Scientia Iranica, Vol. 12, No. 2, pp 241–246

SCIENTIA
IRANICA

Research Note

Reinforced Concrete Column-Supported Hyperboloid Cooling Tower Stability Assessment for Seismic Loads

S. Sabouri-Ghomi* **and M.H.K. Kharrazi**[1]

In recent years, the use of larger Reinforced Concrete (R.C.) column-supported hyperboloid cooling towers has been increased significantly. Thus, the investigation on failure criteria for structural components of such structures under different loads has been found as an essential need. Construction of cooling towers in seismic zones initiated the study on the dynamic behavior of such structures due to seismic loads. In this paper, finite element analyses have been performed to obtain the stress concentration, nonlinear behavior, stability or safety factor of the R.C. tower due to earthquake loads. Outcomes of the study show that considerable plastic hinges were created in the X shape long columns of the R.C. hyperboloid cooling tower due to seismic loads, which resulted in a significant decrease in the stability safety factor and, thus, an increase in concerns.

INTRODUCTION

R.C. hyperboloid cooling towers are generally constructed on column supports, which may have different shapes and configurations. These column shapes can be categorized, mainly, based on their geometry and length in three major groups, such as V, W and X shapes with various lengths. Based on a comprehensive literature study performed on this topic, it was found that different studies have been previously performed on W and V shape supporting columns [1-5]. In 1980, J.P. Wolf and P.E. Skrikerud studied a 144.0 m high cooling tower with relatively short V shape columns on separated foundations. The outcome of their analytical analysis showed that in the investigated cooling towers, a sliding of foundations and an uplift in columns occurred, due to severe earthquake loads [4]. Also, they introduced an optimized value for the angle of the V shaped columns. T. Castiau and R. Gaurios studied, in 1989, the seismic behavior of the Ohaaki cooling tower, due to severe earthquake loads. This cooling tower is 105.0 m high, with a base diameter of 71.5 m and short W shape supporting columns [2,3]. The results of their performed nonlinear analysis showed that the design of the W shape columns was not efficient, because different parts (shell sections) of the cooling tower became inelastic in advance of the columns. The upper part of the shell had 55% more deflection than predicted for the elastic response of the structure. Also, here, a significant uplift in the columns and a sliding of the foundation have been observed.

This paper studies the seismic behavior of R.C. hyperboloid cooling towers with relatively long X shape supporting columns. Thus, the existing RC cooling tower in Isfahan, which has similar configurations, has been selected as the case study for this paper. Isfahan is located in central Iran and is defined as a medium risk seismic zone with an expected peak design acceleration value of 0.25 g [6].

Nonlinear finite element analyses, due to two kinds of earthquake record, were conducted for these structures to obtain our goal, which is to define the stability factor of the R.C. hyperboloid cooling towers with long X shape supporting columns, due to earthquake loads. These nonlinear finite element analyses were mainly obtained by conducting a linear time history finite element analysis, due to two selected earthquake records and with the help of an in-house analysis software, the plastic hinges were defined and inserted in the system. Based on the attained plastic hinges, the model was reanalyzed for nonlinear buckling of the structure. The following contains the

*. *Corresponding Author, Department of Civil Engineering, K.N. Toosi, (Khajeh Nassir-Al-Deen Toosi) University of Technology, Tehran, I.R. Iran.*
1. *Department of Civil Engineering, University of British Columbia, Vancouver, BC, Canada.*

results of the nonlinear finite element analyses and the stability factor of the structure for a high risk seismic zone.

FE MODEL OF THE R.C. HYPERBOLOID COOLING TOWER

A vast amount of information was gathered for producing an accurate finite element model of the selected structure, the Montazeri R.C. hyperboloid cooling tower, such as material properties of the concrete and steel, the exact geometry of the structure, soil conditions, seismic site conditions and previous design and analysis information from the designer and contractors. The Montazeri R.C. hyperboloid cooling tower has a total height of 118.500 m, a span of 102.900 m in diameter on the foundation, a span of 90.600 m in diameter at the transition of columns to shell and a span of 63.530 m in diameter at the top. The total elevation from the grade for the X shaped column is 18.950 m. The columns have a dimension of 0.600 m by 0.900 m and the thickness does not vary throughout the height. The thickness of the shell varies from 1.000 m close to the columns to 0.230 m at an elevation of 28.000 m. From there, it decreases to 0.17 m at the top. The cooling tower is built on a ring strip foundation, which is 3.900 m below grade and with a width of 3.500 m and an average height of 1.200 m. A concrete stiffening ring (or upper ring) with a thickness and width of 0.300 m and 1.405 m, respectively, has been built at the top of the cooling tower. Figure 1 shows the elevation plan of the R.C. cooling tower.

The material properties for the concrete and steel are shown in Table 1. It is good to mention that the

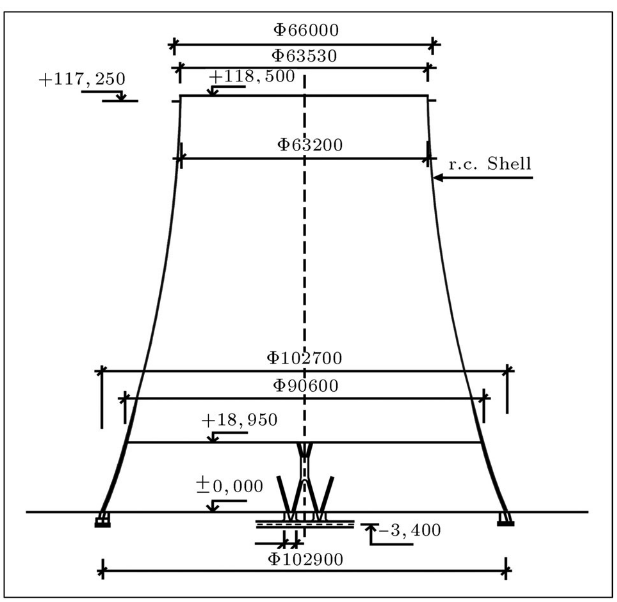

Figure 1. Elevation drawing of Montazeri R.C. hyperboloid cooling tower.

Table 1. Material properties of the R.C. cooling tower.

Material	Yield Point (MPa)	Ultimate Point (MPa)
Concrete	-	25.0 - 30.0
Steel	320.0 - 350.0	420.0 - 450.0

research later assumed different material properties for the concrete, in order to study the stability factor for such structures with different material properties.

The earthquake evaluation reports of the construction site were used to select relevant earthquake records for the study. The selected earthquake records will be introduced in the next section. Also, the soil evaluation of the site provided valuable information, which was applied for the spring characteristics of the soil-foundation model.

To perform the nonlinear analysis, two softwares were used. The first applied software, which is the commercial software, NISA II Version 7.0/NISA-CIVIL®, was used to find the response of the structure in time history linear analysis and, also, after inserting plastic hinges, to determine the stability buckling analysis [7]. The in house software, SAKNT® Version 1.1, was used to define the plastic hinges at each step and to replace the structural connection with a plastic hinge.

For the finite element model of the R.C. cooling tower, shell elements were applied for the ring strip foundation (see Figure 2 and for the concrete shell, see Figure 3). The columns were modeled with solid elements (Figure 3). The transient elements used for the intersection of the column and shell elements were shell elements, in which the degree of freedom for the corner nodes of the column were related to the central axis of the column by the help of the master and slave nodes. The central node of each column then was connected to the transient shell element. The transient elements had the capability to relate the column elements at the bottom to the shell elements at the top (Figure 3).

The finite element model of the R.C. cooling tower was analyzed for dead load with the aim of calibrating

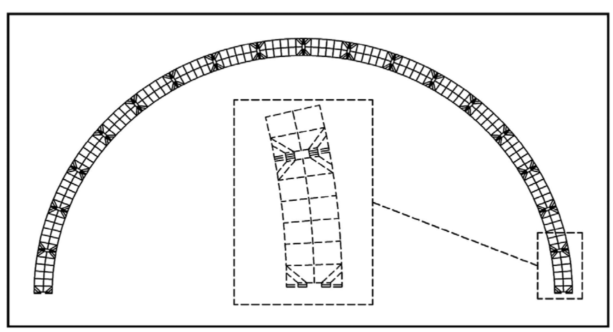

Figure 2. Plan view of the finite element model of the strip foundation.

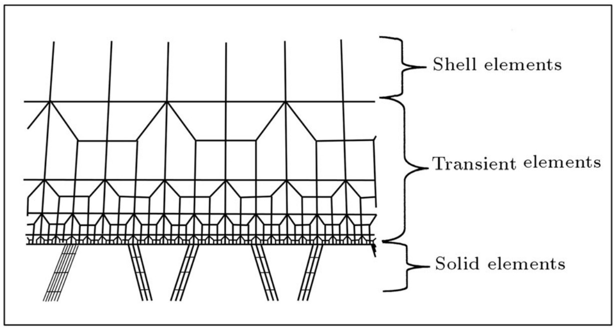

Figure 3. Transient element shown for the R.C. cooling tower model, elevation of the intersection of the column and shell elements.

the model with the previous analysis performed for the design. Thus, the obtained stresses and deflections were in full compliance with the design stress and deflection predicted by the designer.

PERFORMED NONLINEAR DYNAMIC AND STABILITY ANALYSIS

As mentioned above, two different earthquake records were selected for the structural dynamic analysis, based on the site report. These two earthquake records are the 1940 El Centro earthquake, California, USA and the Tabas-e-Golshan earthquake of the 16th of September, 1978, in Iran. The peak acceleration, velocity and displacement of the two earthquake motions are shown in Figures 4 and 5. In the same figures, the spectral acceleration, velocity and displacement for the damping ratio of 5% are shown. The SI index for the 1940 El Centro earthquake and the 1978 Tabas earthquake is defined as 35.7 and 85.9, respectively [8].

The El Centro and Tabas earthquake amplitude were normalized, so that different Peak Ground Acceleration (PGA) were applied in the dynamic analysis with the same time history. The selected magnitudes of the different peak ground accelerations were 0.35 g, 0.45 g and 0.5 g for El Centro and 0.45 g, 0.6 g and 0.65 g for the Tabas record. On the other hand, the ultimate stress for the concrete was considered as a various parameter too, in which this parameter was selected as 0.5 fc', 0.6 fc' and 0.7fc'. To minimize the amount of calculation, the lower yield stresses were matched with lower PGAs. For the El Centro input, PGA = 0.35 g, 0.45 g and 0.5 g were applied with the yield stresses 0.5 fc', 0.6 fc' and 0.7 fc', respectively,

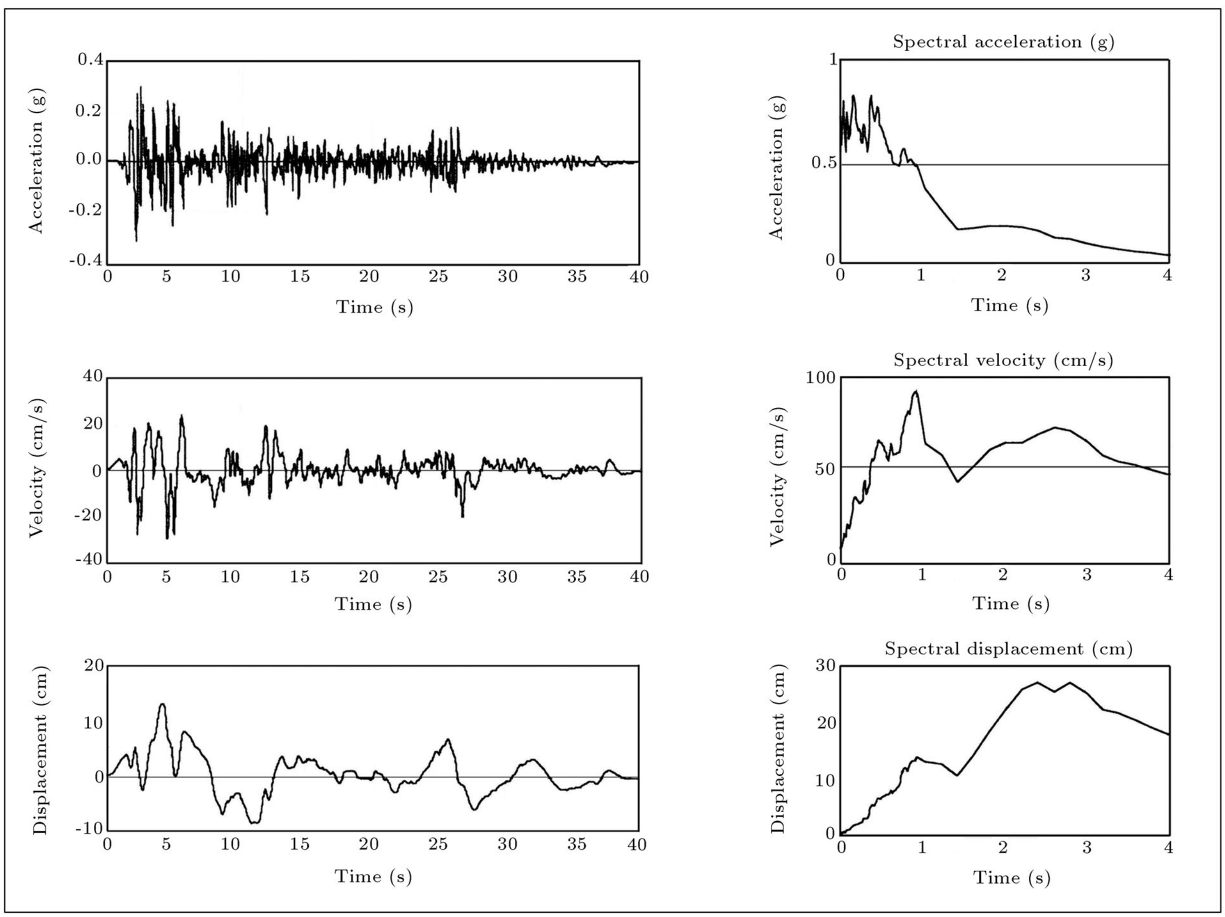

Figure 4. Time history of the 1940 El Centro earthquake record.

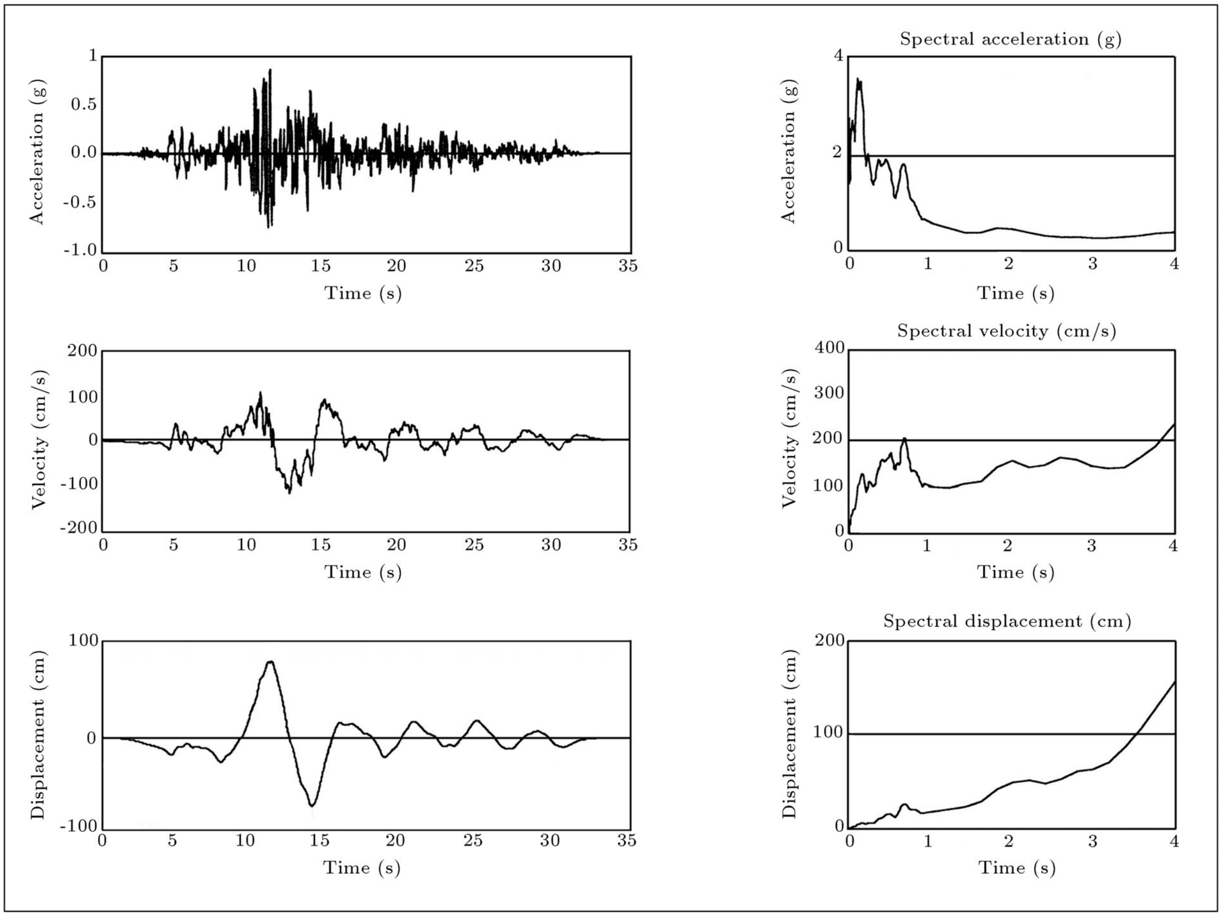

Figure 5. Time history of the 1978 Tabas earthquake record.

and for the Tabas input, PGA = 0.45 g, 0.6 g and 0.65 g were applied with the yield stresses 0.5 fc', 0.6 fc' and 0.7 fc', respectively. For each combination of PGAs and yield stresses, the stress concentrations in the structure and, specially, those in the columns, were investigated.

Time history linear dynamic analyses were conducted for each and every combination of the peak ground acceleration and yield stress. To determine the plastic hinges, the in house software, SAKNT® Version 1.1, was applied. This program checked the analysis at each step for maximum stresses, forces and moments and their time of occurrence. It also organized and arranged the maximum outputs for different members of the cooling tower in a separate file. Based on this file, the yielded locations in the members were defined for the cooling tower.

In all the analyses, no significant plastic deformations were observed in the shell elements, except close to the column connections. The outcome of the analyses for all combinations are shown in Figure 6, which shows that the first plastic hinges are formed in the columns especially close to the shell to column connection, column to foundation connections and column to column intersections. In both load and material combinations, quite a considerable amount of plastic hinges was formed in the columns. Figure 6 shows that all the combinations from the El Centro earthquake record have produced larger amounts of plastic hinges than those from the 1978 Tabas earthquake record.

In the above section, all necessary steps of the dynamic analysis were explained, which were performed to obtain the plastic hinges of the structure. The next step is to calculate the stability of the R.C. cooling tower with the given information of the formed plastic hinges. To perform the stability analysis, it was assumed that the concrete at the plastic hinges degrades due to cyclic loading of the earthquake to a level where it loses most of its capacity. In this research, the worst-case scenarios were assumed, in which, for the stability analysis, the columns with plastic hinges at their top, middle and bottom sections, will have no load carrying capacity at all.

A stability analysis was conducted for the undamaged R.C. cooling tower due to the dead load of the structure. The stability-buckling factors obtained from this analysis for the first ten buckling modes are shown in Table 2. As indicated in Table 2, the

Table 2. Stability factors for different buckling modes due to dead loads and earthquake loads.

Buckling Mode	Stability-Buckling Factor						
	Dead Load	1940 El Centro			1978 Tabas-e-Golshan		
		Fp=0.5 fc' PGA=0.35 g	Fp= 0.6 fc' PGA=0.45 g	Fp=0.7 fc' PGA=0.5 g	Fp=0.5 fc' PGA=0.45 g	Fp=0.6 fc' PGA=0.6 g	Fp=0.7 fc' PGA=0.65 g
1	15.25	0.50	0.69	1.94	1.88	2.49	3.99
2	16.21	1.95	1.55	2.69	2.36	3.93	4.03
3	17.54	2.33	2.09	3.74	2.61	4.04	4.78
4	17.75	3.16	2.15	3.88	3.99	4.86	> 5.0
5	18.54	3.24	3.29	4.82	4.03	> 5.0	> 5.0
6	20.00	3.49	3.62	> 5.0	4.82	> 5.0	> 5.0
7	20.01	3.70	3.70	> 5.0	4.94	> 5.0	> 5.0
8	20.02	4.30	3.81	> 5.0	> 5.0	> 5.0	> 5.0
9	20.03	4.47	4.21	> 5.0	> 5.0	> 5.0	> 5.0
10	20.05	4.81	4.41	> 5.0	> 5.0	> 5.0	> 5.0

stability-buckling factor is 15.28 for the first buckling mode and increases up to a ratio of 20.05 for the tenth buckling mode. Figure 7 shows the first buckling mode of the R.C. cooling tower. Since the Montazeri R.C. cooling tower was designed, based on VGB Power-Tech Codes, the suggested provisions of this code were applied in this research, in which the stability factor has been suggested as a ratio of 5.0 for these types of structure [9]. Based on the outcome for the undamaged R.C. cooling tower (Table 2), the obtained stability factor for the buckling of the tower is relatively higher than the VGB suggested safety index. This index is still considerably high when design wind load is added

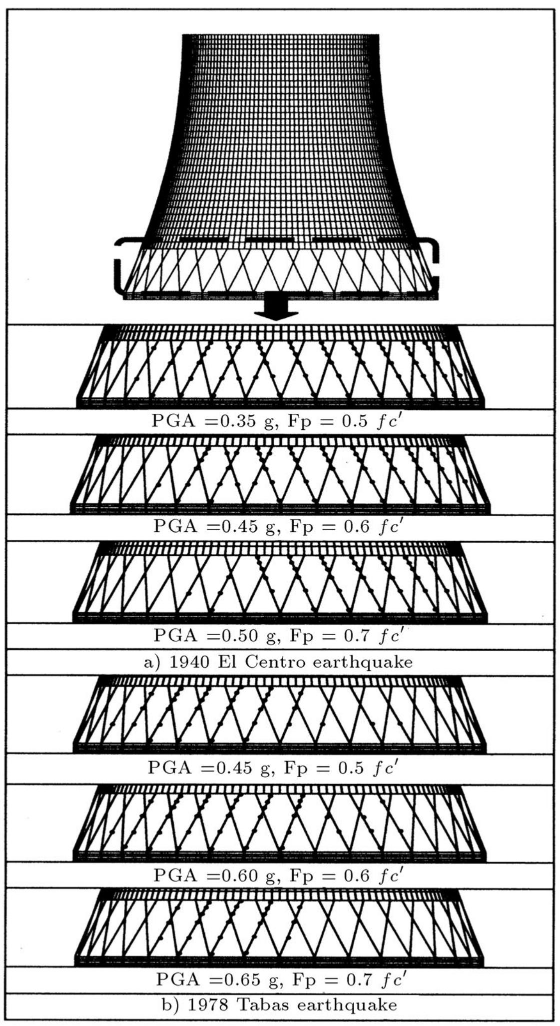

Figure 6. Plastic hinges formed in the finite element model, output of SAKNT®.

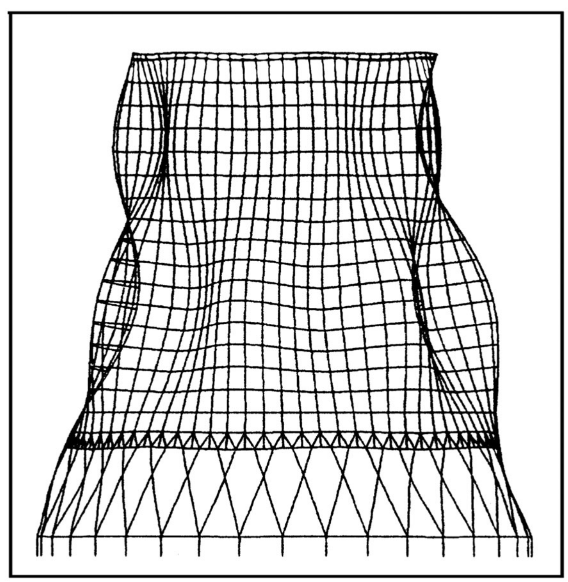

Figure 7. First buckling mode of the Montazeri R.C. cooling tower due to dead load.

to dead load in the stability analysis. The stability factor for buckling, due to wind and dead load, reduces to 11.0 for the first buckling mode.

The next step in the stability analysis is to consider the damaged cooling tower with the formed plastic hinges, due to different combinations of earthquake loads and material properties. For the combination of the El Centro earthquake time history and material properties of PGA=0.35 g, Fp=0.5fc' and PGA=0.45 g, Fp=0.6fc', the structure gets unstable for all buckling modes and the stability factor drops below 1.0 (see Table 2). For the same earthquake record, but for a combination of PGA=0.5 g, Fp=0.7fc', the structure remains stable, however, it does not fulfill the suggested code provision. For all the combinations of the 1978 Tabas earthquake, the structure remains stable for all buckling modes but does not fulfill the suggested code provision at all.

According to the utilized concrete and the seismic zone of the Montazeri R.C. cooling tower and Table 2, the structure is closely fulfilling the code provisions. Based on the outcomes presented in Table 2, R.C. column-supported hyperboloid cooling towers for high risk seismic regions have to be carefully designed. To avoid and postpone the formation of plastic hinges in the columns and to increase the stability factor, the use of relatively shorter columns for the supporting system is highly suggested, since development of plastic moment hinges are insignificant in these types of column.

CONCLUSION

In this paper, the stability factor for the buckling of the Montazeri R.C. column-supported hyperboloid cooling tower was studied, according to two earthquake records, the 1940 El Centro and the 1978 Tabas, with different Peak Ground Accelerations (PGA) and with various material properties. Based on the outcomes of the analysis performed on the calibrated finite element model, it was shown that relatively large stresses occurred in the columns, which caused a considerable number of plastic hinges to be formed close to regions, such as the shell to column connections, column to foundation connections and column to column intersections. The stability factors calculated for buckling, due to a different combination of load and material properties, revealed that the case study is fulfilling the design code provisions and that an unstable condition for high risk seismic regions does exist for such types of cooling tower in future designs.

The shell of the R.C. hyperboloid cooling towers transfers large internal forces to the supporting columns. Since the columns are the weakest components of these structures, large shear displacements have been observed between the top and bottom of the columns for the earthquake loads. This shear displacement causes the formation of plastic hinges in longer column members and great shear and moment stresses in the regions close to the column to foundation and column to shell connections [10,11]. Therefore, for high risk seismic regions, if this type of structure were to be selected for a plant, the design of the columns should be carefully performed to avoid or minimize these effects.

REFERENCES

1. Aksu, T. "A finite element formulation for column-supported hyperboloid cooling towers", *Computers & Structures*, **59**(5), pp 965-974 (1996).
2. Castiau, T. and Gaurios, R. "The design of cooling towers in extremely severe earthquake conditions", *Engineering Structure*, **13** (Jan. 1991).
3. N7S 3103, *The Design of Concrete Structures (Code of Practice Plus Commentary)* (1983).
4. Wolf, J.P., Skrikerud, P.E. "Influence of geometry and of the constitutive law of the supporting columns on the seismic response of a hyperbolic cooling tower", *Earthquake Engineering and Structural Dynamic*, **8**, pp 415-437 (1980).
5. Wittek, U. and Meisinkel, R. "Nonlinear behavior of R.C. cooling towers and its effects on the structural design", *Proceeding of the 4th International Symposium on Natural Drought Cooling Towers*, Kaiserslautern, Germany, pp 29-31 (May 1996).
6. Building and Housing Research Center, *Iranian Code of Practice for Seismic Resistant Design of Buildings*, 2nd Revised Edition, BHRC Publication, **253**, Tehran, Iran (1997).
7. NISA-II, *User's Manual, Training Manual, DISPLAYIII & Element Library.*
8. Katayama, T., Sato, N. and Saito, K. "Si-sensor for the identification of destructive earthquake ground motion", *Proceeding of the Ninth World Conference on Earthquake Engineering*, Tokyo-Kyoto, Japan (Aug. 1988).
9. "VGB-Richtlinie, guideline structural design of cooling towers", *VGB PowerTech Service GmbH, R610U, (BTR Bautechnik bei Kuhlturmen)* (1997).
10. Sabouri-Ghomi, S." State of stress and deformation in R.C. cooling towers due to unequal settlement", *International Journal of Engineering, Iran University of Science and Technology* (May 2000).
11. Sabouri-Ghomi, S. "Research, design and construction of R.C. cooling towers", *Journal of ISCE* (Nov. 1998).