262	0.026957	2.89	0.028111	7.29
13	Third mode in Z direction	0.0244	0.021409	-12.26	0.022831	-6.43

The Figure 4 shows the location of the six sensors installed at the base of the elevator core. A 12-channels K2 datalogger with an internal triaxial sensor was installed on the ground slab of the tower at the bottom part of the elevator shaft. Additional three uniaxial sensors were also installed on the ground slab so that the lateral movement in horizontal directions, the vertical movement, the torsion, and the rocking of the ground slab around both main axes can be recorded. In order to capture the fundamental mode of vibration in both horizontal directions, as well as the torsion of the structure, three additional uniaxial sensors were installed on the roof of the sixth floor. In addition, to capture the second mode of vibration in both horizontal directions a similar set of three uniaxial sensors were installed on the third floor (at about mid-height of the structure). It should be pointed out that an ETNA strong motion accelerograph was installed as free field station. See Photo 1B.

3.4 Structural Capacity

The structural drawings of the control tower were not available, hence, several assumption were necessary to be done. The compressional concrete strength was considered as 4,000 psi. The yield strength of the reinforcing bars was used as 60,000 psi. A vertical as well as horizontal reinforcing ratio in the wall was considered as 0.0025. The constitutive relationship suggested by Kent & Park [3] was used for the concrete as well as for the reinforcing steel. The moment vs curvature diagram was developed for the trapezoidal shape section using an axial load of 1,100 kips as shown in Figure 5. Using the right-hand rule, 0-degrees is taking moment around the x-axis in the negative direction, 180-degrees in the positive direction, and 90-degrees taking moment around the y-axis.

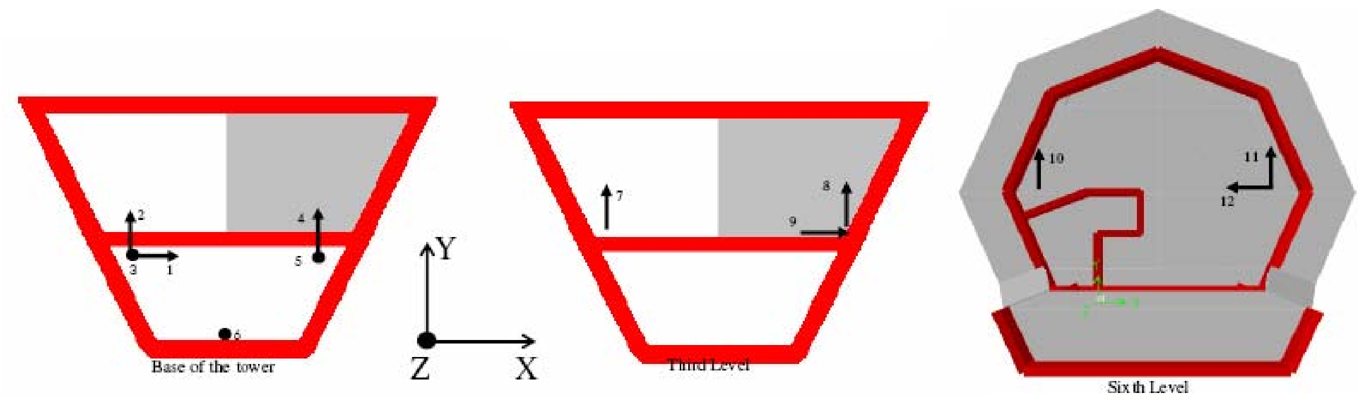

Figure 4 Sensors location

As could be obtained from Figure 5 the yield moment for the RC wall was calculated as 18,458 k-ft, 23,683 k-ft, and 21,500 k-ft for 0, 90 and 180-degree angles, respectively. The ultimate moment was found to be 23,842 k-ft, 29,808 k-ft, and 28,492 k-ft, respectively, for the same angles. On the other hand, the nominal shear strength of the RC wall was calculated using the Eqn. 21-7 of the ACI 318-05. The nominal shear strength was calculated as 2,082 kips and 1,181 kips for the x and y-direction, respectively.

3.5 Behavior of the Control Tower Subjected to El Centro Earthquake, 1940

The computer model of the structure was subjected to the May 18, 1940 El Centro earthquake record using both component simultaneously. This record has a peak ground acceleration of 0.22g in the EW direction, and 0.35g in the NS direction. Two scenarios were considered, in the first one the EW component was considered in the y-direction while the NS component was in the x-direction. In the second scenario the component were inverted.

Table 4 present the maximum base shear and overturning moment for each direction and for each scenarios. It can be noted that maximum base shear in the x-direction is -548 k, while in the y-direction is -509 k. Both of them are well below its respective shear capacity. So the structure is not expected to fail in shear.

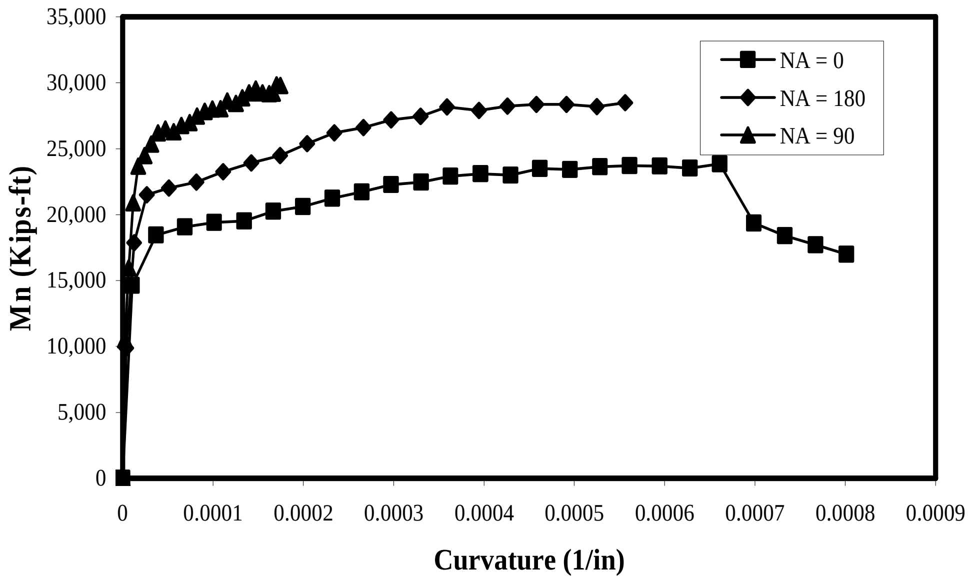

Figure 5 Moment vs curvature diagram for axial load of 1,100 kips and $f'c$ = 4,000 psi.

Regarding the maximum overturning moment around y-axis, it is 29,397 k-ft in the first scenario, which clearly exceeds the yield strength (23,683 k-ft) and almost reaches the ultimate strength of 29,808 k-ft. On the other hand, the maximum overturning moment around the x-axis is 26,239 k-ft in the second scenario, which evidently surpass the yield strength of 21,500 k-ft but, as in the previous case, do not reach the ultimate moment of 28,492 k-ft. It can be conclude that if the Control Tower were submitted to the El Centro Earthquake record it will extensively crack but it will not collapse. It should be pointed out that El Centro record seems to be a very good representation of the worst-case scenario for the Control Tower.

Table 4 Maximum Base Shear and Overturning Moment for El Centro Earthquake Record, 1940

Case	Vx (kips)	OTMy (kips-ft)	Vy (kips)	OTMx (kips-ft)
1st Scenario	405	22,730	286	13,595
	-548	29,397	-334	17,120
2nd Scenario	303	14,723	402	21,611
	-299	15,127	-509	26,239

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