The Strength Reduction Factors for Reinforced Concrete Design Standards Based on Thailand Statistical Data

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บทคัดย่อ

ค่าของตัวคูณลดกำลัง ที่แนะนำในมาตรฐานสำหรับอาคารคอนกรีตเสริมเหล็กโดยวิธีกำลัง วสท 1008-38 เป็นค่าที่อ้างอิงมาจากมาตรฐาน ACI 318-89 ของประเทศสหรัฐอเมริกา ซึ่งค่าตัวคูณลดกำลังเหล่านี้ได้มาจากการวิเคราะห์ข้อมูลทางสถิติ ของการกระจายของคุณภาพวัสดุและมาตรฐานการก่อสร้างของประเทศสหรัฐอเมริกาซึ่งแตกต่างจากประเทศไทย ดังนั้นหากมีการศึกษาข้อมูลดังกล่าวสำหรับการก่อสร้างในประเทศไทย และได้นำมาใช้เป็นตัวกำหนดถึงค่าของตัวคูณลดกำลังสำหรับประเทศไทยเองโดยเฉพาะ ก็ย่อมจะมีความเหมาะสมมากกว่า

ในปัจจุบันได้มีการเสนอให้แบ่งการใช้ตัวคูณลดกำลังออกเป็นสองกรณีดังนี้ กรณีที่ 1 คือกรณีการก่อสร้างที่มีการระบุมาตรฐานงานก่อสร้างและการควบคุมคุณภาพวัสดุเป็นอย่างดี ให้ใช้ค่าตัวคูณลดกำลังเหมือนในมาตรฐาน ACI318-99 ส่วนกรณีที่ 2 คือกรณีการก่อสร้างที่ไม่มีการระบุฯ ให้ใช้ค่าตัวคูณลดกำลังในอัตราส่วน 5/6 เท่าของที่ใช้สำหรับกรณีที่ 1 อย่างไรก็ตามอัตราส่วนนี้ ไม่ปรากฏถึงที่มาอันเป็นกระบวนการทางวิทยาศาสตร์ หรือหลักฐานซึ่งแสดงถึงความเที่ยงตรงของค่าอัตราส่วนดังกล่าวแต่อย่างใด

บทความนี้ได้กล่าวถึงขั้นตอนอย่างย่อในการวิเคราะห์ความเชื่อมั่นของโครงสร้าง และผลการวิเคราะห์สำหรับการเลือกตัวคูณลดกำลังที่เหมาะสม โดยอาศัยข้อมูลทางสถิติของคุณภาพวัสดุและการก่อสร้างในประเทศไทยจากงานวิจัยอื่น เพื่อนำไปใช้สำหรับกรณีที่ 2

จากการศึกษาครั้งนี้ได้ค่าตัวคูณลดกำลังสำหรับโมเมนต์ดัดในคาน 0.80 และเฉือนในคาน 0.87 และแรงตามแนวแกนในเสาปลอกเดี่ยว 0.62 ซึ่งแตกต่างไปจากค่าที่กำหนดไว้ในกรณีที่ แต่เนื่องจากค่าดังกล่าวได้มาจากข้อมูลที่จำกัด ดังนั้นจึงเสนอแนะให้หาข้อมูลเพิ่มเติม เพื่อใช้ในมาตรฐานการออกแบบคอนกรีตเสริมเหล็กสำหรับประเทศไทยต่อไปในอนาคต

คำสำคัญ : ตัวคูณลดกำลัง มาตรฐานออกแบบคอนกรีตเสริมเหล็ก การจำลองมอนติคาโล การวิเคราะห์ความเชื่อมั่นของโครงสร้าง

Abstract

The strength reduction factors recommended in the reinforced concrete design standard EIT 1008-38 were adopted from the American ACI318-89 code. These factors were based on the analyses of statistical material and construction quality data collected in USA which may differ from Thailand. It will be more appropriate if these strength reduction factors are selected based on the data collected in Thailand.

Nowadays, two sets of strength reductions factored were recommended in the draft building codes: case 1 when good quality of the materials and construction were specified. In this case, the strength reduction factors were totally the same as in the ACI318-99 code. Case 2 when good quality of the material and construction used were not specified. In the latter, 5/6 times of the strength reduction factors recommended in case 1 were used. However, there is no any scientific proof or evidence of the accuracy of this number 5/6.

This paper presents brief process and results of structural reliability analyses to select the appropriate strength reduction factors based on the statistical material and construction data collected in Thailand for the case 2 from the other research.

From this study, the chosen strength reduction factors are 0.80 for beam flexure, 0.87 for beams shear and 0.62 for tied column axial. These factors were found to be different from those recommended for draft building codes for case 2. However, dues to the limited numbers of data available, it is suggested that more study must be conducted to ensure the correctness of these factors before any adoption to Thailand building codes.

Keywords: Strength reduction factors, Reinforced concrete design standard, Mote Carlo simulation, Analyses of structure reliability.

Introduction

All The strength reduction factors recommended in the reinforced concrete design standard EIT 1008-38 (Engineering Institute of Thailand Committee, 1995) were adopted from the American ACI318-89 code (ACI Committee 318, 1989). These factors were selected based on the analyses of statistical material and construction quality data collected in USA. It will be more appropriate if these strength reduction factors are based on data collected in Thailand.

Research Significance

This paper presents the selecting of the appropriate strength reduction factors for Thailand reinforced concrete standard based on statistical material and construction data collected in Thailand. These data were collected from single house residential buildings. According to the author's knowledge, there is no pre-existing research other than those related to the author for the strength reduction factors based on Thailand statistical data.

Concept of Selecting the Strength Reduction Factors

In the design process of any structures, one must ensure that the structure can withstand all calculated loads. Any structure will be safe if its resistance R is greater than its load effect Q. When its resistance R is less than its load effect Q, the structure may fail. The limit function $R-Q=0$ signifies the boundary between the safety and failure. Both load effect and resistance are not the deterministic. Figure 1 shows the distribution of the load effect Q and the resistance R.

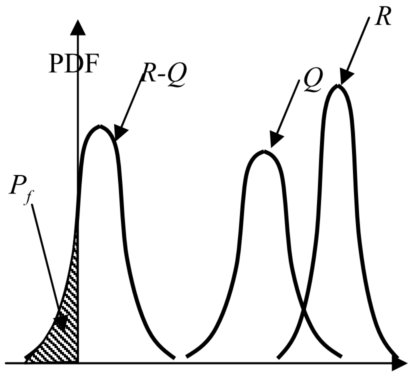

Figure 1 Probability density function (PDF) of *Q, R* and *R-Q*

From figure 1, the average of the resistances R normally larger than those of the load effects Q since the designer must include the design margins for the sake of safety. The left side area under the curve where the limit state $R-Q$ is negative (hatched area) signifies the probability of failure P_f. If both load effect and resistance are normally distributed, the relationship between reliability index β and the probability of failure P_f is known.

Figure 2 shows the definition of reliability index β in reduced variable space. Variables Z_R and Z_Q are defined as normalized resistance and load effect. The reliability index β is the shortest distance between point of origin $(0,0)$ and the limit state function $g(Z_R, Z_Q) = 0$.

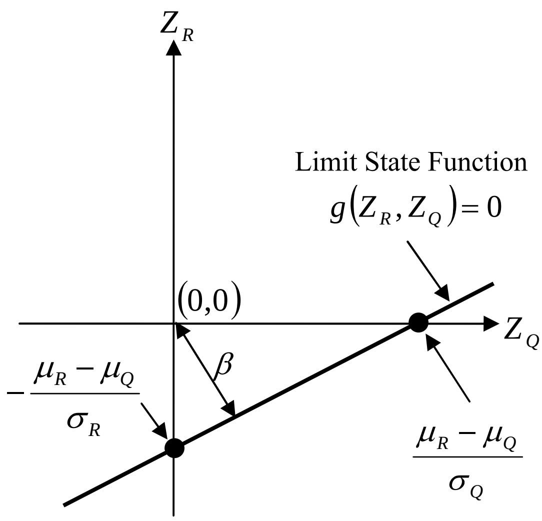

Figure 2 The definition of the reliability index β

From this definition, the reliability index can be calculated by equation (1).

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \quad (1)$$

Nowak and Szerszen (2003) and Szerszen and Nowak (2003) gave the most updated target reliability indices based on the recalibration of ACI318 shown in table 2.

Table 2 Reliability indices based on the recent recalibration (Szerszen and Nowak, 2003)

Types of member and limit state	β
RC beam cast-in-place flexure	3.54
RC beam cast-in-place shear	3.95
RC Tied column cast-in-place	3.98

Reliability analyses and the selection of strength reduction factors

Three different types of structural members/limit states were considered including (1) RC beam flexure (2) RC beam shear and (3) RC tied column axial. The resistances of these three types were Monte Carlo simulated based on the statistical distribution of concrete strengths (Suwaannarat Fuktong et. al., 2004) (Surachai Suchiwaan et. al., 2006) (Muksumna Karengsana et. al., 2006) and rebar strengths (Apidet Tannpisarn et. al., 2005) and also member sizes and rebar locations (Bandit Kongsomkit et al., 2008) (Suwit Kawaan et. al. (2009). These are available data collected by students under supervision of the author and another faculty member. Due to a very limited space allowed, the detail of data collection must be omitted but the reader can search them from the given references. For each data set, fit distribution was performed to select the closest distribution type from 22 available standardized distributions based on least Chi-square χ^2 criterion.

The reliability analysis of structural members was performed. The steps are listed below.

(1) The recalibrate reliability indices listed in Table 2 were chosen as the target reliability indices since they are the most recent. The target reliability indices should be independent of the country because they represent the safety level.

(2) Trial select ϕ factors. In this paper, all ϕ factors between 0.700 and 0.900 with 0.025 increments were included. Then, the nominal resistances R were calculated based on these trial ϕ factors. The load factors: dead load factor 1.4 and live load factor 1.7 were applied as recommended by Thailand EIT1008-38 (Engineering Institute of Thailand, 1995). Therefore, the nominal resistance R_n was calculated using equation (2).

$$R_n = \frac{1.4D + 1.7L}{\phi} \qquad (2)$$

It should be noted that R_n is the nominal value of the resistance. The real resistance R is randomly distributed depending on the distribution of the material strengths, member sizes and rebar locations.

(3) Run Monte Carlo simulations using @Risk for Excel program (Palisade Corporation, 2008). Simulate the load effect $Q = D + L$ using the statistical data shown in table 3 assuming all the loads are normally distributed. The magnitude of dead load D and live load L can be back calculated from equation (2) for a given $D/(D+L)$ ratio. The load statistics from Szerszen and Nowak (2003) were used since there was no load survey data available at the time of this analysis.

Table 3 Statistical parameters for load component (Szerszen and Nowak, 2003)

Load component	Arbitrary-point-in time		Maximum 50-year load	
	Bias	COV	Bias	COV
Dead load (cast-in-place)	1.05	0.10	1.05	0.10
Live load	0.24	0.65	1.00	0.18

From the simulation of the load effect Q, the statistical parameters:

mean μ_Q and standard deviation σ_Q were calculated.

(4) Using @Risk for Excel program to run another set of simulations, to simulate the whole range of member sizes and reinforcement ratios. The distribution of the resistance R can be calculated based on the strength formulation. Then, their means μ_R and standard deviations σ_R were also known.

(5) The distribution of the resistance R from step (4) was not accounted for any error within the design formulas. Therefore, μ_R and σ_R must be adjusted based on professional factors. Different professional factors used are listed in Table 4. These professional factors from Ellingwood et al. (1980) are appropriate since Thailand design formulas are identical to ones recommended by the ACI codes.

Table 4 Professional Factors (Ellingwood et al. 1980)

Member type/limit state	Bias factor λ_P	COV V_P
Beam flexure	1.02	0.06
Beam shear	1.075	0.10
Tied column axial	1.00	0.08

The adjustment formulas are based on the mathematical properties of each statistical parameter as the following.

$$\mu_R^* = \lambda_P \times \mu_R \quad (3)$$

$$\sigma_P = \lambda_P \times V_P \quad (4)$$

$$\sigma_R^* = \sqrt{\sigma_R^2 + \sigma_P^2} \quad (5)$$

Where μ_R^* is adjusted mean of the resistance. σ_R^* is adjusted standard deviation of the resistance. λ_P is bias factor for professional factor. V_P is coefficient of variation (COV) of the professional factor.

(6) From μ_Q, σ_Q, μ_R^*, σ_R^* using the ϕ factors which are trial selected from step (2), calculate the reliability indices β using First-Order Second-Moment method (FOSM) (Nowak and Collins, 2000).

$$\beta = \frac{\mu_R - \mu_Q^*}{\sqrt{\sigma_R^2 + \sigma_Q^{*2}}} \quad (6)$$

(7) Plot different reliability indices β values from step (6) and compare with the target reliability index β_T from Table 3. The ϕ factors which gives the closest β were chosen.

The Simulations

Since the procedure for Monte Carlo simulation is standard, only brief explanation will be discussed. Monte Carlo simulation is usually used when the numbers of the samples are limited or the data collected are not cover the whole scenario of the problem. For example, the statistical data of the strength of concrete f_c' were collected from different construction sites than the one the column sizes were collected because it is sometime impossible to collect the concrete strength data from that particular column. In order to make it possible to determine the column resistance, all the related data were collected from different sources. Then, to calculate the resistance, the simulation is needed. For each scenario, the resistance is calculated using the randomly generated data based on the strength formula. The more number of scenario simulated, the closer to the real distribution it represents. Figure 3 shows the schematic of the column axial resistance simulation.

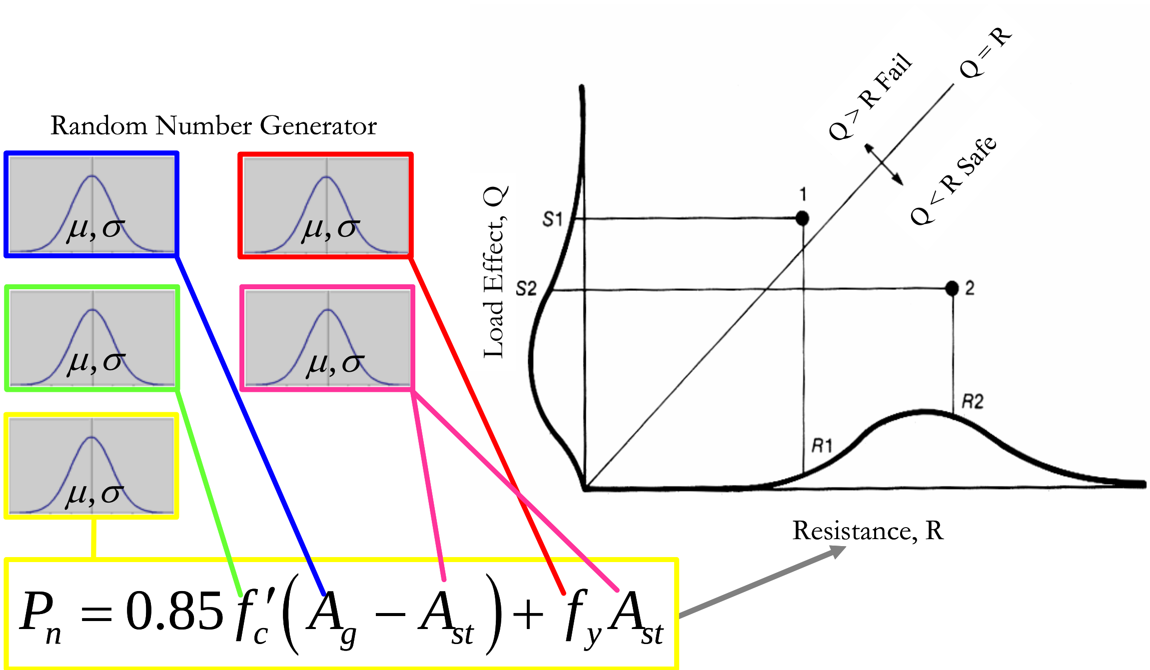

Figure 3 The schematic of the column axial resistance simulation

The Analysis Results

Because the load factors for dead and live loads are different. The ratio between these two loads could affect the final results. Therefore, different load ratios must be considered. The practical range for load ratio $D/(D+L)$ for beam is between 0.3 and 0.7 and for column is between 0.4 and 0.9.

From the distributions of dead and live loads, the distribution of the load effect $Q = D + L$ was Monte Carlo simulated. Their mean and standard deviations are shown in table 5.

Table 5 Mean and standard deviation of the simulated load effect Q

$D/(D+L)$	μ_Q	σ_Q
0.3	0.6311	0.0779
0.4	0.6418	0.0755
0.5	0.6612	0.0654
0.6	0.6759	0.0627
0.7	0.6930	0.0601
0.8	0.7108	0.0619
0.9	0.733	0.0658

Fit distribution were performed for member sizes, rebar locations and material strengths. Table 6 shows the types and parameters of their fitted distributions.

Reliability Analysis for Beam Flexure

For beam flexure, the simulated resistance were calculated by equations (7) and (8) from ACI318-99 code (ACI Committe, 1999).

$$M = A_s f_y \left(d - \frac{a}{2} \right) \quad (7)$$

$$a = \frac{A_s f_y}{0.85 f_c' b} \quad (8)$$

Where M is the flexural resistance. $A_s f_y$ is the product of rebar cross sectional area and yield strength of flexural steel. The distribution of DB12 and DB16 (SD30) together was used. d is the beam effective depth.

Table 6 Type and parameter of fitted distribution

Data	Type and parameters
Beam width	Logistic (1.006688,0.015905)
Beam effective depth	Logistic (1.030329,0.026305)
Beam stirrup spacing	Log Logistic (0.85984,0.16387,8.5798)
Column size	Extreme Value (1.007680,0.010402)
Yield force RB6(SR24)	Log Logistic (0.97134,0.48678,3.6821)
Yield force RB9(SR24)	Logistic (1.39777,0.15191)
Yield force DB16-20(SD30)	Weibull (15.493,4.3796,Shift(-2.7293))
Concrete strength (150 ksc)	Extreme Value (0.93905,0.32647)

b is the beam width. All different beam sizes were simulated as per table 7

The simulated and professional factor adjusted means and COV's for beam flexure are listed in table 8.

From means and standard deviations calculated from equations (3) to (5), the reliability indices β's were calculated and plotted in figures 4 to 7.

From figures 4 to 7, the values of ϕ that give the reliability indices β closest to the target β_T for each

Table 7 All simulated beam sizes for beam flexure and beam shear

Depth (cm)	**Width (cm)**
30	15
	20
40	20
	25
50	20
	25
	30
	35

Table 8 Statistical parameters of simulated and professional factor adjusted beam flexure resistance

Reinf. ratio ρ	Simulated		Professional factor adjusted	
	Mean μ_R	COV V_R	Mean μ_R^*	COV V_R^*
$\rho_{\min}$	1.5003	0.3236	1.5303	0.3291
$0.25\rho_b$	1.4849	0.3168	1.5146	0.3224
$0.50\rho_b$	1.4112	0.3043	1.4394	0.3102
$0.75\rho_b$	1.3241	0.3338	1.3506	0.3391

reinforcement ratio were chosen. It was decided to use the mid-range load ratio $D/(D+L) = 0.5$ to represent the whole practical load range. All chosen ϕ values for different reinforcement ratios were listed in table 9.

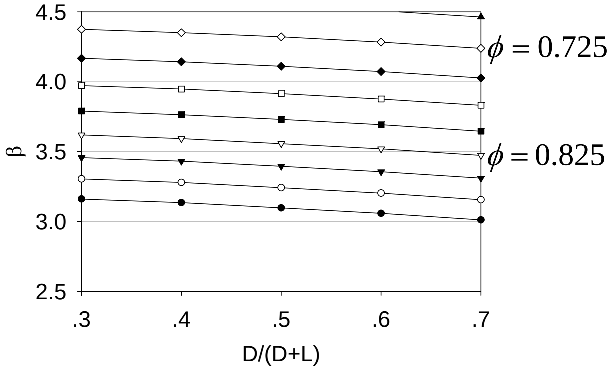

Figure 4 Reliability indices β for beam flexure ($\rho = \rho_{\min}$)

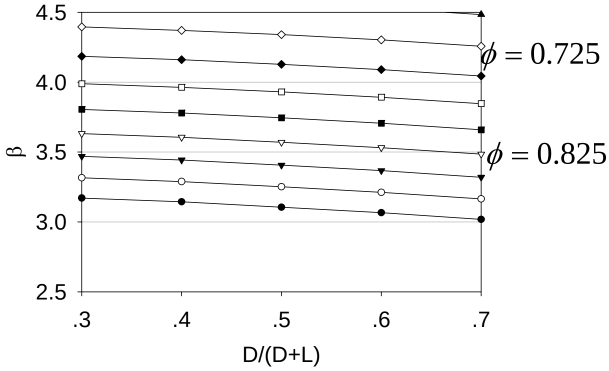

Figure 5 Reliability indices β for beam flexure ($\rho = 0.25\rho_b$)

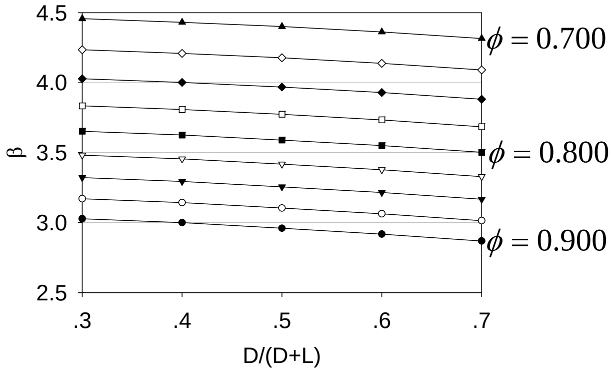

Figure 6 Reliability indices β for beam flexure ($\rho = 0.50\rho_b$)

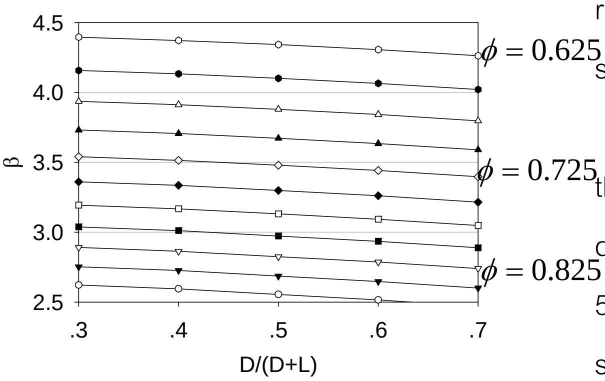

Figure 7 Reliability indices β for beam flexure ($\rho = 0.75\rho_b$)

Table 9 Selected ϕ factors for beam flexure.

Reinf. ratio ρ	ϕ at $\beta_T = 3.54$
$\rho_{\min}$	0.825
$0.25\rho_b$	0.825
$0.50\rho_b$	0.800
$0.75\rho_b$	0.725
Average	0.794

Reliability analysis for beam shear

Beam shear resistances can be predicted using equations (9) to (11) according to ACI318-99 (ACI Committee 318, 1999).

$$V_n = V_c + V_s \quad (9)$$

$$V_c = 0.53\sqrt{f_c'}b_w d \quad (10)$$

$$V_s = \frac{A_v f_{yt} d}{s} \quad (11)$$

Where V_n is total nominal shear resistance. V_c is concrete shear resistance. V_s is stirrup shear resistance. f_c' is beam concrete strength. b_w is beam web width. s is stirrup spacing. $A_v f_{yt}$ is the product of rebar cross sectional area and yield strength of stirrup reinforcing.

It was found that among all of the simulated beam sizes, the distributions of shear resistance fell into 5 different groups depending on their sizes and ratio of stirrup shear resistances over concrete shear resistances $V_s/V_c = 0.25, 0.50, 0.75$.

Simulated and professional factor adjusted for 5 different beam groups are listed in table 10.

Table 10 Statistical parameters of simulated and professional factor adjusted beam shear resistance

Group	Simulated		Professional factor adjusted	
	Mean μ_R	COV V_R	Mean μ_R^*	COV V_R^*
1	1.2589	0.1907	1.3533	0.2122
2	1.2189	0.1798	1.3103	0.2024
3	1.2846	0.1881	1.3809	0.2098
4	1.3123	0.2236	1.4107	0.2422
5	1.3751	0.2548	1.4782	0.2712

The reliability indices β for beam shear were calculated from equation (8) for load ratio varying within the practical range. Figure 8 to 12 show plot between the β values and the load ratios for 5 different beam groups.

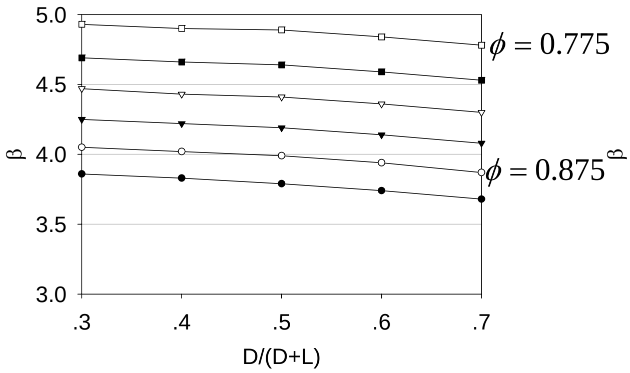

Figure 8 Reliability indices β for beam shear (group 1)

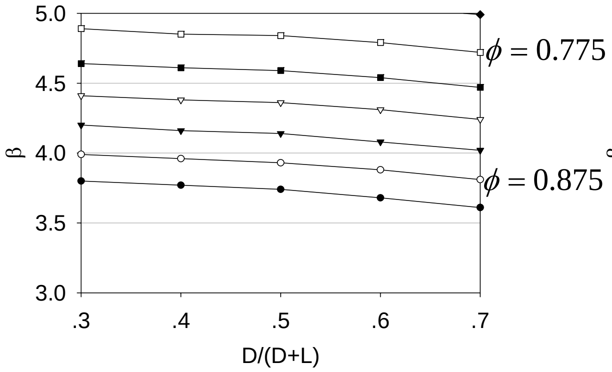

Figure 9 Reliability indices β for beam shear (group 2)

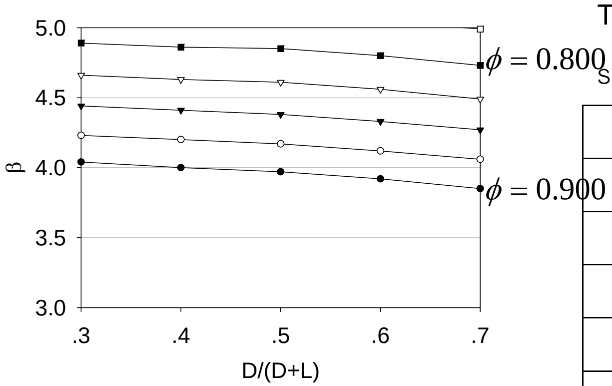

Figure 10 Reliability indices β for beam shear (group 3)

All selected ϕ's for 5 simulated beam groups at mid-range load ratio $D/(D+L) = 0.5$ were listed in Table 11.

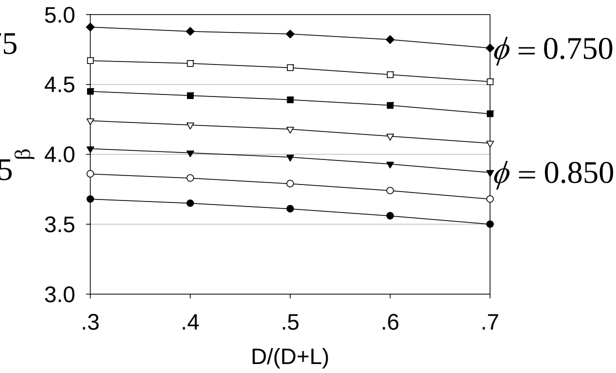

Figure 11 Reliability indices β for beam shear (group 4)

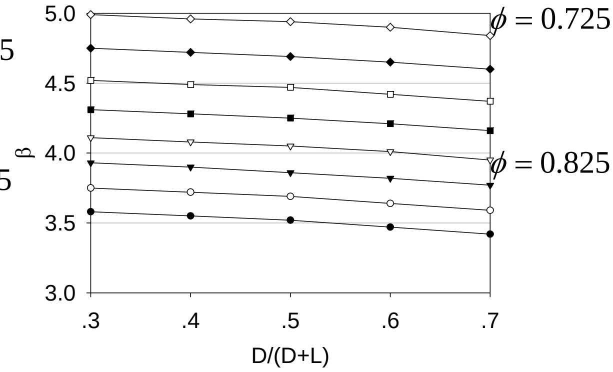

Figure 12 Reliability indices β for beam shear (group 5)

Table 11 Selected ϕ factor for beam shear.

Group	ϕ at $\beta_T = 3.95$
1	0.875
2	0.875
3	0.900
4	0.850
5	0.825
Average	0.865

Reliability analysis for column axial

The design formula for tied column axial resistance from ACI318-99 code (ACI Committee 318, 1999) is given in equation (12).

$$P_n = 0.80 \times 0.85\left[0.85 f_c'\left(A_g - A_{st}\right) + f_y A_{st}\right] \quad (12)$$

Where P_n is nominal axial resistance. A_g is cross sectional gross area. A_{st} is cross sectional area of longitudinal reinforcing steel. $f_y A_{st}$ is the product of rebar cross sectional area and yield strength of longitudinal reinforcing steel. The distribution of DB12 and DB16 (SD30) together was used.

The factor 0.80 in front of the right hand side of equation (12) serves for the reduction of axial resistances due to the load eccentricities not considered in the analysis (ACI Committee, 1999). For the pure axial resistances, this factor should be removed from equation (12) so it becomes

$$P_n = 0.85\left[0.85 f_c'\left(A_g - A_{st}\right) + f_y A_{st}\right] \quad (13)$$

Their statistical parameters including means and COV's are listed in Table 12.

The β versus load ratio plots for simulated column axial are shown in Figures 13 to 16.

The selected ϕ factors which give the closest β's to the β_T are listed in Table 13. This selection are based on the mid-range load ratio for columns $D/(D+L) = 0.65$.

Table 12 Statistical parameters of simulated and professional factor adjusted column axial resistance

Reinf. Ratio ρ_t	Simulated		Professional factor adjusted	
	Mean μ_R	COV V_R	Mean μ_R^*	COV V_R^*
1%	1.1928	0.3708	1.1928	0.3793
2%	1.2401	0.3293	1.2401	0.3293
3%	1.2746	0.3072	1.2746	0.3072
4%	1.3010	0.2961	1.3010	0.2961

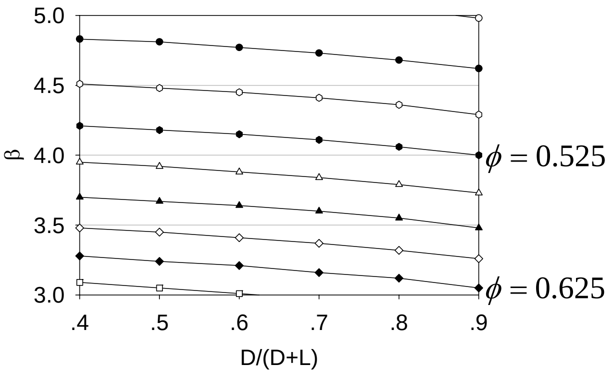

Figure 13 Reliability indices β for column axial (reinf. ratio 1%)

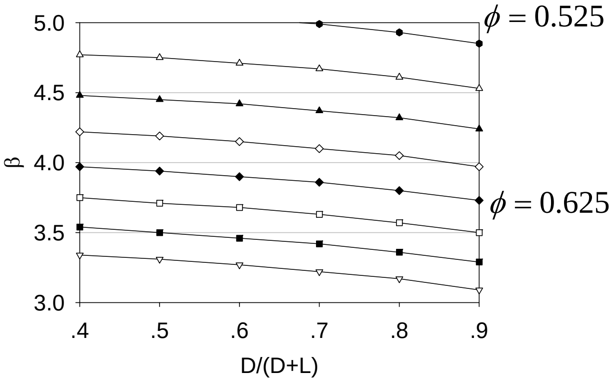

Figure 14 Reliability indices β for column axial (reinf. ratio 2%)

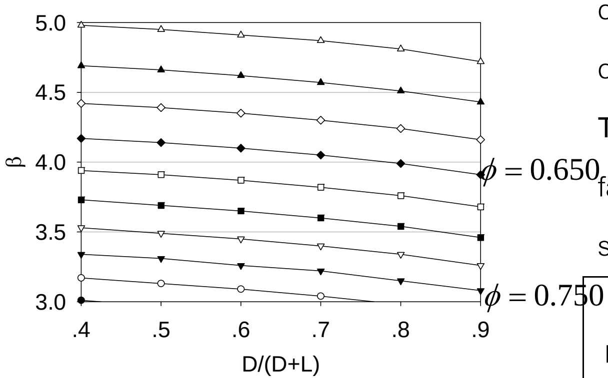

Figure 15 Reliability indices β for column axial (reinf. ratio 3%)

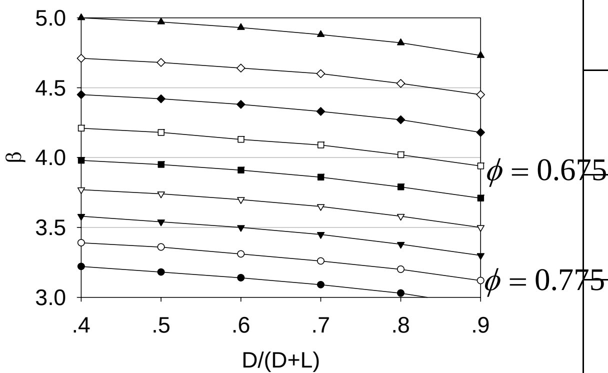

Figure 16 Reliability indices β for column axial (reinf. ratio 4%)

Table 13 Selected reliability indices β for all three member types/limit states.

Reinforcement ratio ρ_t	ϕ at $\beta_T = 3.98$
1%	0.525
2%	0.625
3%	0.650
4%	0.675
Average	0.619

All the chosen ϕ factors are listed and compared to the draft building code case 2 (Thailand structural and soil building codes committee, 2005) and the ACI318-99 code in Table 14.

Table 14 Chosen strength reduction factor ϕ for all three member types/limit states.

Member type/ Limit state	Strength reduction factor ϕ		
	This paper	Draft Thailand building code (Case 2)	ACI318-99
Beam flexure	0.79	0.75	0.90
Beam shear	0.87	0.71	0.85
Tied Column axial	0.62	0.58	0.70

Conclusions and Suggestions

It was found that the selected strength reduction factors from this study are different from those recommended by draft building code for case 2 which are not based on any scientific evidence so it is not worth to compare. The ϕ factors for beam-flexure and tied column-axial are lower 12% and 11% lower than those of ACI318-99 consecutively. These due to the deviation of steel and concrete properties in Thailand are significantly higher than those in USA. In the contrary, the ϕ factor for beam-shear from this study is 2% lower than that of the ACI318-99. This due to the fact that the average value for stirrup spacing is only 86% of specified value. However, the statistical data used in this study are very limited therefore, it is suggested that more study must be conducted to ensure the correctness of these factors before any adoption for the Thailand building codes.

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