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Seismic reliability of shallow footings designed using BNBC 2006

S. K. Shill and M. M. Hoque

Department of Civil Engineering
Dhaka University of Engineering and Technology, Gazipur 1700, Bangladesh

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Abstract

The paper presents the reliability of Reinforced Concrete (RC) shallow footings designed using Bangladesh National Building Code (BNBC) 2006.To achieve the objectives of the study, three model buildings with different number of stories have been designed following BNBC 2006. The bearing failure of soil, punching shear failure of concrete, flexural shear failure of footing and flexural bending failure of footing have been used for selecting the performance functions. In reliability analysis, the statistical parameters of the design variables are selected from available literatures. Monte Carlo Simulation (MCS) method has been adopted in the study to evaluate the reliability of footings. From the analytical investigation, it has been found that the reliability of footings for different failure modes is different. The reliability against bearing failure of footing is lower than the reliability against structural failure of footing. The reliability against bearing failure of soil varies from 2.29 to 2.46 for COV of soil of 40% using a factor of safety of 2.50 under earthquake load. However, the reliability of shallow footings due to flexural shear and flexural moment has been found above average accounting for the gravity loads in combination of seismic load. It is also found that the performance of RC shallow footings designed using factor of safety2.50 is *poor* under the earthquake load.

Keywords: Reliability, Monte Carlo Simulation (MCS), model building, shallow footing, performance functions.

1. Introduction

Bangladesh is an earthquake prone country. Earthquakes are always of stochastic nature. Due to existence of active faults in Bangladesh, there is a high probability of occurrence of a large magnitude earthquake (Ali and Chowdhury 1994, 1992) in Bangladesh. Therefore it is necessary to predict the probability of failure of structure and their supporting foundation due to future earthquakes. Since, uncertainties are present in different parameters accounting for the analysis and design of any structure; so, it is very difficult to measure absolute safety for any structure using deterministic analysis. Because of the presence of uncertainty in the design parameters, the structural members as well as their foundation are certainly uncertain. Therefore, one of the most important ways to specify a rational criterion for measuring the safety of a structure is its reliability or probability of failure. The reliability of a structure is its ability to fulfill its design purpose for some specified design lifetime (Nowak and Collins 2000). Reliability is often understood to equal the probability that a structure will not fail to perform its intended function. The term failure of structure does not necessarily mean catastrophic failure but is used to indicate that the structure does not perform as desired. However, engineering community, building users and building owners always expect any building or non-building structure and their supporting foundation to be designed with a reasonable margin of safety. In practices, these expectations are considered by following the code requirements. Consequently, many design codes in various parts of the world are now under revision from the allowable or the working stress design format (ASD or WSD) to the Load and Resistance Factor Design format (LRFD) based on reliability. Structural reliability concept can be applied to the design of new structure and also can be applied for calibrating codes, developing partial safety factors with an accepted level of reliability in engineering fields. Presently, Norway, Canada, United State of America, United Kingdom is following the reliability based design of structures, and other countries which are in the process of modifying their standards (Ranganathan 1999). So far, the reliability of structure and the foundation of building designed following Bangladesh National Building Code (BNBC), 2006 have not yet been evaluated. So, the principal aim of this study is to evaluate the reliability of shallow footings designed following BNBC 2006.

2. Model buildings

Three lightly loaded Garments manufacturing buildings are considered as model buildings. The location of all model buildings is considered at Zone II in context of Bangladesh. According to BNBC 2006, the building is classified as occupancy G. The structural form of model building is an intermediate moment resisting frame having RC floor panel supported by beam all sides. The beam column grid of model building is presented in Figure 1.

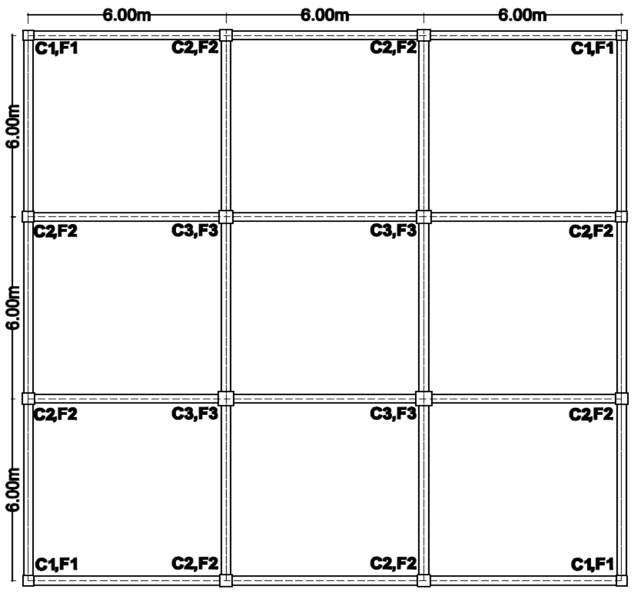

Fig. 1. Beam column grid of model building.

2.1 Building geometry

Geometries of three model building are presented in Table 1. Three model buildings are in same plan. Only number of storey is variable.

Table1
Building geometries of three model buildings

Building ID	No. of span in x-direction	No. of span in y-direction	Span length in both direction (m)	Depth of footing (m)	Typical storey height (m)	No. of storey
Model building-1	3	3	6.00	2.44	3.50	6
Model building-2	3	3	6.00	2.44	3.50	8
Model building-3	3	3	6.00	2.44	3.50	10

2.2 *Materials properties*

The Table 2 presents materials properties those are considered in the analysis and design of all model buildings of the research.

Table 2
The properties of materials

Structural elements	For all Model buildings				
	Concrete (MPa)		Reinforcing Steel (MPa)		Unit weight of concrete (kN/m^3)
Slab, beam, column, footing	*fc'*	Ec	fy	Es	
	24	23456	415	2x10^5	24

2.3 *Design of shallow footings*

In Bangladesh, Standard Penetration Test (SPT) is widely used to determine the bearing capacity of soil. So, in the research SPT data have been used for determining the bearing capacity of soil at footing level. The equations that are commonly used to determine the allowable bearing capacity of soil (Meyerhof 1965, 1974) based on 25mm of settlement have been used in the study. When (1+0.33(D/B))≤1.33 and B ≤ 1.2 m.

$$q_{all} = \frac{N'_{70}}{0.04}(1+0.33\frac{D}{B}) \tag{1}$$

When $(1+0.33\frac{D}{B}) \leq 1.33$ and B > 1.20 m

$$q_{all} = \frac{N'_{70}}{0.06}\left(\frac{B+1}{B}\right)^2 (1+0.33\frac{D}{B}) \tag{2}$$

Where, q_{all} = Allowable bearing pressure in kPa, for ΔH = 25 mm settlement, D = Depth of foundation (m), B = Width of footing (m).The size of shallow footing for gravity load is determined following the relation

$$\mathrm{A_1} = \frac{\mathrm{DL+LL}}{\mathrm{q_{all}}} \tag{3}$$

And the size of shallow footing for gravity plus effect of lateral load is determined following the relation

$$\mathrm{A_2} = \frac{\mathrm{DL+LL+EQ}}{1.33\mathrm{q_{all}}} \tag{4}$$

Where, A= Area of Footing, DL+LL = Gravity loads, EQ = Earthquake loads, q_{all} = Allowable bearing capacity of soil. Hence, greater area is selected as footing area. After the selection of width of footings from geotechnical design, the footing is needed to be designed for flexural moment; flexural shear and punching shear which is called structural design. Structural design of all footings is done following BNBC 2006. The schedule of footings is presented in Table 3.

Table 3
Footing schedule of all model buildings

Model Building	Footing ID	Footing Sizes		Footing Thickness (mm)	Depth of Footing (m)	Steel Reinforcement in both Direction
		Width (m)	Length (m)			
	F1	2.02	2.02	435	2.44	Φ16mm @ 165 mm c/c
1	F2	2.58	2.58	536	2.44	Φ16mm @ 127 mm c/c
	F3	3.05	3.05	665	2.44	Φ16mm @ 100 mm c/c
	F1	2.41	2.41	500	2.44	Φ20 mm @ 212 mm c/c
2	F2	3.09	3.09	650	2.44	Φ20 mm @ 157 mm c/c
	F3	3.67	3.67	778	2.44	Φ20 mm @ 130 mm c/c
	F1	2.74	2.74	550	2.44	Φ20mm @ 190 mm c/c
3	F2	3.48	3.48	725	2.44	Φ16mm @ 140 mm c/c
	F3	4.10	4.10	875	2.44	Φ20mm @ 112 mm c/c

3. Variability in loads

3.1 Dead load

Dead loads are typically treated as normal random variables. Generally the total dead load remains constant throughout the life of structure (Nowak and Collins 2000). In this study a coefficient of variation COV of 10 percent is assigned to dead load and distribution of dead load is considered as normal distribution (Ellingwood et al. 1980).

3.2 Live load

Live loads are always variable in nature. It is normally idealized as a uniformly distributed load. The statistical parameters of live load depend on the area under consideration. The larger the area which contributes to the live load, the smaller the magnitude of the load intensity (Nowak and Collins 2000). A coefficient of variation COV of 25 percent for the live load in office buildings fit a type I extreme value distribution (Ellingwood et al. 1980).

For the reliability analysis of shallow footing a wide range of COV for live loads in industrial building is considered in the present study.

3.3 Earthquake load

The highly variable earthquake load is considered as random variables in this research. A coefficient of variation COV of 138 percent is assigned to earthquake load and distribution of earthquake is considered as is Extreme type I (Ellingwood et al. 1980). Table 4 presents the statistical parameters of loads considered in the study.

Table 4
Statistical parameters of load

Load type	Mean to Nominal Ratio	COV%	Distribution type	References
Dead load	1.05	10	Normal	Galambos et al. 1982
Live load	1.00	25	extreme type I	Ellingwood et al. 1980
Earthquake load	1.00	138	extreme type I	Ellingwood et al. 1980

4. Variability in resistance

4.1 Compressive strength of concrete

The coefficients of variation, COV of the in situ compressive strength for concrete grades 35 and 20 MPa are estimated to be 15% and 18%, respectively (Mirza et al. 1979). Ellingwood estimated the COV to be 20.7% under average control of concrete. For the compressive strength of concrete, normal probability distribution has been found best suitable by many investigators (Mirza 1996; Mirza et al. 1979). In this study, the Coefficient of Variation COV is selected as 0.18 for 24 MPa concrete (Ellingwood et al. 1980). The mean value of this distribution is equal to $0.675f_c^{'} + 1100 \leq 1.15f_c^{'}$ (psi) where, $f_c^{'}$ is the nominal design strength (Mirza et al. 1979).

4.2 Yield strength of reinforcing steel

Different statistical distribution for the yield strength of reinforcing steel has been proposed by different researchers. The variability of static yield strength of reinforcing steel based on nominal area of the bar cross section can be represent as beta distribution (Mirza and MacGregor 1979), normal (Low and Hao 2001), lognormal (Galambos and Ravindra 1978). However, the normal distribution is more appropriate for yield strength of reinforcement at 95% confidence level (Arafah 1997).The COV for yield strength of steel equal to 8-12% (Galambos and Ravindra 1978). The mean and coefficient of variation of yield strength for 60 grade steel are 465 MPa and 9.8% (Mirza and MacGregor 1979).However in this study only 60 grade steel is considered and corresponding mean and coefficient of variation of yield strength is considered as 465 MPa and 9.8% respectively.

4.3 Bearing capacity of soil

The COV of mixed soil is 0.41 (Reese et al.1974). The COV of the inherent variability for N valueare between 25% and 50% (Phoon et al. 1995)and the probability distribution for Nis assumed to be lognormal because: (1) most soil properties can be modeled adequately as lognormal random variables (Spry et al. 1988); Phoon and Kulhawy 1999) and (2) negative values of Nare inadmissible. However, in this study, the COV of SPT is considered as 40% and the distribution of N value is considered as lognormal. Table 5 presents the statistical parameters of resistance considered in the study.

Table 5
Statistical parameters of resistance

Property	Mean	COV%	References
$f_c^{'}$=24MPa	$0.675\mathrm{f}_\mathrm{c}^{'} + 1100 \leq 1.15\mathrm{f}_\mathrm{c}^{'}$	18	Mirza et al. 1979
$f_y = 415\mathrm{MPa}$	465MPa	9.8	Mirza et al. 1979
Soil capacity	292kPa	25-50	Phoon and Kulhawy 1999a

5. Reliability analysis

The objective of the reliability analysis is to determine the probability of failure. The probability of failure p_f is the probability that the realization of the basic variables yield a point in the failure domain, i.e.$p_f=P[G(x)]\leq 0$, Where, x = vector of basic variable; and G(x) limit state function defined such that the region G(x)≤0 corresponds with the failure mode of interest. The corresponding reliability index β can be calculated from $\beta = -\Phi^{-1}(p_f)$, where, Φ^{-1} inverse of the standard normal cumulative distribution function. In this study, Monte Carlo simulations have been used to evaluate the reliability and corresponding failure probability of footings. The graphical presentation of reliability index is shown in Figure 2. Where, Q is the load effects and R is the effect of resistance.

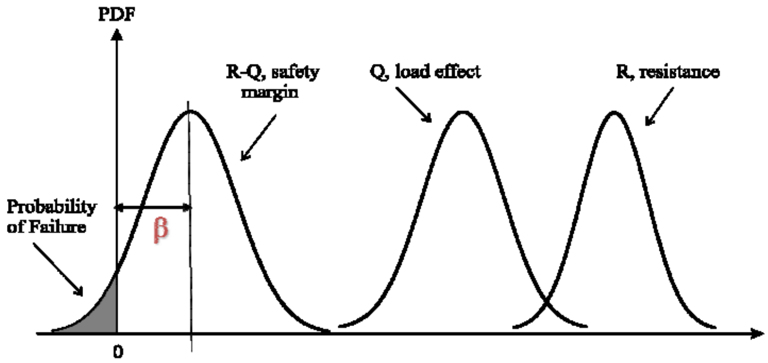

Fig. 2. Failure probability, load effect and resistance effect.

The reliability indices, β for most geotechnical components and systems lie between 1 and 5, corresponding to probabilities of failure ranging from about 0.16 to 3 × 10^{-7}, as shown in Figure 3 and Table 6 (US Army Corps of Engineers 1997).

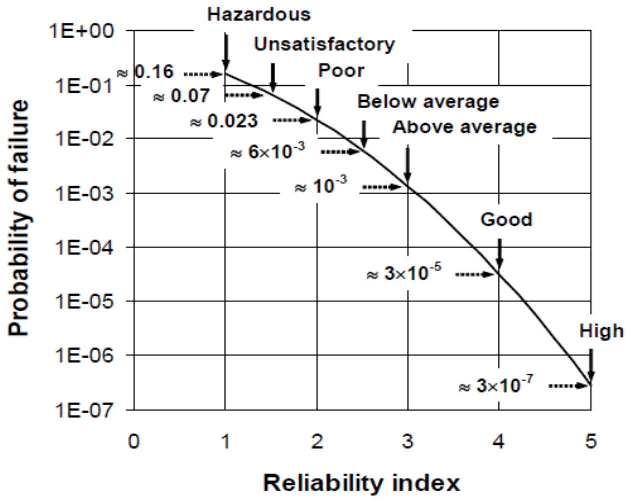

Fig. 3. Relationship between reliability index β and probability of failure p_f

Table 6
The range of geotechnical reliability index (US Army Corps of Engineers 1997).

Reliability Index, β	Probability of failure $p_f = \Phi(-\beta)$	Expected Performance level
1.0	0.16	Hazardous
1.5	0.07	Unsatisfactory
2.0	0.023	Poor
2.5	0.006	Below average
3.0	0.0001	Above average
4.0	0.00003	Good
5.0	0.0000003	High

5.1 Monte Carlo Simulation (MCS)

A reliability problem is normally formulated using a failure function $g(X_1, X_2, \dots \dots .. X_n)$, where $X_1, X_{2,} \dots \dots . X_n$ are random variables. Violation of the limit state is defined by the condition $g(X_1, X_2, \dots \dots .. X_n) \le 0$ and the probability of failure, p_f is expressed by the following expression:

$$p_f = P[g(X_1, X_2, \dots \dots \dots \dots \dots \dots .. X_n) \leq 0] \tag{5}$$

$$p_f = \int_{g(X_1, X_2, \dots\dots X_n) \leq 0} \int \dots \int f_{X_1, X_2, \dots\dots X_n} (x_1, x_2, \dots \dots . x_n) dx_1, dx_2 \dots \dots dx_n \tag{6}$$

Where, $(x_1, x_2, \dots \dots . \dots \dots \dots \dots \dots x_n)$ are values of the random variables and $f_{X_1, X_2, \dots\dots\dots X_n}(x_1, x_2, \dots \dots . x_n)$ is the joint probability density function. The Monte Carlo method allows the determination of an estimate of the probability of failure, given by:

$$p_f = \frac{1}{N} \sum_{i=1}^{N} I(X_1, X_2, \dots \dots \dots \dots .. X_n) \tag{7}$$

Where, $I(X_1, X_2, \dots \dots \dots \dots .. X_n)$is a function defined by

$$I(X_1, X_2, \dots \dots \dots \dots .. X_n) = \begin{cases} 1 & if g(X_1, X_2, \dots \dots .. X_n) \leq 0 \\ 0 & if g(X_1, X_2, \dots \dots .. X_n) > 0 \end{cases}$$

N independent sets of values $(x_1, x_2, \dots \dots . x_n)$ are obtained based on the probability distribution for each random variable and the failure function is computed for each sample. Using MCS, an estimate of the probability of structural failure is obtained by:

$$p_f = \mathrm{NH/N} \tag{8}$$

Where, N_H is the total number of cases where failure has been occurred.

5.2 *Random variables*

The nominal mean values are obtained from the deterministic analysis of model buildings. Table 7 presents the list of basic variables that are considered in the study for reliability evaluation of footings.

Table 7
List of random variables

Sl	X_i	Description	Distribution	Mean	COV	References
01	f_y	Yield strength of steel	Normal	Nominal	0.098	Mirza et.al. 1979
02	f_c'	Cylinder strength of concrete	Normal	Not nominal	0.18	Mirza et.al. 1979
03	$\sqrt{f_c'}$	A measure of concrete splitting strength	Normal	Not nominal	0.18	Mirza et.al. 1979
04	DL	Dead Load	Normal	Nominal	0.1	Ellingwood et al.1980
05	LL	Live Load	Extreme Type I	Nominal	0.25	Ellingwood et al. 1980
06	EQ	Earthquake Load	Extreme Type I	Nominal	1.38	Ellingwood et al. 1980
07	γ	Unit weight of soil	Normal	Nominal	0.10	Lee et al. 1983
08	B_f	Flexural model uncertainty	Normal	1.1	0.12	R. Lu. et al. 1994
09	B_v	Shear model uncertainty (ACI)	Normal	1.2	0.112	R. Lu. et al. 1994
10	q_u	Bearing capacity of soil based on N value	Lognormal	1.0	0.25-0.50	Phoon and Kulhawy 1999 a
11	Bv	Punching shear model uncertainty for seismic loads	Normal	1.00	0.12	Luo et al. 1995
12	Bv	Punching shear model uncertainty for gravity loads	Normal	1.65	0.27	Luo et al. 1995

5.3 *Performance function*

The loads $Q_i$and resistance R_i are treated as random variables. The limit-state functions g_i(X) for the various failure modes are formulated as $g_i(X) = R_i(X) - Q_i(x)$where $R_i$and $Q_i$denote the modal capacities and demands, respectively.

5.3.1 Bearing capacity

The performance function or limit state of interest for bearing capacity of soil can be defined as $g = (q_u - \gamma z) - \frac{P}{BxL}$, Where, $(q_u - \gamma z)$ = the net ultimate bearing capacity of soil P/(B*L) = the upward soil pressure below the base. If g < 0, the footing fails and when g ≥ 0 the footing is safe.

5.3.2 Bending moment

When a reinforced concrete isolated footing is loaded up to failure, three distinct flexural failure modes are possible. The particular failure mode that occurs is dictated by the percentage of reinforcement steel located in the tension zone. If the footing is lightly reinforced, the footing will fail due to sudden yielding of the steel which cannot carry the stress redistribution caused by the cracking of concrete; such a failure is of the brittle type, characterized by a rapid crack development. If the footing is over reinforced, the footing will fail by crushing of the concrete, also in a brittle fashion. The following two limit-state functions define analytically the conditions of light and heavy reinforcement:

$$g_1 = A_s - \frac{1.38}{f_y} bd \tag{8}$$

$$g_2 = A_s - \frac{0.85\beta_1 f_c'}{f_y}\frac{600}{600+f_y} bd \tag{9}$$

The condition $g_1<0$ corresponds to a lightly reinforced member, whereas the condition $g_2>0$ indicates an over-reinforced member, since in the latter case the tension reinforcement area A_s is larger than the balanced one. The footing is moderately reinforced otherwise, namely when the condition $\{(g_1>0) \cap (g_2<0)\}$ holds. The conditional probabilities of flexural failure given that the footing is lightly, moderately, or over-reinforced are determined respectively by using the following limit-state functions:

$$g_3 = B_f\left(1.25bh^2\sqrt{f_c'}\right) - M \tag{10}$$

$$g_4 = B_f A_s f_y\left(d - \frac{A_s f_y}{1.7f_c' b}\right) - M \tag{11}$$

$$g_5 = B_f\left(\frac{1}{3}bd^2 f_c'\right) - M \tag{12}$$

Where, M is the external bending moment produced by the upward soil pressure beneath the footing. The model factors for g_3, g_4, and g_5 should be treated as having different means and COV. However, due to the scarcity of experimental data for lightly and over-reinforced footing, the distribution parameters for the moderately reinforced case have been adopted uniformly for the three cases. This approximation is further justified by the negligible contribution of g_3 and $g_5$to the failure probability. Based on the results of the statistical studies reported by (MacGregor et al. 1983) on the resistance of reinforced concrete members, a mean of 1.10 and COV, of 0.12 have been chosen for B_f. These statistics have been adopted by (Israel et al. 1987).

5.3.3 One way shear

The performance function or limit state function for one way shear is defined as the following equation:
$g = B_v V_c - V_{crit}$, Where, $V_c = \left(0.17\sqrt{f_c'}bd\right)$ = the shear strength provided by concrete, B_v = Shear model uncertainty (ACI) factor, V_{crit} = the critical shear force developed at a distance d from the column face. V_{crit} can be obtained using the following equation:

$$V_{crit} = q_u\, B\left[\left(\frac{L-C}{2}\right) - d\right] \tag{13}$$

5.3.4 Punching shear

The performance function or limit state function for punching shear is defined as following equations:

$$g_1 = B_v 0.17\left(1+\frac{2}{\beta_c}\right)\sqrt{f_c^{'}}\, b_0 d - V_{crit} \tag{14}$$

$$g_2 = B_v 0.17\left(1+\frac{a_s.d}{b_0}\right)\sqrt{f_c^{'}} b_0 d - V_{crit} \tag{15}$$

$$g_3 = B_v 0.33\sqrt{f_c^{'}} b_0 d - V_{crit} \tag{16}$$

Where, V_{crit} = the critical punching shear force developed at a distance *d/2* from the column face. . Hence, the V_{crit} can be obtained using the following Equation,
$V_{u,crit} = (DL + LL + EQ) - q_u(c_1 + d)(c_2 + d)$.

6. Result and discussion

6.1 Reliability of footings under earthquake loads

The reliability indices of shallow footing for the flexural moment, flexural shear, punching shear, and bearing capacity of soil considering individual failure modes for gravity loads plus the effect of earthquake loads are presented in Table 8.

Table 8
Reliability of shallow footings under seismic loads

Model Building	Footing ID	Gravity Loads		EQ Load	Width of footing B	Reliability Indices for			
		DL	LL			Flexural Moment	Flexural Shear	Punching Shear	Bearing Capacity
		(kN)	(kN)	(kN)	(m)	β	β	β	β
	F1	810	354	68	2.02	3.58	4.08	3.26	2.34
1	F2	1261	635	84	2.58	3.79	4.06	3.16	2.38
	F3	1505	1150	1	3.08	4.06	4.06	3.20	2.44
	F1	1146	506	94	2.41	3.55	4.06	2.84	2.32
2	F2	1725	990	127	3.09	3.72	3.89	2.60	2.36
	F3	2005	1820	05	3.68	3.89	4.06	2.52	2.43
	F1	1491	657	137	2.74	3.43	3.89	2.81	2.29
3	F2	2192	1271	180	3.48	3.67	3.79	2.97	2.35
	F3	2537	2271	10	4.10	3.89	4.06	3.16	2.46

From Table 7, it is seen that the reliability index of all footings against bearing failure of footing is lower than other modes of footing failure. The reliability of shallow footings against one way shear and flexural moment is not critical taking the earthquake load into account. The reliability index for punching shear varies from 2.52 to 3.26 considering gravity plus earthquake loads. The reliability of footings against bearing failure of soil varies from 2.29 to 2.46 under earthquake load considering F.S = 2.50. The reliability of shallow footings decreases with the increase of earthquake loads. The reliability of corner footings under earthquake load is lower than other footings of same building. In the case of interior footings where earthquake load is lower, the reliability of footings against soil bearing capacity is approximately similar in both cases of gravity loads and under earthquake loads.

6.2 Effect of COV of soil on the reliability of footings under earthquake load

The effect of coefficient of variation (COV) of soil bearing capacity on the reliability of shallow footings under earthquake loads is presented in Figure 4.

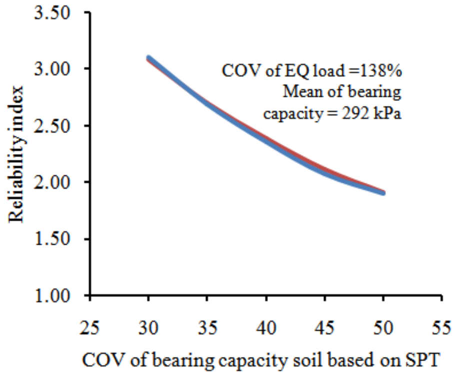

Fig. 4. Effect of COV of soil on the reliability of footings under earthquake loads using F.S=2.50

Figure 4 presents that the reliability of footing decreases with the increase of COV of soil bearing capacity. When the COV of bearing capacity of soil ≤ 30%, the reliability of shallow footing is *above average* under the earthquake loads using a F.S=2.50. On the other hand, it is also observed that if the COV of bearing capacity of soil ≥ 40%, the reliability of shallow footing is unsatisfactory to poor using a F.S = 2.50.

6.3 Effect of factor of safety on the reliability of footings

The factor of safety in calculating the allowable soil bearing capacity has greater influence on the reliability of footing. Figure 5 presents the effect of factor of safety on the reliability of shallow footings under the earthquake load.

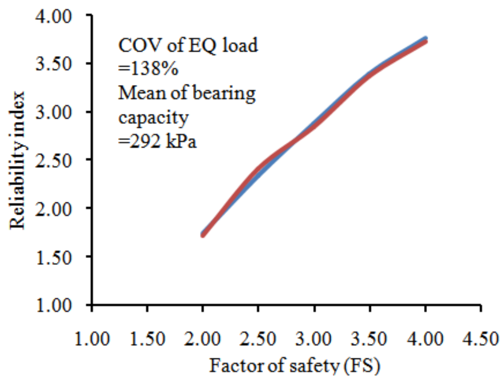

Fig. 5. Effect of factor of safety on the reliability of footings under earthquake load.

The reliability of footings against bearing failure of soil increases with the increase of factor of safety of soil. However, the performance of shallow footing under earthquake load is poor if a factor of safety 2.50 is used. The performance of footing is below average when factor of safety is 3.00. To ensure the performance of shallow footing above average under earthquake load, a factor of safety at least 3.50 should be used in footing design.

6.4 Effect of COV of live load on the reliability under earthquake load

The COV of live load for lightly loaded industrial building (occupancy G) is unknown in context of Bangladesh. Therefore, a wide range of COV of live load is considered to determine the effect of COV of live load on the reliability of shallow footing under earthquake load. From the results of the analytical investigation presented in Figure 6 and Figure 7, it is seen that the reliability for both bearing failure of soil and punching shear is decreasing with increasing of COV of live load. The reliability index for punching shear varies from 2.52 to 3.26 under earthquake loads depending on the COV of live load.

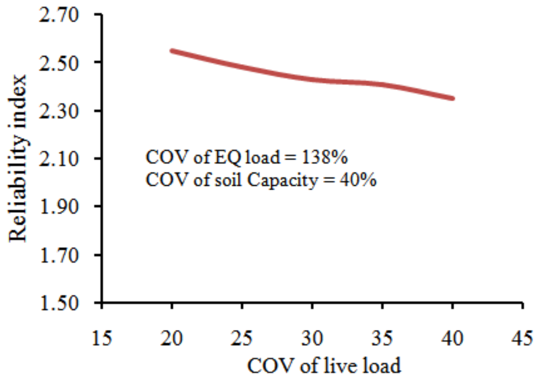

Fig. 6. Effect of COV of live load on the reliability of footings under earthquake load using F.S = 2.50

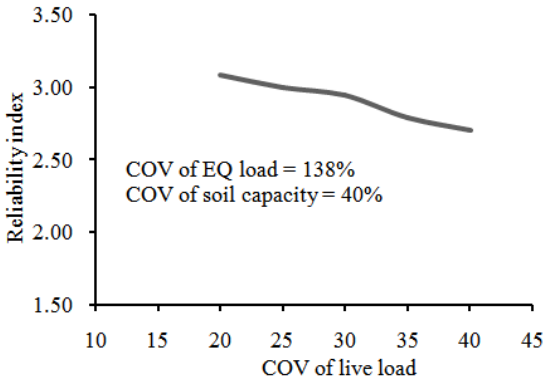

Fig. 7. Effect of COV of live load on the reliability of footings against punching shear failure under earthquake loads using a F.S=2.50

It is also observed that the reliability against bearing failure of soil varies from 2.29 to 2.46 under earthquake load depending on the COV of live load. In the case of interior footing where the intensity of live load is large, the reliability against punching shear and also the reliability against soil bearing decreases highly with the increase of COV of live load.

6.5 Effect of COV of earthquake load

The COV of highly varied earthquake load is also so far unknown in context of Bangladesh. Therefore, a wide range of COV of earthquake load is considered to evaluate the effect of COV of earthquake load on the reliability of shallow footings under the constant COV of soil capacity.

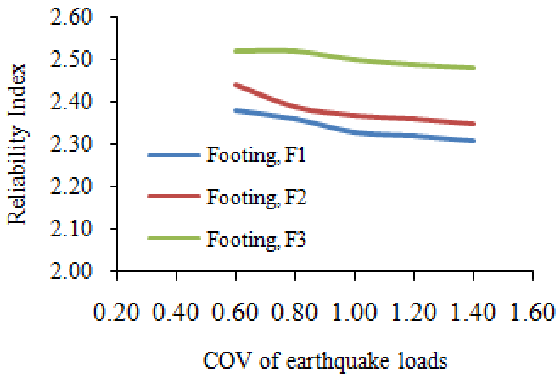

Fig. 8. Effect of COV of earthquake load on the reliability of footings using COV of soil capacity of 40% for Model building-1 using a F.S = 2.50

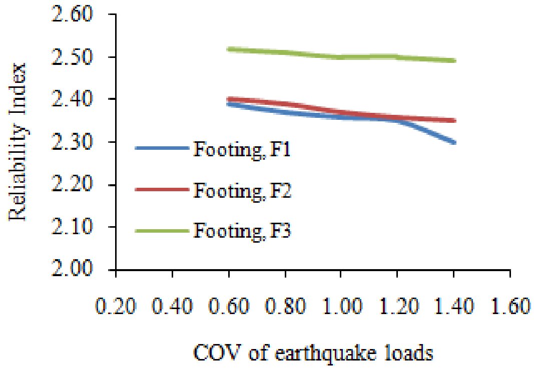

Fig. 9. Effect of COV of earthquake load on the reliability of footings using COV of soil capacity of 40% for Model building-2 using a F.S = 2.50

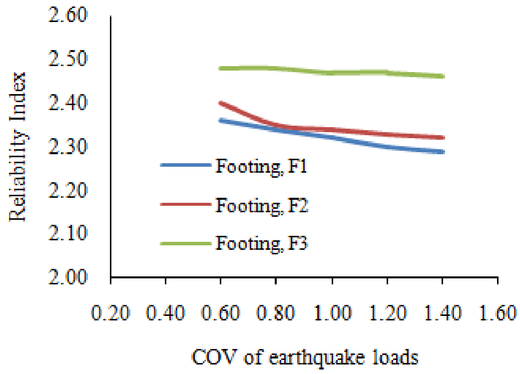

Fig. 10. Effect of COV of earthquake load on the reliability of footings using COV of soil capacity of 40% for Model building-3 using a F.S = 2.50

From Figure 8, Figure 9 and Figure 10 it is observed that the reliability of shallow footings against bearing failure of soil decreasing with the increase of COV of earthquake load. It is also observed that the reliability of corner footings is lower than reliability of exterior and interior footings in case of all model buildings under earthquake load.

7. Conclusion

The reliability of shallow footings for different failure modes is different. The reliability against bearing failure of soil is lower than that of other failure modes of footing. Reliability index against punching shear failure of concrete is lower than any other modes of structural failure of footings. The reliability of footings against flexural shear and flexural moment is above average under the effect of earthquake loads. The reliability of shallow footings also decreases as the live load to dead load ratio increase. Considering all types of failure modes of footings, the reliability index varies from 2.29 to 2.46 under earthquake load.

However, the performance of shallow footings designed using BNBC 2006 is poor under the effect of earthquake load if a factor of safety 2.50 is used. The reliability index against punching failure of footing varies from 2.52 to 3.26 under earthquake loads. It is also seen that the reliability of corner footings under earthquake load is lower than other footings of same building. In the case of interior footings where earthquake load is very lower or negligible, the reliability of footings against bearing failure of soil is approximately similar in both cases of gravity loads and for earthquake loads.

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