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Earthquake Response of a Historical Castle

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تأثرقلعة نزوى التاريخية بهزات أرضية

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خلاصة: إن خطر وقوع زلزال في سلطنة عمان يعتبر حقيقة، إذ أن هناك عدة تقارير لهزات أرضية وقعت بين سنة 977م و سـنة 1998م ولكن دون حدوث أضرار على الإنشاءات. هناك حاجة إلى عمل خارطة زلزالية لعمان وجمع بيانات أخرى من أجـل وضـع مواصفات عامة لتصميم الإنشاءات ضد القوى الزلزالية. ففي هذه الورقة العلمية تم تقييم مدى تأثر قلعة نزوى التاريخـية بقـوى الزلزال بإستخدم المواصفات العامة للمباني (Uniform Building Code). هذا وبغياب المعلومات عن عامل المنطقة الزلزالية ومعامل الموقع فإن دراسة رد الفعل الناتج عن الهزة تمت بإفتراض قيم مناسبة لهذه المؤثرات و تشير النتائج التى توصلنا إليها إلى الحاجة إلى إجراء بعض أعمال تقوية للقلعة لتتمكن من مقاومة الزلازل المحتملة.

ABSTRACT: The earthquake hazard potential in the Sultanate of Oman is considered real as there are several reports of felt earthquakes in the Sultanate of Oman during the period from years 977 to 1998, although no damage to the existing structures has been reported due to earthquakes. In Oman, seismic zoning map and other basic data are required for the development of a standard code for seismic design of structures. In this paper, seismic response of a historical Nizwa castle in Oman is assessed employing the methodology of Uniform Building Code (UBC, 1994). Also, in the absence of data pertaining to the seismic zone factor and the site coefficient, the seismic response study is carried out by assuming appropriate values of these parameters. The results of such investigation are presented in this paper. The results show that some retrofitting measures have to be undertaken in order that the existing historical building may survive in a possible damaging earthquake.

KEYWORDS: Earthquake Records, Seismic Response, Uniform Building Code, Masonry Wall and Equivalent Static Approach.

The assessment of earthquake hazard involves collecting and evaluating a wide range of data pertaining to the history and occurrence of earthquakes in a region and to their origin. There are several reports of felt earthquakes in the Sultanate of Oman (Figure 1) during the period from years 977 to 1998 (Table 1). Although Oman does experience earthquake ground motions from time to time, no structural damage has been reported due to earthquakes. The historical records, however, suggest a possibility of such damage during the Qalhat earthquake of 15th century (Dickson, 1986). Some major events, however, that occurred at a great distance away in southern Iran, Pakistan or Yemen have been reported in Oman. For example, several individuals reported (Dickson, 1986) the effects of 27 November 1945 earthquake (M = 8.2) whose epicenter was just off the southeastern Iranian Makran Coast about 470 km from Muscat (in Oman). One witness reported damage, casualties, and a Tsunami surge. Also, 18th April 1983 earthquake event (M = 6.5) that occurred in Baluchistan was felt by some persons residing in multistory buildings in Muscat (590 km from place of earthquake occurrence) and was also felt in Buraimi (in Oman) at an even greater distance (Dickson, 1986). The Yemen earthquake (M = 5.7) of 13th December 1982 was also felt by residents in Salalah region (Southern Oman) but not in Northern Oman (Dickson, 1986).

Some faults were reported in the Batinah Coastal Plain in Northen Oman north to the Daymaniyat islands (Figure 2). Khaboura fault zone (Dickson, 1986) is located in south of Sohar about 80 km to the vicinity of Ras A’ssawadi along the coast (Figure 2). No systematic investigations concerning the seismic activity in Oman was undertaken before 1985. Earthquakes magnitudes recorded along the northeastern margin of the Arabian Plate have reached the most severe record magnitude of 8.2 (27 November 1945) (Dickson, 1986).

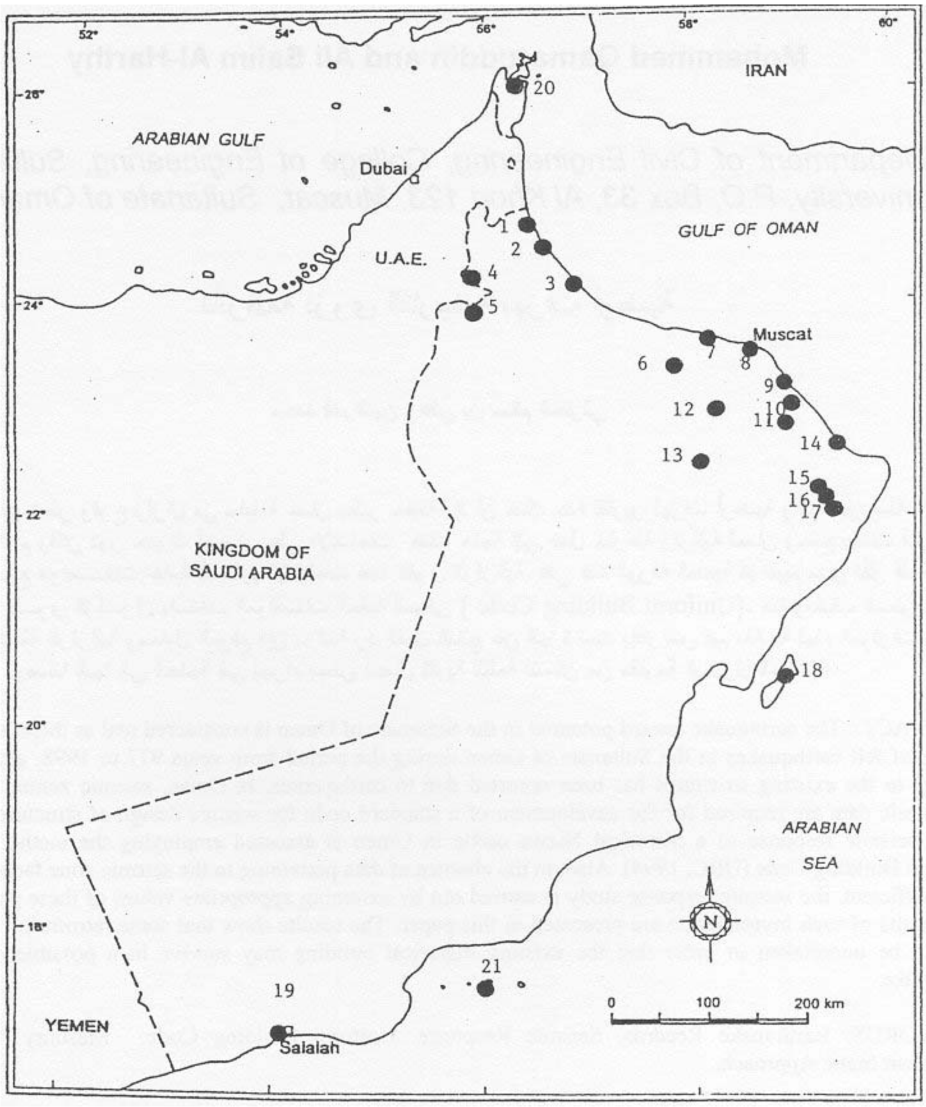

Figure 1. Felt Earthquake Locations in Oman.

The National Geophysical Data Center in Denver, Colorado (U.S.A) has some seismic data from about almost all regions of the world. It turns out from such seismic data that all the earthquakes that occurred in an area between 45.0º E to 65.0º E and 10.0º N to 30.0º N (Figure 3) show that a significant seismic activity is concentrated along the Zagrous fault zone in Iran. Also, it can be seen from Figure 3 that there are numerous epicenters along the Batinah Coastal Plain and offshore of Oman. Prior to 1980 very few earthquakes had been recorded in Oman mainly because of the lack of the local seismographic stations or sensitivity of the world seismic network or perhaps it was difficult to receive signals from events in the Oman area because of placement of seismic stations (Dickson, 1986). It is important to note that nearly all of the earthquakes after 1980 were reported by new stations in Norway, Sweden and England. The magnitude of these earthquakes ranges from 3.9 to 4.9 on modified Ritcher scale. Considerable efforts have already been made for assessing seismic hazard in the Sultanate of Oman and various other aspects of seismic monitoring in the Sultanate of Oman. Through these efforts, it turns out that several recent earth movements have occurred in Jabal Salak, Jabal Khubayb and Batinah coastal plains area (Dickson, 1986). An investigation has been undertaken to assess the response to seismic forces of an existing historical masonry structure (Nizwa castle) in the Sultanate of Oman. Seismic analysis of the existing building has been made employing the methodology of the Uniform Building Code (UBC), 1994. In the absence of data concerning the seismic zone factor and the site coefficient, the seismic response study is carried out selecting the appropriate basic data for the earthquake response computation. The results of such investigation are presented in this paper.

Methodology for Earthquake Response

The method for earthquake response determination presented in this section is based on the requirements of the UBC, 1994. The UBC is most extensively used in the U.S.A., particularly in the western part of that country. This building code is intended to provide guidelines, and formulas, which constitute minimum legal requirements for design and construction within a particular region. These requirements are intended to achieve satisfactory performance of the structure when subjected to earthquake ground motion. The safety of the structure is not assured in the event of a major earthquake. The objective of the code is that a minor or moderate earthquake will not damage the structure and that a major earthquake will not produce collapse of the structure.Whenever a building is subjected to an earthquake, its foundation will start to move randomly because of soil movement of the site under the building. Then, the whole building moves due to the movement of its foundation, which causes deformations in the building. The earthquake forces can be divided into three components: vertical and two horizontals. In the present investigation, the vertical component of the ground motion is ignored. Only horizontal earthquake motion is considered in two orthogonal directions, but considering one component at a time.

Table 1: Felt Earthquakes in Oman (Dickson, 1986)

Site	Place/City	Year(s) of Felt Earthquake(s)
1	Shinas	1935,1945,1955,1965
2	Liwa	1969
3	Saham	1944,1945
4	Buraimi	1965, 1983
5	Al Qabil	1934, 1968, 1971
6	Nakhal	1965 (2)*
7	Seeb	1983
8	Muscat	1958, 1965 (4)*, 1967, 1969, 1983 (2)*
9	Quriyat	1965 (3)*
10	Musara	1965
11	Sumayyah	1965
12	Ghugrat Al-Tam	1965 (2)*
13	Sinaw	1945
14	Qalhat	Late 15th Century
15	Al Kamil	1949
16	Al Wafi	1955
17	Bilad Bani Bu Hasan	1969
18	Masirah	1965
19	Salalah	1982, 1994
20	Musandam	977, 1184, 1483
21	Al Hallaniyat	1995

* Numbers in parenthesis indicate number of felt earthquakes in a particular year.

Building Idealization

Details of the Nizwa castle's structural system are reported elsewhere (Al-Shariqi, 1998). Load bearing masonry walls have been employed as a structural system in the castle. In view of this, the castle is idealized as a three-storied lumped-mass-stiffness shear building system. The earthquake response of the building is completely defined by the physical properties of its structural elements, such as its mass, stiffness, damping and load-displacement characteristics on one hand, and time varying accelerations introduced at its foundation support on the other hand. Thus, the evaluation of structural properties and the selection of earthquake input are the most critical factors in the earthquake response analysis. In the present investigation equivalent static approach of earthquake analysis of multistory structures is employed (UBC, 1994).

Equivalent Static Approach

This method is adopted in most of the building codes for moderately high buildings due to its simplicity and due to the fact that many structures designed on the basis of code coefficients have satisfactorily withstood past earthquakes. Static horizontal forces are applied based on the values of seismic coefficients to simulate the effects of the designed earthquake. The distribution of the shear forces along the height of the building is adopted to be similar to that obtained by dynamic analysis. Design forces specified by most codes are smaller than those indicated by dynamic elastic analysis. However, to design any building in accordance with UBC-94, many factors should be taken into account by which the building shall be designed and constructed to resist a minimum lateral seismic force applied statically and independently in the direction of each of the two main axes of the structure. The equivalent static method used in the present study is briefly described in the following paragraphs.

TOTAL BASE SHEAR FORCE: The total base shear force (V) of the structure is determined by using the relation (UBC, 1994):

$$V = \frac{ZIWC}{R_w}$$

where, W is the seismic weight of the structure, Z, the seismic zone factor, I, the occupancy importance coefficient, R_w, the structural factor and C (dynamic factor) = $1.25S/T^{0.66} \le 2.75$ in which S is the site coefficient, T is the fundamental period of the building and is equal to $C_t\,(h_N^{0.75})$ while h_N is the total height of the building in feet. C_t is the time coefficient factor which depends on the structural system of a structure to be analyzed.

DISTRIBUTION OF LATERAL FORCES: The lateral force (F_x) at a height level x above the base is determined as:

$$F_x = (V - F_t)W_x h_x / \left(\sum_{i=1}^{N} W_i h_i\right)$$

where $F_t = 0$ for $T \le 0.7$ sec or $F_t = 0.07TV < 0.25$ for $T > 0.7$ sec, N is the total number of stories above the base of the building, F_x, F_i, F_N are the lateral forces applied at level x, i and N, F_t is the portion of the base force (V) at the top of the structure in addition to F_N, h_x, h_i are the heights of level x and i above the base of the building, W_x, W_i are the seismic weights of ith level.

STOREY SHEAR FORCE: The shear at ith storey (V_x) is calculated using the formula:

$$V_x = F_t + \sum_{i=x}^{N} F_x$$

DRIFT AND STOREY LATERAL DISPLACEMENT: Drift (Δ_x) and storey lateral displacement (δ_x) are computed employing $\Delta_x = V_x/K_x$ and $\delta_x = \Sigma\Delta_x$,where, K_x is the flexural stiffness for ith storey. The code stipulates that the storey drift should not exceed ($0.04/R_w$) times the storey height or 0.005 times the storey height.

NATURAL PERIOD OF THE BUILDING: The natural period of the building is determined using Rayleigh's method.

OVERTURNING MOMENT: The overturning moment at ith level (M_x) of the building is computed by the following equation:

$$M = F_t(h_N - h_x) + \sum_{i=1}^{N} F_i(h_i - h_x)$$

HORIZONTAL TORSIONAL MOMENT: Diaphragms are considered flexible when the maximum lateral deformation of the diaphragm is more than twice the average storey drift of the associated stories. The provisions (UBC-1994) have been made for the increased shear force resulting from horizontal torsion where diaphragms are flexible. The accidental torsional moment (T_x) is $0.05DV_x$, where D is the

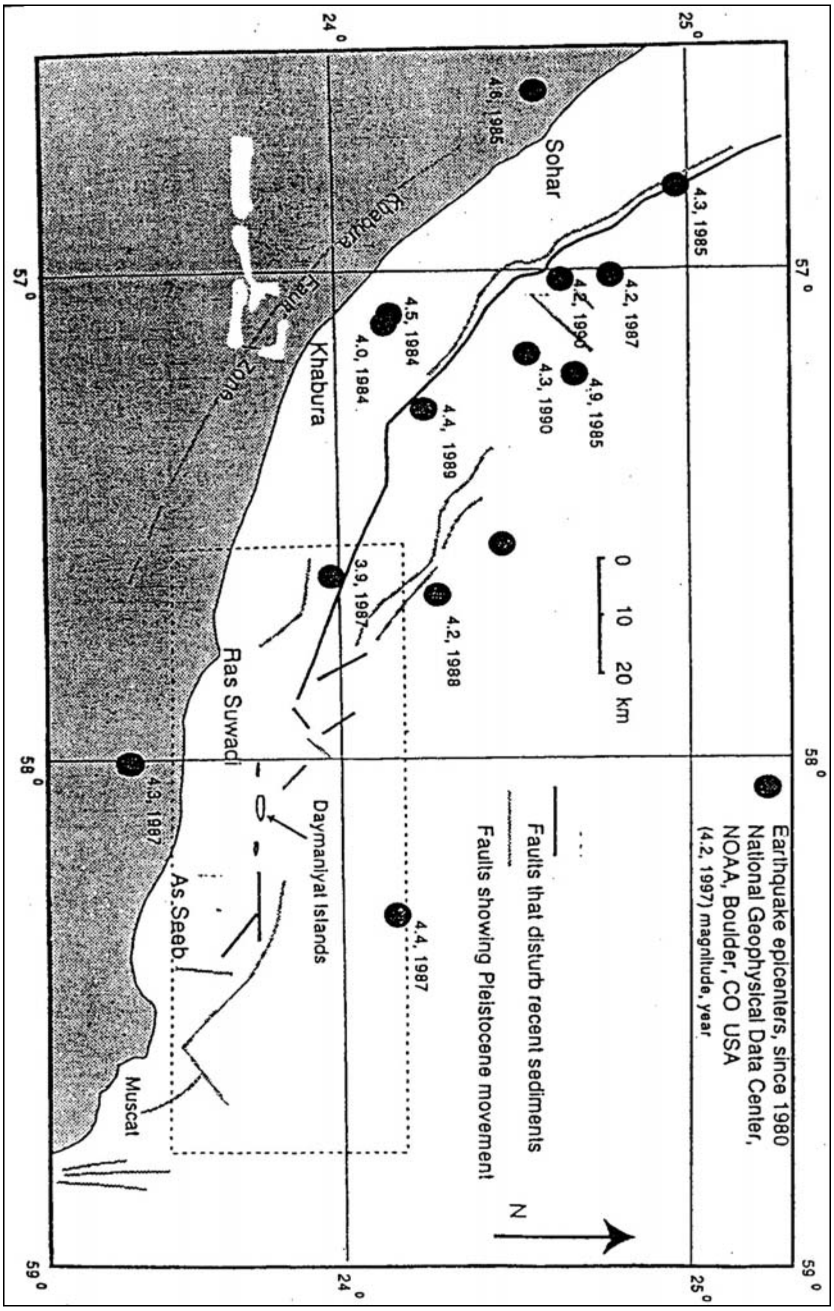

Figure 2. Epicenters and Active Faults along the Batinah Plain and Offshore in Oman.

dimension of the building perpendicular to the direction under consideration.

P-DELTA (P-Δ) EFFECT: The ratio (θ_x) of the secondary moment at ith storey (M_{xs}) resulting from P-Δ effect to the primary moment at ith storey (M_{xp}) is estimated by the following equation

$$\theta_x = M_{xs}/ M_{xp} = P_x \Delta_x/V_x H_x$$

where M_{xs} is the secondary moment at xth storey, M_{xp} is the primary moment at xth storey, P_x is the total weight at xth storey and above, H_x is the height of xth storey.

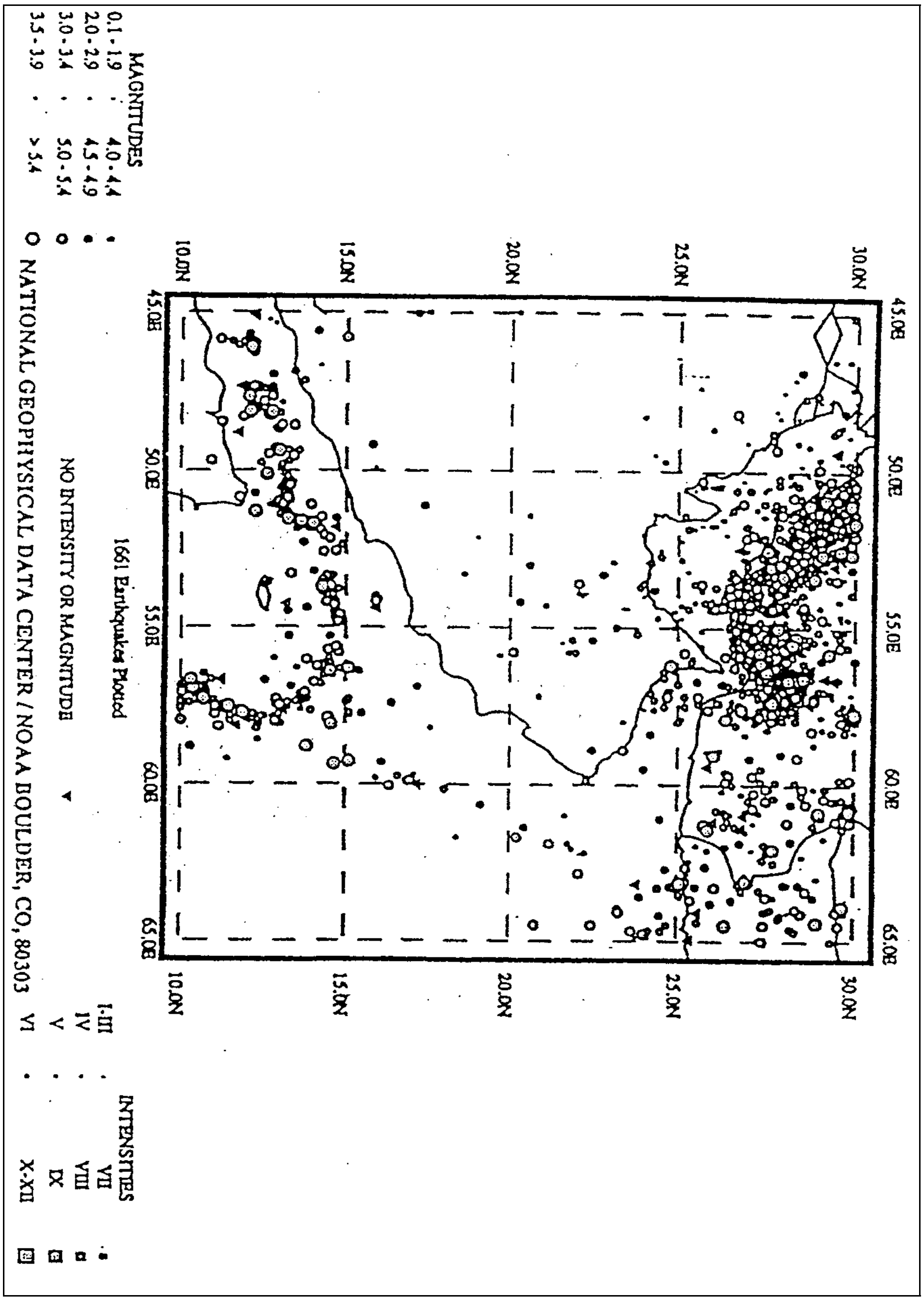

Figure 3. Seismicity in Oman since 1980.

According to the provision of the code, when θ_x at each storey level of the building is less than 0.10, there is no need to account for the P-Δ effect.

DIAPHRAGM DESIGN FORCE: The code requires that horizontal diaphragms (floors and roofs) be designed to resist the following force given by:

$$F_{px} = (F_t + \sum_{i=x}^{N} F_i) W_{px} / \left(\sum_{i=1}^{N} W_i \right)$$

in which W_{px} is the weight of the diaphragm and attached parts of the building at the ith storey. The code states that F_{px} need not exceed 0.75ZI W_{px}, but it shall not be less than 0.35ZI W_{px}.

Data for Seismic Response

In the absence of appropriate data for seismic response study, it is assumed that the Nizwa castle is located in a seismic zone equivalent to seismic zone 3 (UBC-94) and hence seismic zone factor Z = 0.3. Seismic zone 3 has been selected in view of the historical importance of the Nizwa castle. Further, the foundation soil of the castle is considered such that the site coefficient S = 1.2. The structural factor (R_w), the occupancy importance factor (I) and the time coefficient factor (C_t) are selected as 6.0, 1.0 and 0.02, respectively, in accordance with the appropriate building material and building system of the Nizwa castle.

Seismic Weight Determination

Seismic weight of the three storied Nizwa masonry castle has been determined employing its appropriate material and mechanical properties. Details of the seismic weight determination are reported elsewhere (Al-Shariqi, 1998). The results of the seismic weight computation of the masonry building are shown in Table 2.

Stiffness Determination of Shear Walls

Determination of the shear wall stiffness is reported in this section. A new methodology (Qamaruddin, 1999) has been employed in determining the wall stiffness. In this method, the wall is divided into three elements: namely pier, top spandrel and bottom spandrel. Then the three non-dimensional parameters (pier aspect ratio, top and bottom spandrel aspect ratio) are evaluated for different walls. Employing the appropriate tables (Qamaruddin, 1999), the stiffness of the walls has been determined. In the present study, the stiffness of the walls parallel to transverse (T) and longitudinal (L) directions of the building has been computed separately.

This has been done in view of the assumption that the walls located parallel to the transverse direction of the building would resist any expected ground motion occurring parallel to the transverse direction of the structure. Similar is the condition for the walls located in the longitudinal direction of the building, for the expected earthquake motion in the longitudinal direction of the building. The results for the story-wise lumped stiffness of the shear walls located parallel to the transverse and longitudinal directions of the building are also tabulated in Table 2. Details of the stiffness determination of the shear walls are reported elsewhere (Al-Shariqi, 1998).

Seismic Response of the Castle

Earthquake response has been determined using the methodology of UBC-94 for the masonry building subjected to two orthogonal horizontal ground motions separately, taking them one at a time. The results of such response computation have been shown in Tables 3 to 7.

Table 2: Data for Seismic Response Computation

Storey level	Storey height (m)	Seismic weight (kN)	Storey stiffness (kN/m) for T-shear walls*	Storey stiffness (kN/m) for L-shear walls+
3	9	87245	3502139	6931218
2	4	884063	85230520	57793623
1	4	2178666	38868586	28148832

* T-shear walls parallel to earthquake motion in transverse direction of the castle.
+ L-shear walls parallel to earthquake motion in longitudinal direction of the castle.

Results and Discussions

The Nizwa castle is subjected to earthquake ground motion in two orthogonal directions, i.e., longitudinal and transverse separately. The discussion of the results thus obtained (Tables 3 to 7) is described in the following sub-sections.

LATERAL FORCES COMPARISON:

Table 3: Lateral Force and Shear Force

Storey level	Lateral force, F_x (kN) for T-direction*	L-direction+	Shear force, V_x (kN) for T-direction	L-direction
3	30028	25970	30028	25970
2	143186	123837	173214	149807
1	176433	152591	349647	302398

* Earthquake ground motion in transverse direction of the castle.
+ Earthquake ground motion in longitudinal direction of the castle.

The results from Table 3 show that the lateral forces developed in the structure subjected to ground shaking in the T-direction are about 13.5 % larger than the corresponding lateral forces values obtained for the ground motion in the L-direction in all the stories of the castle.

COMPARISON OF BASE SHEAR FORCES: It can be seen from Table 3 that the values of the shear forces in the respective stories of the structure subjected to ground motion in the T-direction are also about 13.5 % higher than the corresponding values obtained for the earthquake ground motion in the L-direction.

COMPARISON OF STOREY DRIFTS:

Table 4: Storey Drift and Lateral Displacement

Storey level	Storey Drift (m) for T - direction	L- direction	Lateral Displacement (m) for T - direction	L- direction
3	0.008	0.0037	0.019	0.017
2	0.002	0.0026	0.011	0.013
1	0.009	0.01	0.009	0.01

From Table 4, it is seen that the values of the storey drift show different variation trends in the building as obtained by the static method. For example, the storey drift values obtained in stories 1 and 2 are respectively about 11% and 30% higher when the structure is subjected to the L-direction of the earthquake ground shaking in comparison with the corresponding values as obtained for T-direction of the ground motion. Unlike this trend, the storey drift value obtained in third floor is about 53% higher for the T-direction of the ground motion compared to the L-direction one. The above trend of storey drift variation may be attributed mainly to the variation of the storey stiffness (Table 2).

According to the UBC-94 code, the maximum permissible storey drift for the castle is determined as 0.020 m. From Table 4, it is established that the storey drifts determined in all the stories of the castle are less than the maximum permissible value of the storey drift. In view of this, the serviceability requirement of the historical castle is met

VARIATION OF LATERAL DISPLACEMENTS: It can be observed from Table 4 that the story-wise lateral displacements of the building under the L-direction of earthquake force are about 11% and 18% higher in the first and second stories respectively in comparison with the corresponding values obtained for the T-direction of the ground motion. In contrast to this observation, the lateral displacement of the third storey in the L- direction of earthquake motion is about 10.5% lower than that obtained for the T-direction.

Table 5: Overturning and Torsional Moment

Storey level	Overturning Moment (kN.m) for T-direction	L-direction	Torsional Moment (kN.m) for T-direction	L-direction
3	-	-	51648	70119
2	270252	233730	297928	404479
1	963108	832958	601393	816475
Base	2361696	2042550	-	-

COMPARISON OF OVERTURNING AND TORSIONAL MOMENTS: From Table 5, it is observed that the overturning moment in all the stories of the structure subjected to the T-direction of the ground shaking are about 13.5% larger than those obtained for the ground motion in the L-direction. This trend is true because the lateral forces developed in the structure under the T-direction of the ground motion are more than their corresponding values for the L-direction of the earthquake shaking. But, the torsional moment values in all the stories of the masonry castle are about 26.3% higher when the structure is subjected to the ground shaking in the L-direction in comparison with the response obtained for the T-direction of the ground motion.

VARIATION OF SECONDARY MOMENTS: It is observed from Table 6 that there is a different trend in the story-wise development of the secondary moment. For example, the Table 6 shows greater values of the secondary moments in the first and second stories of the masonry castle subjected to the T-direction of the ground motion than the corresponding values in the L-direction of the earthquake force. But, the secondary moment in third storey of the structure for the L-direction of earthquake is greater than the corresponding secondary moment in the T-direction.

Table 6: Primary (M_{xp}) and Secondary (M_{xs}) Moment

Storey level	M_{xp} (kN.m) for T-direction	M_{xp} (kN.m) for L-direction	M_{xs} (kN.m) for T-direction	M_{xs} (kN.m) for L-direction	Ratio of M_{xs}/ M_{xp} for T-direction	Ratio of M_{xs}/ M_{xp} for L-direction
3	270252	233730	698	323	0.026	0.0014
2	692856	599228	1943	2526	0.0028	0.0042
1	1398588	1209592	28350	31499	0.02	0.026

It is seen from Table 6 that the ratios (θ_x) of the secondary moment (M_{xs}) resulting from P-Δ effect to the primary moment (M_{xp}) in all the stories of the masonry castle are much less than 0.10. This statement is true for the castle subjected to the T- as well as L-directions of earthquake ground motion. Therefore, there is no need to account for P-Δ effect in the design of the castle because according to the provision of the UBC-94 code, θ_x at each storey level of the building should be less than 0.10.

DIAPHRAGM FORCES VARIATION:

Table 7: Diaphragm Forces (F_{px})

Storey level	Minimum F_{px} (kN)	Computed F_{px} (kN) for T-direction	Computed F_{px} (kN) for L-direction	Maximum F_{px} (kN)
3	14398	47195	40817	30853
2	44822	76126	65839	96048
1	138522	146437	126649	296833

Table 7 shows that the diaphragm forces developed in the roofing/flooring systems of the castle subject to ground shaking in the T-direction are higher than the corresponding values for the earthquake motion in the L-direction. It can also be seen from the table that the diaphragm forces developed in the third storey are more than the maximum diaphragm force permitted by the UBC-94 code. In view of this observation, it is established that the roofing system of the castle may fail in any future earthquake consistent with basic data considered in the seismic response determination of the Nizwa masonry castle.

Conclusions

Firstly, the seismic activity in the Sultanate of Oman has been studied through available literature. Then, the seismic response of the Nizwa castle subjected to the expected earthquake ground shaking has been determined. Based on the present study, the following conclusions can be drawn:

1. Since the Sultanate of Oman is situated within the seismic belt, contrary to past belief, there is a possibility of occurrence of earthquakes, especially in Batinah Coastal plain.
2. The earthquake response analysis of the three storied Nizwa castle show that there is no definite trend in the variation of different seismic responses.
3. The seismic response results may be used in the evaluation of seismic capability of the structural elements of the existing Nizwa castle. If required, strengthening measures will have to be undertaken to guard against possible failure of this structure during any future earthquake.

4. The results of the present investigation show that some retrofitting measures with respect to the roofing system has to be undertaken in order that the existing historical building may survive in any possible future damaging earthquakes.

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