Comparison of the reliability index between American codes and Chinese codes

by

Haofan Zhao

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Approved by

Andrzej Nowak, Chair, Professor of Civil Engineering
J. Michael Stallings, Professor of Civil Engineering
Robert Barnes, Associate Professor of Civil Engineering

Abstract

The primary purpose of this research is to apply the reliability analysis techniques to the evaluation of two different series' structural design codes using in America and China. This research is focused on two kinds of structures: beams and columns. For the objective, eight calculation examples were generated. They would be separately designed by Chinese codes (the GB50010-2010 & GB50009-2012) and American codes (ASCE 7-10 and ACI318-14). In the eight calculation examples, two of them were beam structures and the others were column structures.

The GB 50010-2010 is the national code for design of concrete structures in China. The GB50009-2012 is the load code for design of building structures. They are widely used in structural design in China. The examples designed by Chinese codes would progress the load combination and determine the required resistance by GB50009-2012 (load code for design of building structures). Afterward, according to the GB50010-2010, reinforcement ratios were estimated. For the examples designed by American codes, the load combination and required resistance would be determined by ASCE 7-10, reinforcement ratio would be calculated based on ACI 318-14.

In this research, the Monte Carlo method was used to evaluate the reliability index for examples. Then the reliability index for each example was applied to the sensitivity analysis, the results of which would give a good view for comparison of the two series' codes.

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List of Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ASCE	American Society of civil engineers
GB	National standards of People's Republic of China

1. Introduction

1.1. Overview

The reliability index is an essential parameter for the structure design. Reliability in the structures is defined as “the probability of a device performing its purpose adequately for the period of time intended under the operating conditions encountered” (Bazovsky, 1961). Generally, the reliability index directly describes how stable the structures are. In addition, it is also an important factor of economic control.

The reliability analysis is a kind of analytical technology based on statistics and probability theory. With the statistical data, it can provide a value to represent how many possibilities this structure will have a failure. Generally, 3.50 is the number of reliability when the structures had enough resistance and excellent economic benefit. This research focused on the reinforcement concrete beams and reinforced concrete columns. The actual concrete structural element’s capacity is varying due to many factors, such as yield strength of steel rebars, the compression strength of concrete, and dimensions of the structural elements. Actual loads are also changed.

1.2. Objective

The main purpose of this research is to apply the reliability analysis techniques to the evaluation of different series structural design codes (ACI 318-14, ASCE 7-10 and GB 50010-2010, GB 50009-2012) and compare the reliability performances of the examples designed by these two series’ codes. There would be two kinds of concrete component examples involved in this research: reinforced concrete beam and reinforced concrete column. The beams’ examples would mainly show how the different loads' design methods affect the reliability. And the columns' examples would mainly show how the different resistance design methods affect the reliability.

1.3. Introduction of Chinese codes and American codes

The Chinese codes used to design and analyze were GB 5009-2012 "Load code for the design of building structures" and GB 50010-2010 "Code for design of concrete structures." GB is the abbreviation for the Chinese National Standard. It is a series of design codes and requirements for all kinds of engineer areas. GB 5009-2012 is the load's design code edited by Ministry of Housing and Urban-Rural Development of the People's Republic of China. It is the most official and widely used code for the load's design. GB 50010-2010 is the concrete component structural design code, also edited by Ministry of Housing and Urban-Rural Development of the People's Republic of China. It provides the minimum requirements of structural concrete components. Any kinds of structural concrete components should meet the requirements of GB 50010-2010 and required extra codes.

The American codes used to design and analyze were ASCE 7-10 "Minimum design loads for buildings and other structures" and ACI 318-14 "Building code requirements for structural concrete." ASCE 7-10 is the load's design codes published by American Society of Civil Engineering. It is only specific for building design. "It provided minimum loads requirements for the design of buildings and other structures that are subject to building code requirements" (American society of civil engineers, 2010). ACI 318-14 is the concrete component design codes published by American Concrete Institute. It provides the minimum requirements for materials, design and detailing of structural concrete building and, where applicable, nonbuilding structures (American Concrete Institute, 2014). It is only specific for building design like ASCE 7-10.

In this research, the examples for analysis were assumed as components of office buildings and residential buildings. Each example applied different series codes for design and reliability analysis. Due to different requirements in the codes, the performance of reliability had significant differences.

1.4. Scope and research approach

For the purpose of this research, there were several different comparison examples. Due to the limited research resources, the focus was on two types of structures: reinforced concrete beams and reinforced concrete columns. There were two beams examples: one designed by GB codes and another one designed by ACI and ASCE codes. The basic resistance design principles are similar to each other with only a slight difference. There were six examples for the columns. Half of them were designed by GB codes, while the others were designed by ACI and ASCE codes. The reason for the number of column examples was that the resistance requirements and design methods were entirely different from each other. Therefore, more examples were required to cover all situations. Chapter four will discuss in more detail about the resistance design method and chapter seven will provide detailed differences between the two series' codes. All of the examples will be applied the reliability analysis to calculate reliability index. Then, the sensitivity analysis should be considered. Through the sensitivity analysis results, it could be easier to get conclusions about the two series' codes.

1.5. Organization of the research

This research is organized by seven chapters. Chapter one gives the brief introduction to this report. It also includes the objective, introduction of design codes, scope and approach. Chapter two illustrates the load design methods of GB 50010-2010 and ASCE 7-10. It also provides load requirements, load combinations and statistical parameters in each code. Chapter three reviews the beam resistance design methods of GB 5009-2012 and ACI 318-14, includes resistance functions and statistical parameters for each factor. Chapter four introduces the column design methods of GB 5009-2012 and ACI 318-14 while illustrating different situations requirements in each code and provides the statistical parameters of factors. Chapter five presents the procedure of reliability analysis and how it is to be applied to the examples. Chapter six discusses the comparison between two series codes and compares the results after reliability analyzing. Chapter seven will provide the conclusion from chapter two to chapter six, briefly stating the advantages and disadvantages of the two series' codes.

2. Loads

All the structural components are designed to have certain required resistance, which should be larger than the design loads on the component. As a result, determining the design loads is the first step in the procedure of finishing the component design. This chapter will introduce the load requirements in GB 5009-2012 “Load code for the design of building structures” and ASCE 7-10 “Minimum design loads for buildings and other structures” and provide the statistical parameters for each load, which will be used for reliability analysis. For the objective of this research, dead load, live load, snow load and wind load were involved in the study.

2.1. Load design methods and requirements in Chinese code (GB 5009-2012)

In Chinese code GB 5009-2012, all loads are separated into three categories: permanent load, variable load and accidental load. Permanent load concludes the gravity load due to the self-weight of the structure component, retaining member, surface layer, decoration, fixed equipment, long-term storage; soil pressure; water pressure; and any other loads which are considered as the permanent load (Ministry of Housing and Urban-Rural Development of the People's Republic of China, Beijing 2012). Variable load is the load that changes over time during the design reference period or whose change could not be neglected compared to the average. Accidental load is the load that does not absolutely appear during the design reference period and with a large magnitude and a short duration, such as earthquake load and hurricane load. The accidental load was not involved in this research. The permanent load in this research was dead load, and the variable loads were live load, wind load and snow load.

2.1.1. Requirements and calculation functions of loads in GB 50009-2012

The dead load in this research was mainly from the self-weight of the structures, which is the reinforced concrete components. GB 50009-2012, “load code for design of building structures,” provides the approximate self-weight density for different kind of materials, which is shown in Table 1.

Table 1-Regular materials self-weight

name		self-weight
lime, cement, mortar and concrete (kN/m^3)	Regular reinforced concrete	24.0~25.0
	Reinforced concrete masonry	20.0
	Wire mesh cement	25.0
	Water glass concrete with acid resistance	20.0~23.5
	Fly ash ceramsite concrete	19.5

Live load is always applied on the floors and roofs. It is mainly the weight of people and their possessions, furniture and movable partitions. For different types of buildings, there are different values of the live load. In this research, all examples were functional in the office building or residential building. GB 50009-2012 provides a design value of live load in different functional buildings. Some of them are shown in Table 2.

Table 2-Norminal value of live load , combination value, frequent value and quasi-permanent value

category	nominal value	factor of combination value	factor of frequent value	factor of quasi-permanent value
	(kN/m^2)	ψc	ψt	ψq
Residential, dorm room, hostel, office building, hospital ward, nursery and kindergarten	2.0	0.7	0.5	0.4
Laboratory, reading room, conference room and hospital clinic room	2.0	0.7	0.6	0.5
Classroom, canteen, restaurant, archive room	2.5	0.7	0.6	0.5

Snow load is prevalent in northern and/or mountainous regions all over the world. It is the result of the accumulation of snow from many storms over the course of a winter season. Between winter storms, the roof systems may lose some of the accumulated snow as the result of wind activity and/or melting from either warm temperatures, or from building heat (Ministry of Housing and Urban-Rural Development of the People's Republic of China, Beijing 2012).

In GB50009-2012, the snow load is determined by the following function:

$$s_k = \mu_r s_0$$

Where s_k is the nominal value of snow load, μ_r is the distribution factor of roof accumulated snow and s_0 is basic snow pressure. GB50009-2012 requires that basic snow pressure should use the nominal value in 50 years. However, if the structures are sensitive for snow load, the basic snow pressure should use the nominal value in 100 years. GB50009-2012 also provides the basic snow pressure in different locations in China, Table 3 shows the snow pressure that was used in this research.

Table 3-Basic snow pressure in Beijing

Location	Snow pressure (KN/m^2)		
	R=10 years	R=50 years	R=100 years
Beijing	0.25	0.4	0.45

According to requirements in the GB50009-2012, the distribution factor of roof μ_r was 0.85 in this research. Through the previous equation, the nominal value of snow load was calculated to be 0.34 KN/m^2.

Wind is a mass of air that moves in a mostly horizontal direction from an area of high pressure to an area with low pressure (J. Struct. Eng., 1999, 125(4): 453-463). High winds can be very destructive because they generate pressure against the surface of a structure. The intensity of this pressure is the wind load. The effect of the wind is dependent upon the size and shape of buildings. In GB50009-2012, the wind load is determined by the following function:

$$w_k = \beta_z \mu_s \mu_z w_0$$

Where w_k is the nominal value of wind load, β_z is the factor of wind vibration at the height z, μ_s is the building shape factor for the wind load, μ_z is the variety factor of the different heights for wind pressure and w_0 is basic wind pressure. GB50009-2012 requires that the basic wind pressure should use the nominal value in 50 years but should not be less than 0.3 KN/m^2. Table 4 shows the wind pressure of the example's location from GB50009-2012.

Table 4-Basic wind pressure in Beijing

Location	Altitude (m)	Wind pressure (KN/㎡)		
		R=10 years	R=50 years	R=100 years
Beijing	54	0.3	0.45	0.5

In GB50009-2012, the terrain roughness known as exposure is an important factor in determining β_z (the factor of wind vibration at the height z) and μ_z (the variety factor of the different height for wind pressure). There are four different kinds of roughness: Category A refers to offshore seas and islands, coasts, lakeshores and desert areas; Category B refers to fields, villages, jungles, hills, and townships where housing is sparse; Category C refers to urban areas of densely populated clusters; Category D refers to cities with dense buildings and high-rising buildings in urban areas. In this research, the roughness was classified as Category D. GB50009-2012 provides the function of β_z:

$$\beta_z = 1 + 2gI_{10}B_z\sqrt{1+R^2}$$

Where g is the peak factor (the value of which is 2.5 in this research); I_{10} is the turbulence intensity at the height of 10 m. Its value is 0.12 for Category A, 0.14 for Category B, 0.23 for Category C and 0.39 for Category D; R is resonance component factor of pulsating wind load, determined by the function in GB50009-2012; R is the resonance component factor of pulsating wind load, which is determined by the following functions in GB50009-2012:

$$\mathrm{R} = \sqrt{\frac{\pi}{6\zeta_1}\frac{x_1^2}{(1+x_1^2)^{4/3}}}$$

$$x_1 = \frac{30f_1}{\sqrt{k_w w_0}}, x_1 > 5$$

Where f_1 is the first order of the structure natural frequency (Hz); k_w is the roughness correction factor, it is equal to 1.28 for Category A, 1.0 for Category B, 0.54for Category C and 0.26 for Category D; ξ_1 is the structural damping ratio, for steel structure it is 0.01 and for reinforce concrete structure it is 0.02. B_z is the background component factor of pulsating wind load, which is determined by the function from GB50009-2012:

$$B_z = kH^{\alpha_1}\rho_x\rho_z\frac{\phi_1(z)}{\mu_z}$$

Where $\phi_1(z)$ is the structural first order vibration mode coefficient; H is total height of the structure, H should not be more than 300 m for Category A, no more than 350 m for Category B; no more than 450 m for Category C; no more than 550 m for Category D; ρ_x is a factor related to the horizontal direction of pulsating wind load; ρ_z is a factor related to the vertical direction of pulsating wind load; k, α_1 are the factors directly provided from GB50009-2012 shown in Table 5;

Table 5-Factor k and α_1

Roughness category		A	B	C	D
High rise building	k	0.944	0.670	0.295	0.112
	α1	0.155	0.187	0.261	0.346
Tower building	k	1.276	0.910	0.404	0.155
	α1	0.186	0.218	0.292	0.376

ρ_x and ρ_y are determined by the functions from GB50009-2012 as following:

$$\rho_z = \frac{10\sqrt{H + 60e^{-H/60} - 60}}{H}$$

$$\rho_x = \frac{10\sqrt{B + 50e^{-H/60} - 50}}{B}$$

Where B is the width of windward side, $B \leq 2H$. μ_s (the building shape factor for the wind load) and μ_z (the variety factor of the different height for wind pressure) were directly determined from the requirements in GB50009-2012. μ_s is related to the shape of the buildings. Due to the shape of the examples in this research, μ_s was -0.6. μ_z is a factor related to the exposure and height. The value was 0.51, determined by Table 6 from GB50009-2012 as following:

Table 6- the variety factor of the different height for wind pressure

height (m)	Roughness category			
	A	B	C	D
5	1.09	1.00	0.65	0.51
10	1.28	1.00	0.65	0.51
15	1.42	1.13	0.65	0.51
20	1.52	1.23	0.74	0.51
30	1.67	1.39	0.88	0.51
40	1.79	1.52	1.00	0.60
50	1.89	1.62	1.00	0.69
60	1.97	1.71	1.20	0.77
70	2.05	1.79	1.28	0.84
80	2.12	1.87	1.36	0.91
90	2.18	1.93	1.43	0.98
100	2.23	2.00	1.50	1.04

2.1.2. Load combination in GB 50009-2012

After the nominal value for each kind of loads is determined, the next step is to finalize the nominal design load value, which is calculated by the load combination. GB50009-2012 requires that this function should control the load and resistance as following (Ministry of Housing and Urban-Rural Development of the People's Republic of China, Beijing 2012):

$$\gamma_0 S_d \leq R_d$$

Where γ_0 is the structure importance factor, S_d is the design value of load combination effect and R_d is the design value of structure resistance. The design value of load combination effect S_d should get the largest value from two different functions.

The first function is mainly controlled by the variable load, the load combination function is (Ministry of Housing and Urban-Rural Development of the People's Republic of China, Beijing 2012):

$$S_d = \sum_{j=1}^{m} \gamma_{G_j} S_{G_j k} + \gamma_{Q_1} \gamma_{L_1} S_{Q_1 k} + \sum_{1=2}^{n} \gamma_{Q_i} \gamma_{L_i} \psi_{c_i} S_{Q_i k}$$

Where γ_{G_j} is the j-th bias factor of the permanent load, γ_{Q_i} is the i-th bias factor of the variable load, γ_{Q_1} is the factor for the mainly controlled variable load, γ_{L_i} is the i-th bias factor of the service life considered by the variable load, γ_{L_1} is the factor for the mainly controlled variable load, $S_{G_j k}$ is the j-th the nominal value of the load effect of the permanent load, $S_{Q_i k}$ is the i-th the nominal value of the load effect of the variable load, ψ_{c_i} is the i-th combination factor of variable load Q_i, m is the quantity of the permanent load and n is the quantity of the variable load;

The second function is mainly controlled by permanent load, the load combination function is (Ministry of Housing and Urban-Rural Development of the People's Republic of China, Beijing 2012):

$$S_d = \sum_{j=1}^{m} \gamma_{G_j} S_{G_j k} + \sum_{1=2}^{n} \gamma_{Q_i} \gamma_{L_i} \psi_{c_i} S_{Q_i k}$$

The value of γ_{G_j} in these two functions are different, $\gamma_{G_j} = 1.2$ in the first function and $\gamma_{G_j} = 1.35$ in the second function. Due to the load case in this research, these two functions could be rewritten as following:

① $1.35\mathrm{D} + 0.98\mathrm{L} + 0.98\mathrm{S} + 0.84\mathrm{W}$

② $1.2\mathrm{D} + 1.4\mathrm{L} + 0.98\mathrm{S} + 0.84\mathrm{W}$

③ $1.2\mathrm{D} + 0.98\mathrm{L} + 1.4\mathrm{S} + 0.84\mathrm{W}$

④ $1.2\mathrm{D} + 0.98\mathrm{L} + 0.98\mathrm{S} + 1.4\mathrm{W}$

Through load combination, load effect q_u could be estimated. Once q_u is determined, the required resistance would be calculated through the LRFD (Loads and Resistance Factor Design) method.

2.1.3. Statistical parameters of loads in GB5009-2012

The reliability analysis requires the statistical data for each factor in the limit state function. Therefore, the statistical parameters of loads are necessary to analyze. Because of the different code's requirements, the statistical datas for each load in GB50010-2010 are not the same as they are in ASCE 7-10.

This research contained dead load, live load, wind load and snow load as the forces applied to the examples. Dead load is a constant load, unlike the live load, wind load and snow load, in that dead load does not change along the service life. The statistic parameter of dead load was presented by Zhenchang Li and Jiayan Wang，1986. Generally, live load has a significant influence on the reliability analysis. In GB50009-2012, the statistical survey results for live load separates into two kinds of live loads: constant live load and temporary live load. The constant live load is considered to be an invariable live load in a certain period, such as the load from furniture. The temporary live load is supposed to be a live load occasionally applied in a short term, such as the human weight in a party. GB50009-2012 presents the statistical parameters for these two kinds of live loads. The statistical parameter of snow load, which is based on the meteorological data, was presented by Bonian Hou and Caiang Wei，1986. Wind load is also based on meteorological data, the statistic parameter of wind load was presented by Xiangyuan Tu，1986. The summary of statistical parameters of loads is shown in Table 7.

Table 7- Statistical parameter of loads in GB5009-2012

Load type	Bias factor	COV	Distribution type
Dead	1.06	0.07	normal
Constant live	0.193	0.4611	extreme 1
Temporary Live	0.1775	0.6873	extreme 1
Wind	0.610	0.27	extreme 1
Snow	0.358	0.712	extreme 1

2.2. Load design methods and requirements in American code (ASCE 7-10)

For the comparison, the examples designed by American codes applied the same kinds of loads: dead load, live load, snow load and wind load. ASCE 7-10 provides the detailed requirements and calculation functions for each load, which are quite different from those provided from GB50009-2012.

2.2.1. Requirements and calculation functions of loads in ASCE 7-10

"Dead Loads consist of the weight of all materials of construction incorporated into the building including, but not limited to, walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding, and other similarly incorporated architectural and structural items and fixed service equipment including the weight of cranes" (ASCE 7-10, Reston, 2010). The definition of dead load in ASCE 7 is similar to the description in GB50009-2012. Due to the comparison, each couple of examples kept the same dimensions and materials. Therefore, the dead load of the examples designed by ASCE 7-10 was also the self-weight of the reinforced concrete components. The approximate unit weight of reinforced concrete for design is 150 pcf, which is equal to 24 KN/m^3.

In ASCE 7-10, there are several types of loads considered to be live load. It includes: fixed ladder, grab bar system, guardrail system, handrail system, helipad, live load, roof live load, screen enclosure and vehicle barrier system (ASCE 7-10, 2010). In this research, live load was the typical live load, usually "produced by the use and occupancy of the building or other structure that does not include construction or environmental loads" (ASCE 7-10, 2010). ASCE 7 -10 provides a design live load 2.4 KN/m^2 for an office building, which is larger than the value in the Chinese code.

Snow load is always one of the majority vertical load for the buildings in the northern region. In ASCE 7-10, it is required that the snow load should be calculated related to ground snow load. The ground snow load is "based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2 percent annual probability of being exceeded (50-years mean recurrence interval)" (ASCE 7-10, 2010). Generally, with the data of the ground snow load, flat roof snow load can be calculated. Then, the sloped roof snow load is a function of flat roof snow load. The snow load can be calculated according to the following functions (ASCE 7-10, 2010):

$$p_f = 0.7 C_e C_t I_s p_g$$

$$p_s = C_s p_f$$

Where p_f is the flat roof snow load, C_e is the exposure factor, C_t is the thermal factor, I_s is the importance factor, p_g is the ground snow load, p_s is the sloped roof snow load and C_s is the roof sloped factor.

In ASCE 7-10, the wind load calculations are separated into two types. One is applied to main wind force resisting systems; another is applied to components and claddings. In this research, beams and columns were the main research objects. They were part of the main wind force resisting system. There are several methods to calculate wind load. The envelope procedure was selected to be the calculation method here, which is specified to low rise buildings. The wind pressure can be calculated according to the following function (ASCE 7-10, 2010):

$$\mathrm{p} = q_h\left[\left(GC_{pf}\right) - \left(GC_{pi}\right)\right]$$

Where p is the design wind pressure, q_h is the velocity pressure evaluated at mean roof height, GC_{pf} is the external pressure coefficient and GC_{pi} is the internal pressure coefficient.

2.2.2. Load Combination in ASCE 7-10

Load combination is the most important step to determine the load effect on the structure. Just like GB50010-2010, ASCE 7-10 also followed the LRFD method to proceed design. However, even both codes used the load combination to figure out the total load effect, the exact combination functions are not the same. In ASCE 7-10 chapter 2, the basic load combinations are given:

① $1.4\mathrm{D}$

② $1.2\mathrm{D} + 1.6\mathrm{L} + 0.5(L_r\ or\ S\ or\ R)$

③ $1.2\mathrm{D} + 1.6(L_r\ or\ S\ or\ R) + (L\ or\ 0.5W)$

④ $1.2\mathrm{D} + 1.0\mathrm{W} + \mathrm{L} + 0.5(L_r\ or\ S\ or\ R)$

⑤ $1.2\mathrm{D} + 1.0\mathrm{E} + \mathrm{L} + 0.2\mathrm{S}$

⑥ 0.9D+1.0W

⑦ 0.9D+1.0E

Load effect q_u is the largest value throughout the previous load combinations. Once q_u is determined, the required resistance is gained by factored q_u.

2.2.3. Statistical parameters of loads in ASCE 7-10

In this research, all the examples show later to meet the requirements in ASCE 7-10 and cooperated with ACI318-14. The design load（factored load）was defined by the load combination in chapter 2.2.2. Here, it contained dead load, live load, wind load and snow load. Unlike GB 50010-2010, live load does not separate into two individual categories. The statistical parameter of dead load was obtained from F.M. Bartlett, H.P. Hong, and W. Zhou, Can, 2003. The statistical parameter of live load was obtained from Nowak, A.S. and Collins, K.R, CRC Press,2013. The statistical parameter of wind load was obtained from Bruce R. Ellingwood and Paulos Beraki Tekie, 1999. The statistical parameter of snow load was obtained from Kyung Ho Lee and David V. Rosowsky, 2005. The summary of loads statistical parameters is shown in Table 8.

Table 8- Statistical parameter of loads in ASCE 7-10

Load type	Bias factor	COV	Distribution type
Dead	1.05	0.1	normal
Live	0.273	0.598	extreme 1
Wind	0.66	0.37	extreme 1
Snow	0.224	0.82	lognormal

3. Resistance of reinforced concrete beam

Generally, beams will resist loads from the floor or roof, primarily in flexure load. This research will focus on the beam moment resistance. The requirements in Chinese code GB 50010-2010 and American code ACI 318-14 are similar. The only difference is the equivalent rectangular concrete stress.

3.1. Beam resistance design method in GB 50010-2010

The example was a simple supported beam, which is considered as resisting positive bending moment in the mid-span. Figure 1 shows the details of the resistance of the positive bending structural element from GB 50010-2010.

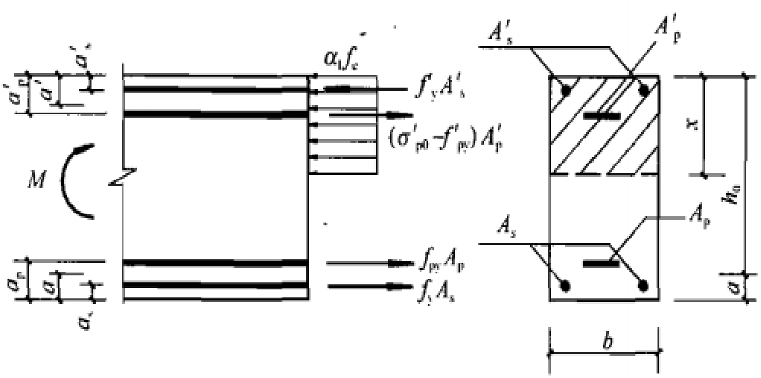

Fig 1. Moment resistance details in GB 50010-2010

Where α_1 is the factor related to the concrete compression strength. When the concrete strength level is under C50 (32.4 MPA), $\alpha_1 = 1$ and when the concrete strength level is C80 (50.2 MPA), $\alpha_1 = 0.94$. The value for the other concrete strength can use linear interpolation method to calculate. f_c is the design value of concrete axial compressive strength. A_s is longitudinal nominal steel rebar area in the tension zone through the cross-section. A'_s is longitudinal nominal steel rebar area in the compression zone through the cross-section. b is the width of the rectangular cross-section. h_0 is the effective height of the section. a'_s is the distance between the edge to the total force point of regular bars in the compression zone. x is the height of the concrete compression block, $2a' \leq x \leq \xi_b h_0$. a' is the distance between the edge to the total force point of all reinforcements in the compression zone. ξ_b is the relative limit pressure zone height. Because there were no prestressed rebar or wires and no reinforcement to resist negative moment, the resistance function could be rewritten as following:

$$\mathrm{M} \leq \alpha_1 f_c b x \left(h_0 - \frac{x}{2}\right)$$

$$\alpha_1 f_c b x = f_y A_s$$

This function provided by GB 50010-2010 is the beam resistance function. The reinforcement area of the cross-section can be calculated from this function. Afterward, this resistance function can be used for reliability analysis.

3.2. Beam resistance design method in ACI 318-14

Due to the same assumption and principle for beam design, the moment resistance function of the beam was similar. Because there was only positive reinforcement in the beam examples, the resistance function for a simple supported beam could be calculated by the following function:

$$\mathrm{R} = A_s f_y \left(d - \frac{a}{2}\right)$$

$$\mathrm{a} = \frac{A_s f_y}{\beta_1 f'_c b}$$

The details of the positive moment resistance are shown in Figure 2.

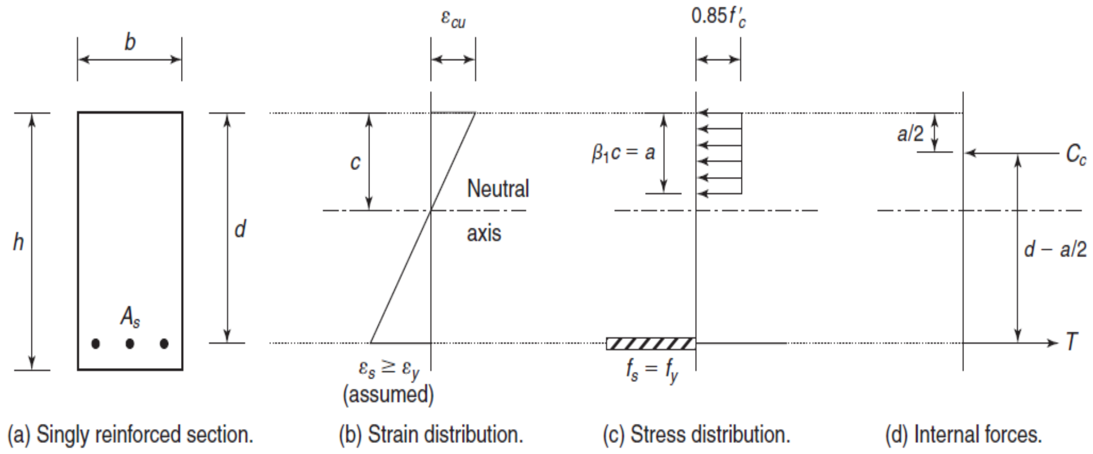

Fig 2. Moment resistance details of beam cross-section

In ACI 318-14, the value of β_1 is not constant. It changes related to the compression strength of concrete, from 0.85 to 0.65. The value of β_1 is the only difference in the resistance function between Chinese code and American code.

3.3. Statistical parameters of moment resistance

There are three major aspects of the parameter of resistance: material, fabrication and analysis. The actual resistance R is defined from the nominal resistance, R_n , material factor, M_F , fabrication factor, F_F and professional factor, P_F , as following (Nowak, A.S. and Collins, K.R 2013):

$$\mathrm{R} = R_n \times M_F \times F_F \times P_F$$

Material factor is related to uncertainty in the strength of material, modulus of elasticity, cracking stresses, and chemical composition (Nowak, A.S. and Collins, K.R 2013). In this study, the underlying variable that influenced the resistance of beam belong to material aspect were the compression strength of concrete and yield strength of the reinforcement. Fabrication factor is related to uncertainty in the overall dimensions of the component which can affect the cross-section area, moment of inertia and section modulus (Nowak, A.S. and Collins, K.R 2013). The primary variable that influenced the resistance of beam belong to the fabrication factor were dimensions of the cross-section here. Professional factor is related to uncertainty resulting from approximate methods of analysis and idealized stress/strain distribution models (Nowak, A.S. and Collins, K.R 2013). In GB 50010-2010 and ACI 318-14, the principle of moment resistance function is almost the same. However, the statistic parameters of the material factors, fabrication factors and professional factors are not the same. The summary of moment resistance statistical parameters of GB 50010-2010 is shown in Table 9 (Xinyan Shao, Chongxi Bai, Wang Liang, Atlantis Press, 2015).

Table 9- Statistical parameters of moment resistance in GB 50010-2010

parameter	nominal	bias factor	COV	distribution type
f_c	20.1 MPa	1.41	0.19	normal
f_y	400 MPa	1.12	0.093	normal
ρ	0.0045-0.01	1	0.05	normal
b	300 mm	1	0.02	normal
h	800 mm	1	0.02	normal
d	760 mm	1	0.03	normal

The summary of moment resistance statistical parameters of ACI 318-14 is shown in Table 10 (Andrzej S. Nowak and Maria M. Szerszen, Boca Raton, 2013).

Table 10- Statistical parameters of moment resistance in ACI 318-14

parameter	nominal	bias factor	COV	distribution type
f'_c	20.6 MPa	1.25	0.135	normal
f_y	413.67 MPa	1.2-1.125	0.057	normal
ρ	0.0045-0.01	1	0.049	normal
b	300 mm	1.01	0.04	normal
h	800 mm	1.01	0.04	normal
d	760 mm	0.99	0.04	normal

The ultimate strength of the beam would vary due to the factors showed in Table 9 and Table 10. With the moment resistance function, the actual resistance for reliability analysis can be calculated.

4. Resistance of reinforced concrete column

Column is one important element of the load resistance structural system. Rather than the simply supported beam, the column mainly resists the bending moment and axial force simultaneously. It is also required to resist torsion and shear. The design and analysis of columns are more complicated than the simple supported beam. The design procedure of columns is similar as beams. At first, the load combination estimates required resistance, then the reinforcement can be calculated by the functions from the code. In the Chinese codes, the column separates into two categories: large eccentric compression member and small eccentric compression member. These two kinds of column design methods have different requirements and design procedure, which will be introduced carefully in the following paragraph. In American codes (ACI 318), the column design separates as two methods: non-sway frame column design and sway frame column design. Determination of sway frame and non-sway frame depends on the rigidness of whole frame structures. Due to each code having different categories in the column design methods, this research generated six basic calculation examples for comparison. In addition, the design requirements of loads are the same as chapter two.

4.1. Column resistance design method in ACI 318

In GB 50010-2010, columns are considered as a structural member that resists moment and axial force in tandem. There are two types of column: large eccentric compression member and small eccentric compression member. Each of them has different design methods.

Before estimating the column type, it should verify the slenderness effect on the design moment. If $l_c/i \leq 34 - 12(M_1/M_2)$, the slenderness effect could be neglected; if not, the influence of the slenderness effect on the design requirement moment should be considered. With the slenderness effect, the design requirement moment should be enlarged. It could be replaced by M, which could be expressed by:

$$\mathrm{M} = C_m \eta_{ns} M_2$$

$$C_m = 0.7 + 0.3\frac{M_1}{M_2} \geq 0.7$$

$$\eta_{ns} = 1 + \frac{1}{1300\left(\frac{M_2}{N} + e_a\right)/h_0}\left(\frac{l_c}{h}\right)^2 \zeta_c$$

$$\zeta_c = \frac{0.5 f_c A}{N}$$

Where N is the design axial force due to M_2; e_a is the additional eccentricity, which is the larger value between 20 mm and 1/30 of cross-section height; A is the gross area of the cross-section;

After the required moment resistance enlarged due to slenderness, the columns should be verified as large eccentric compression members or small eccentric compression members. The central principle to verify the column types is to compare the factored eccentricity with the $0.3h_0$, where h_0 is the distance from the edge of the compression block in the cross-section to the edge of longitude steel rebar in the tension zone. If the factored eccentricity is larger than $0.3h_0$, the design column should be considered as a large eccentric compression member; if not, it should be considered as a small eccentric compression member. According to GB 50010-2010, the factored eccentricity has two components: eccentricity enlarge factor, η, and initial eccentricity, e_i. The initial eccentricity, e_i, could be expressed by

$$e_i = e_0 + e_a$$

Where e_0 is the design eccentricity, which could be expressed by

$$e_0 = \frac{M_u}{P_u}$$

Where M_u is the design moment requirement; P_u is the design axial force requirement; e_a is the additional eccentricity, which is the larger value between 20 mm and 1/30 of cross-section height. The eccentricity enlarge factor, η, could be expressed by

$$\eta = 1 + \frac{1}{1400e_i/h_0}\left(\frac{l_0}{h}\right)^2 \zeta_1\zeta_2$$

Where, l_0 is the effective length of the compression member; h is the total height of cross-section; ζ_1 is the correction factor for the eccentric compression member curvature, which could be expressed by

$$\zeta_1 = \frac{0.5f_cA}{P_u}$$

Where ζ_2 is the slenderness effect factor, which could be expressed by

$$\zeta_2 = 1.15 - 0.01\frac{l_0}{h}$$

So, if the design column is estimated as a large eccentric compression member, the next step is to calculate required reinforcement by the functions in the codes. Because of the axial force, when the column achieves the limit state, the reinforcement in the tension area does not always achieve yield strength. That means the moment capacity is related to the axial force capacity, which cannot be considered separately. The limit state of the column is either moment or axial force achieving the ultimate strength. In GB 50010-2010, the ultimate compression strain in concrete is defined as, ε_{cu}, which can be calculated by $\varepsilon_{cu} = 0.0033 -$

$(f_c - 50) \times 10^{-5}$. With the ultimate compression strain in concrete and the stress-strain relationship, the stress of reinforcement in the ultimate strength could be calculated. In addition, the strength capacity functions of columns are the combination of reinforcement contribution and concrete contribution. The height of the concrete compression block, x, is important to determine the stress of the reinforcement in the limit state. Once x is known, the strain of any heights through the cross-section could be calculated by using ε_{cu} and the linear stress-strain relationship. GB 50010-2010 illustrates that the limit state of the large eccentric compression column is the reinforcement in the tension zone achieving yield strength and the concrete in the compression zone achieving the ultimate compression strain ε_{cu} concurrently. GB 50010-2010 defines the relative limit concrete compression block height, ζ_b , which could be expressed by

$$\zeta_b = \frac{\beta_1}{1 + \frac{f_y}{E_s \varepsilon_{cu}}}$$

It is a value without unit. GB 50010-2010 defines that ζ_b times effective height of cross-section h_0 is the height of concrete compression block when reinforcement achieves yield strength and concrete achieves ultimate compression strain ε_{cu} at the same time. So, the height of the concrete compression block, x , in the limit state for the large eccentric compression member could be calculated directly by $\zeta_b \times h_0$. According to the equilibrium of moment and axial force and requirement in the GB 50010-2010, the reinforcement area could be calculated by

$$A'_s = \frac{P_u e - \alpha_1 f_c b h_0^2 \zeta_b (1 - 0.5\zeta_b)}{f'_y (h_0 - a')}$$

$$e = \eta e_i + \frac{h}{2} - a$$

$$A_s = \frac{\alpha_1 f_c b h_0 \zeta_b + f'_y A'_s - P_u}{f_y}$$

Where a' is the distance from the neutral axis of reinforcement in compression zone to the edge of the concrete compression block; a is the distance from the neutral axis of reinforcement in the tension zone to the edge of the concrete. In addition, GB 50010-2010 requires that the reinforcement ratio in the compression zone should not be less than 0.2%; the reinforcement ratio in the tension zone should not be less than 0.2%; the total reinforcement ratio should not be less than 0.6%. It also requires checking the compression resistance, which could be expressed by

$$P_u = 0.9\varphi\left(f_c A + f'_y A'_s\right)$$

Where φ is the stability factor of the column, which is related to the length of member and width of the cross-section.

If the design column estimates as small eccentric compression member, there are two steps to verify the column eccentric type. The first step is the same method to verify the eccentric column type as large eccentric compression member, to compare the factored eccentricity ηe_i with the $0.3h_0$, If ηe_i is larger than $0.3h_0$, the design column should be considered as large eccentric compression member. If ηe_i is less than $0.3h_0$, it could be possible small eccentric compression member. The second step is to assume that the member is small eccentric compression column, and then calculate the relative height of compression concrete block, ζ . GB 50010-2010 provided the function of ζ specific for small eccentric compression member, which could be expressed by

$$\zeta = \frac{P_u - \zeta_b \alpha_1 f_c b h_0}{\frac{P_u e - 0.43\alpha_1 f_c b h_0^2}{(\beta_1 - \zeta_b)(h_0 - a'_s)} + \alpha_1 f_c b h_0} + \zeta_b$$

$$\mathrm{e} = \eta e_i + \frac{h}{2} - a$$

If $\zeta > \zeta_b$, the column could be considered as small eccentric compression member. If not, the column should follow the rules of large eccentric compression member. So once the design column is defined as small eccentric compression member, the next step should also be to calculate required reinforcement. For the purpose to calculate reinforcement, the limit state is the key to determine the functions, which is quite different between large eccentric compression member and small eccentric compression member. The limit state of large eccentric compression member is that the reinforcement in tension zone is yield and concrete strain achieves limit height of compression block in the same time. But for the small eccentric compression member, when the member achieves the axial force capacity, the reinforcement in the tension zone could not achieve the yield strength. That requires to calculate the stress of reinforcement in the tension zone in the limit state, which would not be equal to f_y. It is back to the same procedure as large eccentric compression member; calculate the height of the concrete compression block, x, then calculate strain of reinforcement in the tension zone by using stress-strain relationship and ε_{cu}. But GB 50010-2010 provides the approximate functions to calculate reinforcement area for small eccentric compression member if $A_s = A'_s$ (reinforcement area in the tension zone is equal to the reinforcement area in the compression zone), it could be expressed by

$$A_s = A'_s = \frac{P_u - \zeta(1 - 0.5\zeta)\alpha_1 f_c b h_0^2}{f'_y(h_0 - a'_s)}$$

Where the relative height of compression concrete block, ζ should be calculated when column eccentric types were verified. After estimating the reinforcement, GB 50010-2010 requires to check minimum reinforcement ratio and the compression resistance, which is as same as large eccentric compression member.

4.2. Column resistance design method in GB 50010-2010

There are two types of design method in ACI 318-14: sway frame design and non-sway frame design. For the purpose to verify the sway or non-sway fame, stability index, Q, should be calculated first, which could be expressed by

$$\mathrm{Q} = \frac{\sum P_u \Delta_0}{V_{us} l_c}$$

Where $\sum P_u$ is total vertical load in the story corresponding to the lateral loading case; V_{us} is factored story shear in the story corresponding to the lateral loading case; Δ_0 is first-order relative lateral deflection between top and bottom of the story due to V_{us}. If $\mathrm{Q} < 0.05$, this story level should be considered as a non-sway frame. If not, it should be consider as a sway frame.

Generally, the column design requires the all detailed loads and deflection analysis, which is quite different from GB 50010-2010. In addition, the stability index should be calculated due to each load combination provided in the ASCE 7-10. If there is anyone combination making the stability index, Q, exceed 0.05, this story should be considered as a sway frame.

A non-sway frame is the structure with small interstorey displacements. Once the design column's story level is estimated as a non-sway frame, the next step is to verify the slenderness effect could be neglected or not. In ACI 318-14, for columns not braced against sidesway, if $\mathrm{k}l_u/r \leq 22$, the slenderness effect could be neglected; for columns braced against sidesway, if $\mathrm{k}l_u/r \leq 34 + 12(M_1/M_2)$ and $\mathrm{k}l_u/r \leq 40$, the slenderness effect could be neglected. If the slenderness could be neglected, the column design should follow first-order analysis. If the slenderness effect could not be neglected, the column design should consider the slenderness effect through the column length, using the second order analysis. If $M_{2nd-order} \leq 1.4M_{1st-order}$, then use the $M_{2nd-order}$ as required design moment. If not, the structural system should be revised.

The sway frame is the structure capable to resist lateral loads by itself. It is not necessary to require additional bracing for stability; therefore, the sway frame would have a larger displacement rather than the non-sway frame. The first step for sway frame design is the same as non-sway design: the slenderness effect could be neglected or not. If slenderness effect could not be neglected, ACI 318-14 requires considering the slenderness effects at column ends first, which is second-order elastic analysis. The next step is to check the slenderness effect along the column length, which is as same as non-sway frame column. After these second-order analyses, the next step is to calculate the critical moment $M_{2nd-order}$, made sure $M_{2nd-order} \leq 1.4M_{1st-order}$, used the $M_{2nd-order}$ as required design moment, otherwise revised the structural system.

After determining the required axial force and moment resistance, they should be applied into the column strength interaction diagram for rectangular section to find out the required reinforcement ratio. But column strength is varied due to different axial force and moment. The resistance of rectangular cross-section is enveloped based on the equilibrium, stress-strain relationship of the materials. With applied eccentric axial load, the stress-strain distribution details through the cross-section are shown in Figure 3 from Maria M. Szerszen, Aleksander Szwed and Andrzej S. Nowak, 2005.

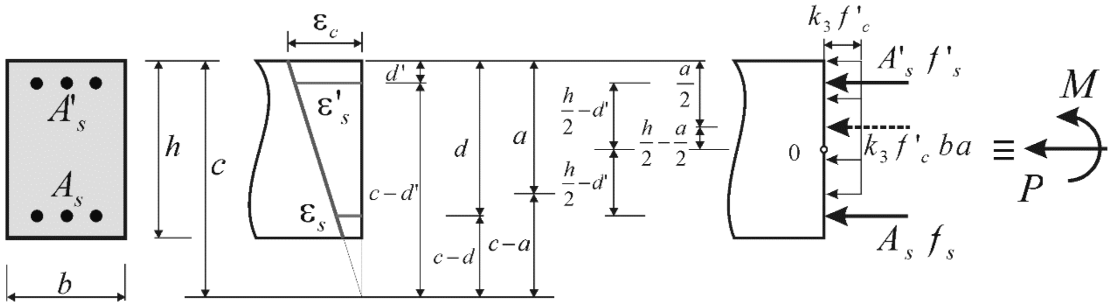

Fig 3. Strain and stress distribution in cross-section of an eccentrically loaded column.

All the columns were symmetric reinforced cross-section in this study, $A_s = A'_s$. The ultimate concrete compressive strain is assumed as $\varepsilon_{cu} = 0.003$. The concrete stress block is estimated with the depth of $\mathrm{a} = \beta_1 c$. If a part of cross-section is in compression, and the average compressive strength is equal to $k_3 f'_c$ (Maria M. Szerszen, Aleksander Szwed and Andrzej S. Nowak, 2005). Factor k_3 defines a ratio of the maximum stress in the compression zone of a cross-section to the cylinder strength of concrete (Maria M. Szerszen, Aleksander Szwed and Andrzej S. Nowak, 2005). For the concrete compression block, according to ACI 318-14, β_1 is taken as: $\beta_1 = 0.85$ for concrete strengths up to $f'_c \leq 20.5\, MPa$; $\beta_1 = 1.05 - f'_c/138$ for concrete strengths between $20.5\,\mathrm{MPa} < f'_c \leq 55.0\, MPa$; $\beta_1 = 0.65$ for concrete strengths greater than $f'_c > 55.0\, MPa$. The stress-strain relationship of reinforcement follows Hooke's Law in the elastic period, which is $f_s = E_s \varepsilon_s$, and keeps yield strength constantly in the elastic period.

The resistance functions of the cross-section are generated based on the equilibrium of axial force and moments, which could be expressed by (Maria M. Szerszen, Aleksander Szwed and Andrzej S. Nowak, 2005):

$$P = A'_s f'_s + A_s f_s + k_3 f'_c ba,$$

$$M = A'_s f'_s \left(\frac{h}{2} - d'\right) - A_s f_s \left(\frac{h}{2} - d'\right) + \beta f'_c ba \left(\frac{h}{2} - \frac{a}{2}\right)$$

The resistance of the column (moment and axial capacity) could be expressed as an interaction diagram, shown in Figure 4 (Maria M. Szerszen, Aleksander Szwed and Andrzej S. Nowak, 2005).

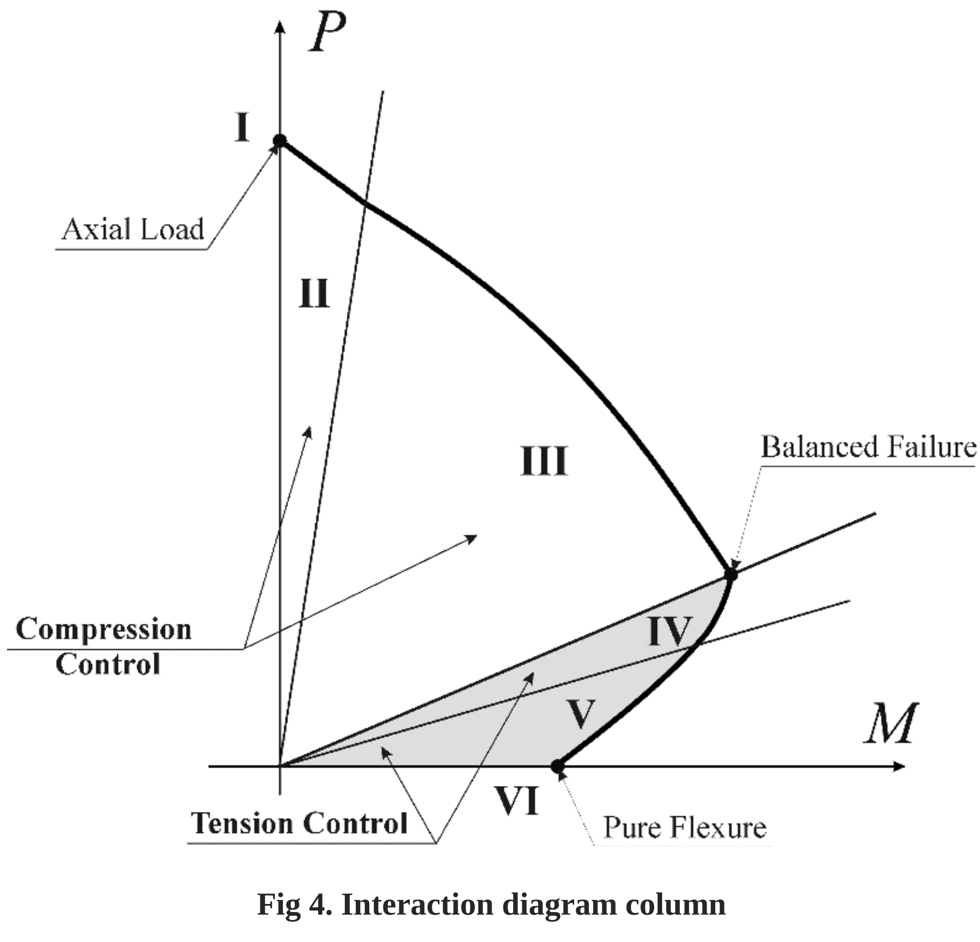

Fig 4. Interaction diagram column

There are seven control points. To simplify the analysis, interaction diagram between the adjacent two control points is assumed linear.

① Control point 1: Pure compression

In this situation, there is no moment resistance, assume that the tension reinforcement has no contribution to the axial force resistance, the compression reinforcement achieves the yield strength, the resistance function could be rewritten as:

$$\mathrm{P} = 0.85f'_c\left(A_g - A'_s\right) + f'_y A'_s$$

$$\mathrm{M} = 0$$

② Control point 2: Bar stress near tension face equal to 0 ($\varepsilon_s = f_s = 0$)

In this situation, the neutral axial depth, c, is equal to the distance from the neutral axis of tension reinforcement to the compression edge, d. Once the neutral axial depth, c, is determined, the strain in the compression reinforcement level could be estimated by using linear stress-strain relationship $\varepsilon'_s = (c - d')\frac{\varepsilon_{cu}}{c}$. According to the extreme strain of reinforcement $\varepsilon_y = \frac{f_y}{E_s}$, the compression reinforcement could be estimated as yield or not, then the stress of compression reinforcement can be calculated, f'_s, $(f'_s \leq f'_y)$. The resistance function could be rewritten as:

$$\varepsilon'_s = (c - d')\frac{\varepsilon_{cu}}{c}$$

$$C_c = 0.85f'_c ab$$

$$C_s = (f'_s - 0.85f'_c)A'_s$$

$$\mathrm{P} = C_c + C_s$$

$$\mathrm{M} = C_c\left(\frac{h-a}{2}\right) + C_s\left(\frac{h}{2} - d'\right)$$

③ Control point 3: Bar stress near tension face equal to $0.5f_y$ ($f_s = -0.5f_y$)

In this situation, tension reinforcement is not yielded, but the stress is already known, according to Hooke's Law, the strain of the tension reinforcement is also determined by $\varepsilon_s = {\varepsilon_y}/{2}$. The neutral axial depth, c, could be calculated by using the stress-strain relationship through the cross-section:

$$\mathrm{c} = \frac{d}{\varepsilon_s + \varepsilon_{cu}} \varepsilon_{cu}$$

After the neutral axial depth, c, is confirmed, the strain in the compression reinforcement level could be calculated by the following function:

$$\varepsilon'_s = (c - d')\frac{\varepsilon_{cu}}{c}$$

Next is to compare ε'_s with ε_y, to estimate the compression reinforcement yield or not, then gain the stress of compression reinforcement, f'_s, $(f'_s \leq f'_y)$. The resistance function could be rewritten as:

$$C_c = 0.85 f'_c ab$$

$$C_s = (f'_s - 0.85 f'_c) A'_s$$

$$T_s = f_s A_s$$

$$\mathrm{P} = C_c + C_s - T_s$$

$$\mathrm{M} = C_c \left(\frac{h-a}{2}\right) + C_s \left(\frac{h}{2} - d'\right) + T_s \left(d - \frac{h}{2}\right)$$

④ Control point 4: Bar stress near tension face equal to f_y $(f_s = -f_y)$

In this situation, tension reinforcement is just yielded, the strain of the tension reinforcement is equal to the limit yield strain, $\varepsilon_s = \varepsilon_y$. The neutral axial depth, c, could be calculated by using the stress-strain relationship through the cross-section:

$$c = \frac{d}{\varepsilon_s + \varepsilon_{cu}} \varepsilon_{cu}$$

After the neutral axial depth, c, is confirmed, the strain in the compression reinforcement level could be calculated by the following function:

$$\varepsilon'_s = (c - d') \frac{\varepsilon_{cu}}{c}$$

Next is to compare ε'_s with ε_y, so that it could figure out the compression reinforcement yield. Then the stress of compression reinforcement, f'_s, ($f'_s \leq f'_y$), could be confirmed. The resistance function could be rewritten as:

$$C_c = 0.85 f'_c ab$$

$$C_s = (f'_s - 0.85 f'_c) A'_s$$

$$T_s = f_y A_s$$

$$P = C_c + C_s - T_s$$

$$M = C_c \left(\frac{h-a}{2}\right) + C_s \left(\frac{h}{2} - d'\right) + T_s \left(d - \frac{h}{2}\right)$$

⑤ Control point 5: Bar strain near tension face equal to $0.005 \mathrm{in/in}$ ($\varepsilon_s = 0.005$)

The strain value $0.005 \mathrm{in/in}$ is the tension-controlled limit strain from ACI 318. In this situation, the strain of the tension reinforcement passes the limit yield strain. The neutral axial depth, c, could be estimated by using the stress-strain relationship through the cross-section:

$$c = \frac{d}{\varepsilon_s + \varepsilon_{cu}} \varepsilon_{cu}$$

After the neutral axial depth, c, is confirmed, the strain in the compression reinforcement level could be calculated by the following function:

$$\varepsilon'_s = (c - d') \frac{\varepsilon_{cu}}{c}$$

Next is to compare ε'_s with ε_y, so that it could figure out the compression reinforcement yield. Then the stress of compression reinforcement, f'_s, ($f'_s \le f'_y$), could be confirmed. The resistance function could be rewritten as:

$$C_c = 0.85f'_c ab$$

$$C_s = (f'_s - 0.85f'_c)A'_s$$

$$T_s = f_y A_s$$

$$\mathrm{P} = C_c + C_s - T_s$$

$$\mathrm{M} = C_c\left(\frac{h-a}{2}\right) + C_s\left(\frac{h}{2} - d'\right) + T_s\left(d - \frac{h}{2}\right)$$

⑥ Control point 6: pure bending

In this situation, the axial force resistance should be 0. It requires iteration to estimate the neutral axial depth, c, or stress block depth, a. Because the simulation would use Monte Carlo Method, it was not possible to iterate for every thousand simulations. So, the approximate function of stress block depth, a, was applied in this analysis (Maria M. Szerszen, Aleksander Szwed and Andrzej S. Nowak, 2005):

$$a = 2\sqrt{p}\cos\left[\frac{1}{3}\arccos\left(\frac{q}{\sqrt{p^3}}\right)\right] - u$$

$$u = \frac{A_2}{3A_1}, \qquad p = \left(\frac{A_2}{3A_1}\right)^2 - \frac{A_3}{3A_1}, \qquad q = \frac{A_2 A_3}{6A_1^2} - \left(\frac{A_2}{3A_1}\right)^3 - \frac{A_4}{2A_1},$$

$$A_1 = k_3 f'_c b, \qquad A_2 = k_3 f'_c b(2e - h),$$

$$A_3 = A_s E_s \varepsilon_c (2e - h + 2d') - A_s f_y (2e + h - 2d'),$$

$$A_4 = -A_s E_s \varepsilon_c \beta_1 d'(2e - h + 2d').$$

Since stress block depth, a, is confirmed, neutral axial depth, c, could also be calculated by $a = \beta_1 c$. By using the stress-strain relationship through the cross-section, the strain of tension reinforcement level could be calculated by:

$$\varepsilon_s = (d - c)\frac{\varepsilon_{cu}}{c}$$

Next is to compare ε'_s with ε_y, so that it could figure out the compression reinforcement is yielded or not. Then the stress of compression reinforcement, f'_s, $(f'_s \leq f'_y)$, could be confirmed. With the same principle, by using the stress-strain relationship through the cross-section, the strain of compression reinforcement level could be calculated by：

$$\varepsilon'_s = (c - d')\frac{\varepsilon_{cu}}{c}$$

The resistance function could be rewritten as:

$$C_c = 0.85f'_c ab$$

$$C_s = (f'_s - 0.85f'_c)A'_s$$

$$T_s = f_s A_s$$

$$P = C_c + C_s - T_s = 0$$

$$M = C_c\left(\frac{h-a}{2}\right) + C_s\left(\frac{h}{2} - d'\right) + T_s\left(d - \frac{h}{2}\right)$$

⑦ Control point 7: pure tension

In this situation, the load considered as concentric tension load, the tension strength of concrete is neglected, all the reinforcement achieved limit yield strain, and all of them are in tension. In addition, there is no moment resistance due to symmetric reinforced cross-section. The resistance function could be rewritten as:

$$P = f_y(A_s + A'_s)$$

The resistance of column will be determined according to the interaction diagram contributed by these seven-control points.

4.3. Statistical parameters of column resistance

The actual resistance of cross-section was controlled by six parameters: concrete compression strength f_c, reinforcement yield strength f_y, width of cross-section b, height of cross-section h, effective height of cross-section h_0, reinforcement ratio ρ. The reliability analysis for columns would use the Monte Carlo Method to simulate the actual resistance of each example. The statistical parameters for each factor are important for the analysis. The statistical parameters for GB 50010-2010 are shown in Table 11 (Xinyan Shao, Chongxi Bai, Wang Liang, Atlantis Press, 2015).

Table 11- Statistical parameter for column resistance in GB 50010-2010

parameter	nominal	bias factor	COV	distribution type
f_c	38.5 MPa	1.41	0.19	normal
f_y	400 MPa	1.12	0.093	normal
ρ	0.0045-0.01	1	0.05	normal
b	610 mm	1	0.02	normal
h	610 mm	1	0.02	normal
d	564 mm	1	0.03	normal

The statistical parameters approaching the reliability analysis for ACI 318-14 are shown in Table 12 (Maria M. Szerszen, Aleksander Szwed and Andrzej S. Nowak, 2005).

Table 12- Statistical parameter for column resistance in ACI 318-14

parameter	nominal	bias factor	COV	distribution type
f'_c	41.4 MPa	1.25	0.1	normal
f_y	413.67 MPa	1.145	0.05	normal
ρ	0.0045-0.01	1	0.015	normal
b	610 mm	1.05	0.04	normal
h	610 mm	1.05	0.04	normal
d	564 mm	1.05	0.04	normal

5. Reliability analysis

Reliability analysis starts from the limit state function. The general limit state function g for beam and column is the same, defined as:

$$g = R - Q$$

Where R is resistance; Q is load effect. The PDF (Probability Density Function) of g defines desired (safe) performance if $g \geq 0$ or undesired (not safe) performance if $g \leq 0$ (Maria M. Szerszen, Aleksander Szwed and Andrzej S. Nowak. 2005). In case of linear limit state function, the reliability index, β, can be expressed by the following formula (Nowak, A.S. and Collins, K.R 2013):

$$\beta = \frac{m_R - m_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}}$$

Where m_R is mean value of resistance; m_Q is mean value of load effect; σ_R is standard deviation of resistance; σ_Q is standard deviation of load effect. The reliability index in a general case of limit state function (non-linear) is defined as a function of the probability of failure, P_f , which is equal to the probability of occurrence of undesired performance (Nowak, A.S. and Collins, K.R 2013). For normal PDF's of resistance and load, the probability of failure is expressed as (Nowak, A.S. and Collins, K.R 2013),

$$P_f = \Phi(-\beta)$$

Where Φ is standard normal distribution function defined by the integral $\Phi(x) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{x} e^{-\frac{z^2}{2}} dz.$

The reliability index in this research was calculated by using Monte Carlo method.

Monte Carlo Procedure:

① Randomly generate values and calculate R .

② Randomly generate values of Q using their probability distribution.

③ Calculate g = R - Q.

④ Save the calculated value of g.

⑤ Repeat steps 1-4 until enough quantity of g values have been generated.

⑥ Plot the results on the probability paper and read the probability of g being negative.

The function of resistance, R, was using resistance functions from two codes, where they were introduced in chapter three and chapter four. The required nominal (design) resistance, R_n, is:

$$R_n = \frac{Q_n}{\phi}$$

Where Q_n is the factored load effect. ϕ is resistance factor; in ACI 318 ϕ is equal to 0.9 for flexure controlled reinforced concrete, 0.65 for compression controlled reinforced concrete for tied columns. In GB 50009-2012, ϕ is only related to the importance of building. For the examples in this study, it was constantly equal to 1. The mean resistance was determined with the statistical data stated in chapter three and chapter four. The nominal value of loads was based on the load combination from ASCE 7 and GB 50009-2012. The mean of load effect was calculated using the statistical data described in chapter two.

After the reliability index calculated, sensitivity analysis was applied. The sensitivity analysis is aiming to find out which factor had greater influence on the reliability index of structures. The analysis was performed taking into account the following parameters for beam examples: reinforcement ratio (0.2%, 0.3%, 0.4%, 0.5%, 0.6%, 0.7%, 0.8%, 0.9%, 1.0%), load ratio between live load and dead load plus live load ($L/D+L=$

$0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9$). For the column examples, the analysis was performed taking into account the following parameters: concrete strength ($f'_c = 30\ MPa, 40\ MPa, 50\ MPa, 60 MPa$), reinforcement ratio ρ (0.5%, 1%, 1.5%, 2%, 3%).

6. Comparison of Chines codes and American codes

6.1. Examples of reliability comparison

In this research, there were four pairs of comparison examples, totaling eight examples. One pair was beams, the other three were columns. Because the column resistance design method is entirely different in the two series' codes, it had to have three pairs of examples to cover all situations.

The beam examples were simple supported beam as roof beams, shown in Figure 5.

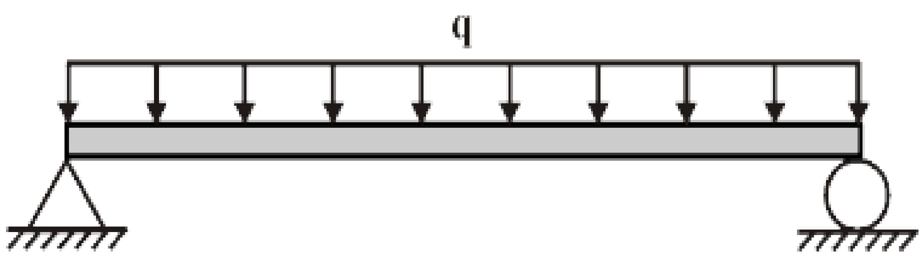

Fig 5. Calculation model: simple supported beam

The reliability analysis concentrated on the moment capacity at mid-span. There were two examples for beam comparison, which are listed as follows: Example 1 was designed by Chinese codes, Example 2 was designed by American codes. They were assumed to be parts of the three stories office buildings. The span length was determined as six meters. In addition, during the design procedure, the same dimensions were kept in this pair of examples. These two beam examples were assumed to be in the same situation -- 3-floor apartment beams. The roof type was a double slope roof. The slope angle was 15 degrees. In addition, Example 1 assumed located in Beijing, which would apply GB 50009-2012, "load code for the design of building structures" and GB 50010-2010, "code for the design of concrete structures"; Example 2 was assumed to be located in New York, which would apply ASCE 7-10, "Minimum design loads for buildings and other structures" and ACI 318, "Building Code Requirements for Reinforced Concrete". The reinforcement ratio of Example 1 and Example 2 would be calculated according to the two series' codes.

There was a total of 6 column examples. Example 3 and Example 4 were one pair of comparative examples. They were assumed to have the same surrounding conditions. Example 3 was designed by Chinese codes. Example 4 was designed by American codes. They were defined as small eccentric compression column and non-sway frame simultaneously. The assumption and load data of Example 3 and Example 4 was from Notes on ACI 318-08 example 11.1, the details of this example are shown in Figure 6. It was 10-story office building, the clear height of first story was 21ft-4in, and 11ft-4 in for other stories. The selected analysis column was Column C3, it was 24 x 24 in column, the concrete strength was $f'_c = 6000\,psi$, reinforcement yield strength was $f_y = 60\ ksi$. This office building applied the dead load, live load, roof live load and wind load. The computed load of column C3 based on ASCE7 is shown in Table 13.

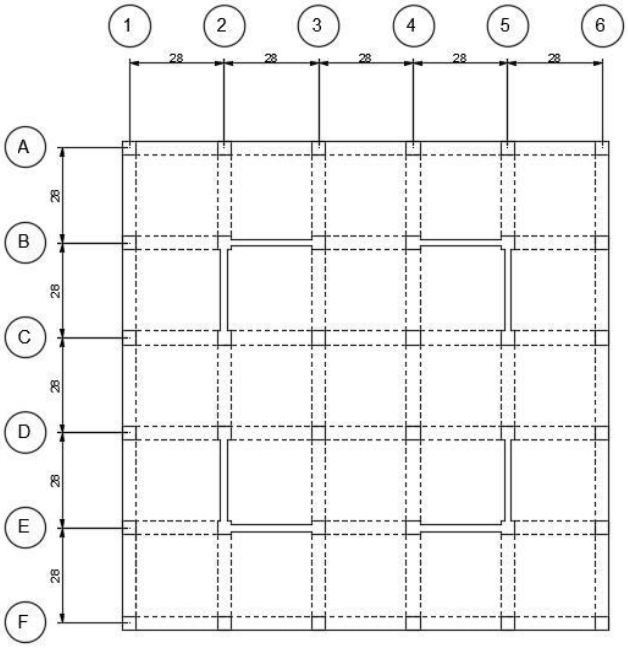

Fig 6. The dimension of first floor for Example 3 and Example 4

Table 13- The axial force and bending moment for Example 3 and Example 4

Load case	Axial load (kips)	Bending moment (kip-ft)	
		top	bottom
Dead	1269.0	1.0	0.7
Live	147.0	32.4	16.3
Roof live load	24.0	0.0	0.0
Wind	±3.0	±2.5	±7.7

Example 5 and Example 6 were another pair of comparative examples. Example 5 was designed by Chinese codes. Example 6 was designed by American codes. They were assumed to have the same condition. They were defined as small eccentric compression column and sway frame instantaneously. The design data of Example 5 and Example 6 was from Notes on ACI 318-08 example 11.2, the details of this example are shown in Figure 7. It was 12-story office building, the clear height of first story was 13ft-4in, and 10ft-4 in for other stories. The selected analysis column was Column C2; it was 24 x 24 in the column, the concrete strength was $f'_c = 6000\,psi$, reinforcement yield strength was $f_y = 60\,ksi$. This office building applied the dead load, live load, roof live load and wind load. The computed load of column C2 based on ASCE7 is shown in Table 14.

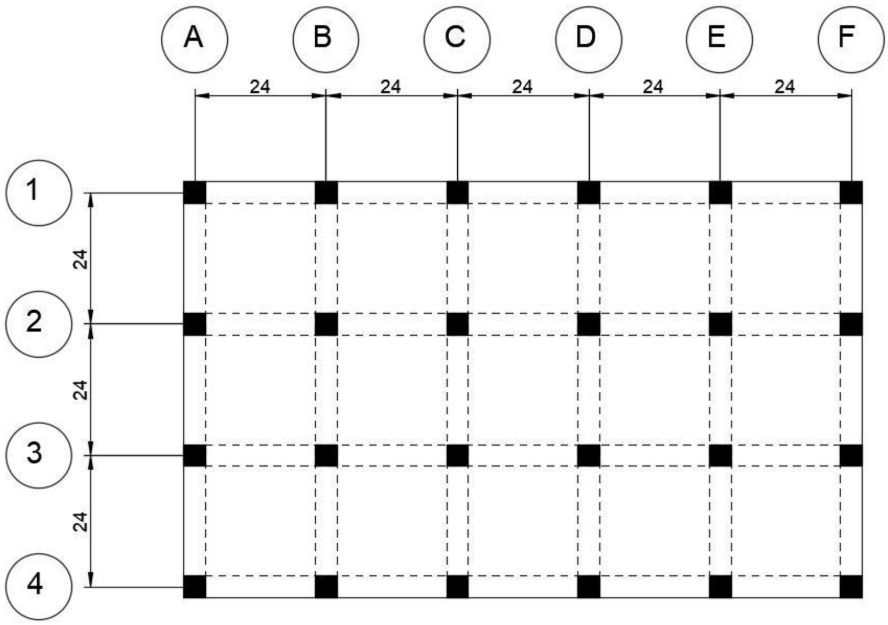

Fig 7. The dimension of first floor for Example 5 and Example 6

Table 14- The axial force and bending moment for Example 5 and Example 6

Load case	Axial load (kips)	Bending moment (kip-ft)	
		top	bottom
Dead	1087.6	-2.0	-1.0
Live	134.5	-15.6	-7.8
Roof live load	17.3	0.0	0.0
Wind (N-S)	-0.3	43.5	205
Wind (E-W)	0.3	-43.5	-205

Example 7 and Example 8 were the last pair of comparative examples. They were also assumed to have the same surrounding conditions and defined as large eccentric compression column and sway frame concurrently. Example 7 was designed by Chinese codes. Example 8 was designed by American codes. The design data of Example 7 and Example 8 was based on Notes on ACI 318-08 example 11.2. It was still a 12-story office building, the clear height of first story was 18ft-4in, and 10ft-4 in for other stories. The selected analysis column was Column C2; it was 24 x 24 in column, reinforcement yield strength was $f_y = 60\ ksi$, but the concrete strength was $f'_c = 4000\,psi$. This office building still applied the dead load, live load, roof live load and wind load. In addition, the moment load increased to two times of the loads in Example 5 and Example 6. The plan view kept the same as Figure 7. The load applied on the column C2 is shown in Table 15.

Table 15- The axial force and bending moment for Example 7 and Example 8

Load case	Axial load (kips)	Bending moment (kip-ft)	
		top	bottom
Dead	1087.6	-4	-2
Live	134.5	-31.2	-15.6
Roof live load	17.3	0	0
Wind (N-S)	-0.3	87	410
Wind (E-W)	0.3	-87	-410

6.2. Reliability indexes of Chinese codes

6.2.1. Reliability index of beam designed by Chinese codes

Example 1 was designed by Chinese codes; the design reinforcement ratio was 0.357%. After Monte Carlo simulation proceeded, the CDF (Cumulative Distribution Function) of limit state function g of Example 1 is shown in Figure 8 as follows:

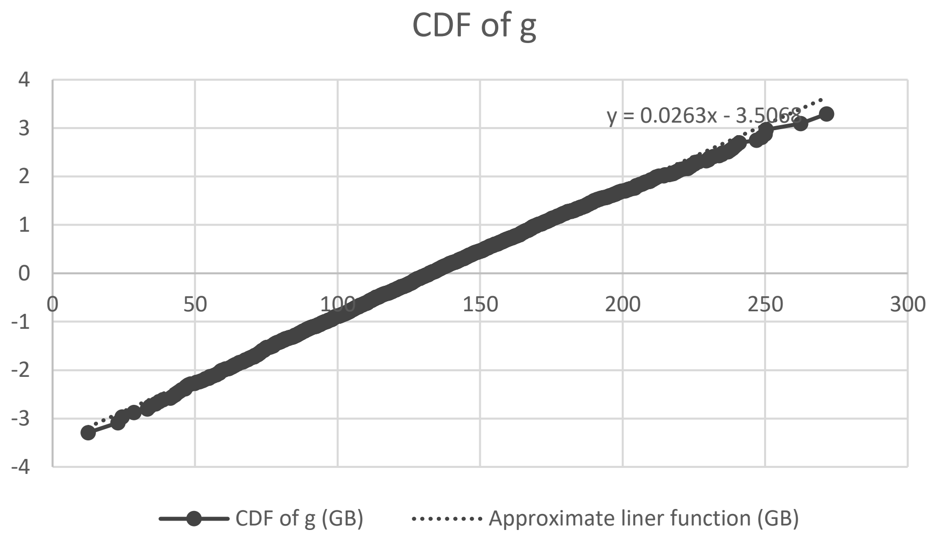

Fig 8. The CDF of g of Example 1

The reliability index performed perfectly at a 3.5, which means this design was economical with enough strength.

6.2.2. Reliability indexes of columns designed by Chinese codes

According to the requirements of GB 50010-2010, the proper reinforcement ratio of Example 3 was 0.6%, the CDF of limit function is shown in Figure 9.

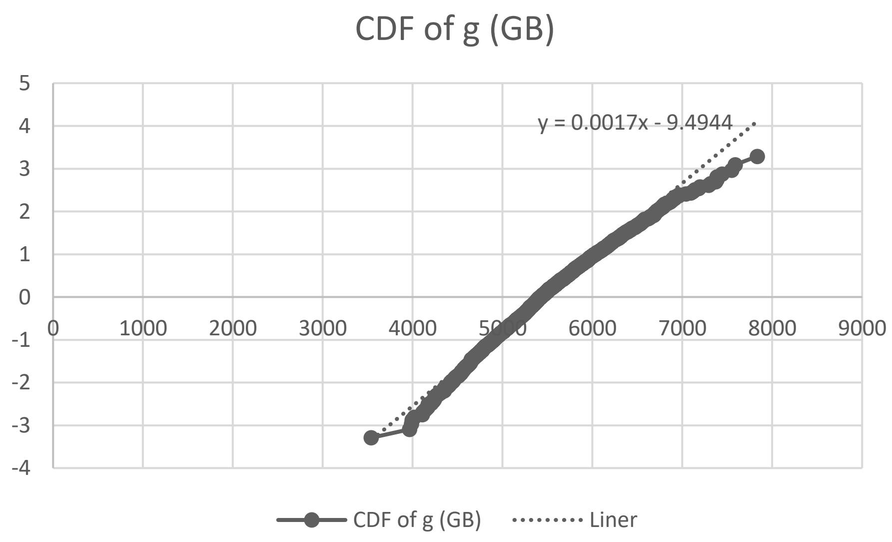

Fig 9. CDF of g for Example 3

The reliability index was extremely high, because the equilibrium did not determine the reinforcement ratio of Example 3; preferably, it was determined by the minimum reinforcement ratio requirement. If the minimum reinforcement requirement is neglected, the reinforcement ratio of Example 3 would be 0.2%, the CDF of the limit function in this situation is shown in Figure 10. The reliability index was 8.82, decreased a little bit from the previous result, but still a high value.

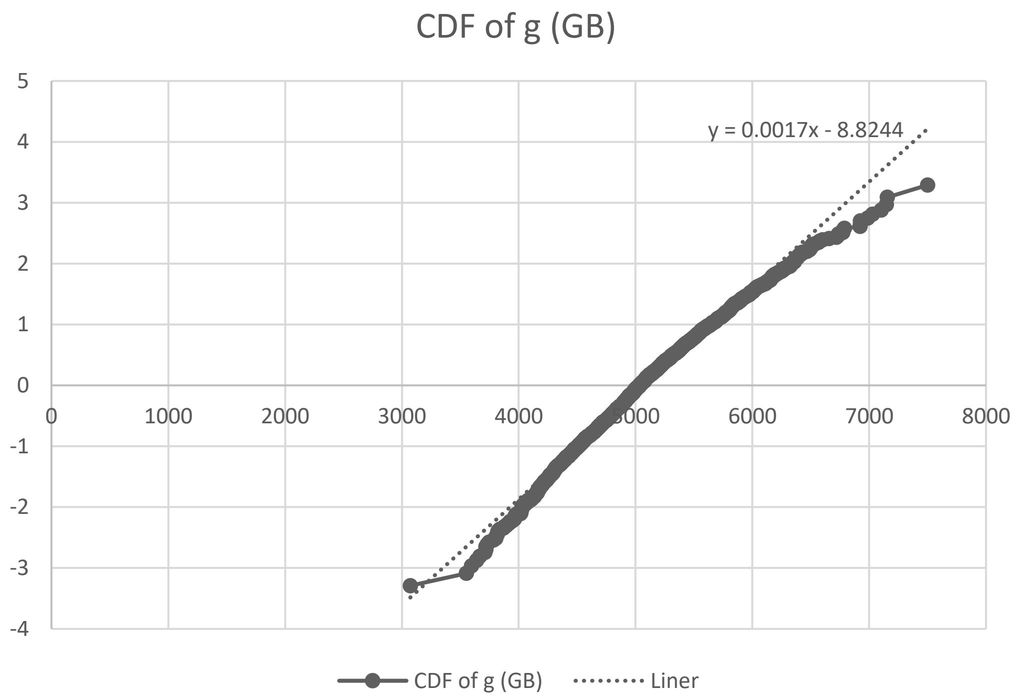

Fig 10. CDF of g for Example 3 neglected minimum reinforcement requirement

For Example 5, the reinforcement ratio was 0.8%, the CDF of limit function is shown in Figure 11. The reliability index was 8.90, which is also extremely high.

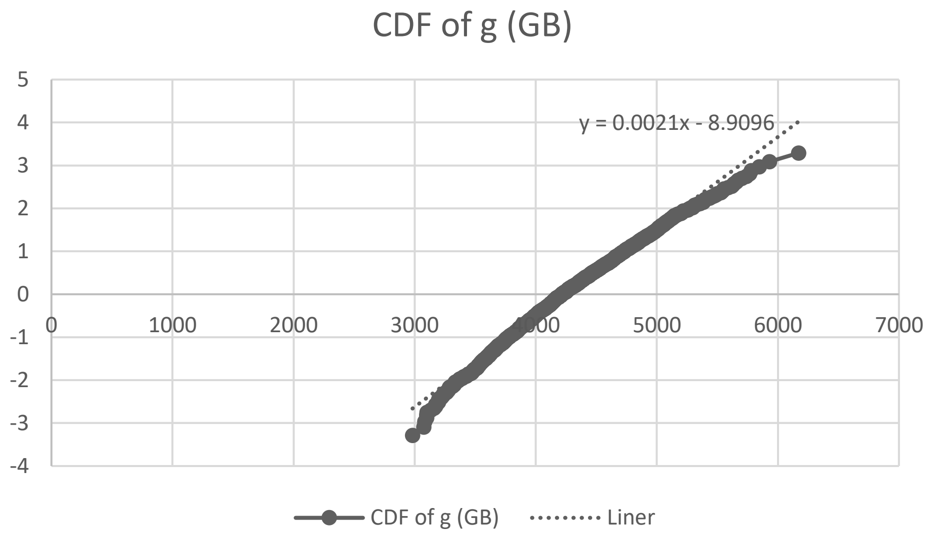

Fig. 11. CDF of g for Example 5

For Example 7, the reinforcement ratio was 2%, the CDF of limit function is shown in Figure 12. The reliability index was 7.24, which is still extremely high.

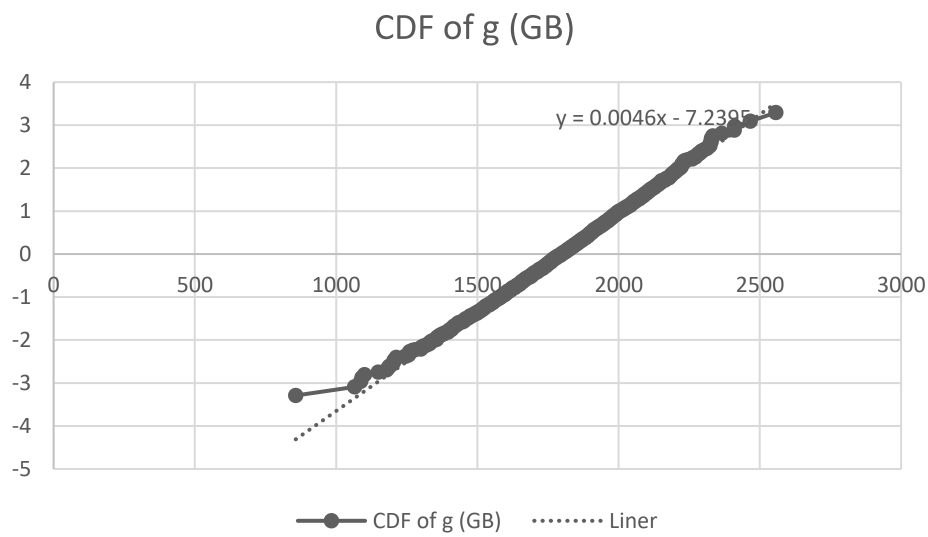

Fig 12. CDF of g for Example 7

6.3. Reliability indexes of American codes

6.3.1. Reliability index of beam designed by American codes

Example 2 was designed according to ACI 318 and ASCE 7. The reinforcement ratio was 0.2713%. The CDF of limit state function g for Example 2 is shown in Figure 13 as follows:

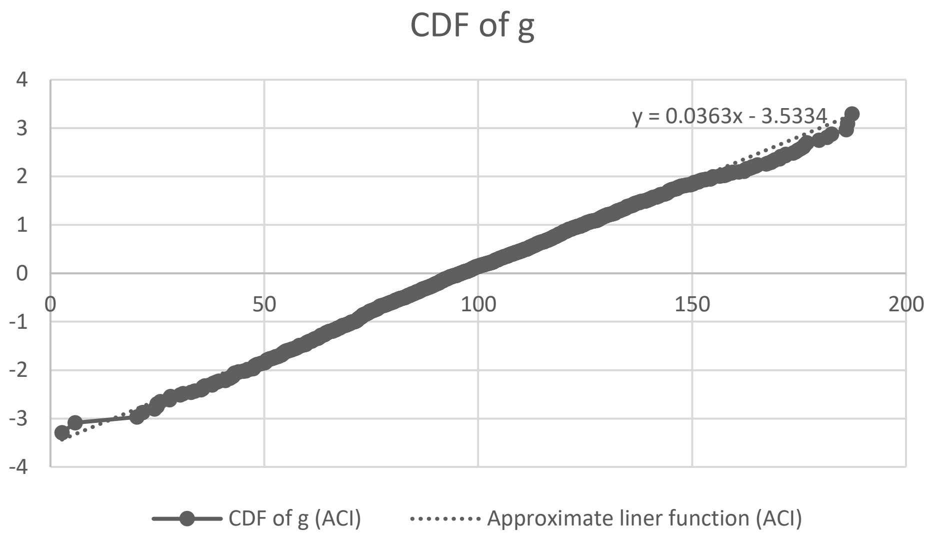

Fig 13. The CDF of g of Example 2

The reliability index performed close to 3.5. It was almost the same results as Example 1. Both examples were very close to 3.5. But the CDF of Example 1 was flatter than Example 2.

6.3.2. Reliability indexes of columns designed by American codes

According to the requirements in ACI 318, the reinforcement ratio of Example 4 was 0.017%. The CDF of limit function is shown in Figure 14. The reliability index was 3.37, which is a little below than 3.5.

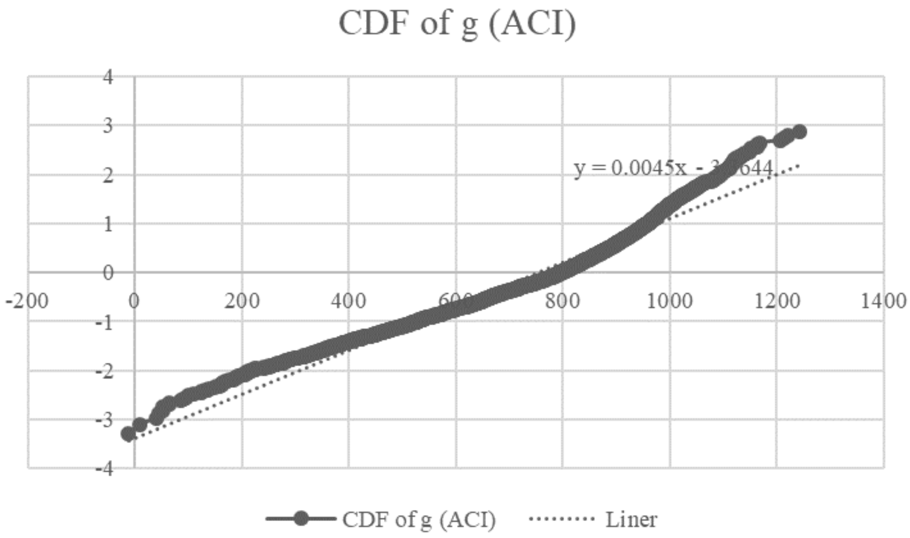

Fig 14. CDF of g for Example 4

The reinforcement ratio of Example 6 was 0.012%, the CDF of limit function is shown in Figure 15. The reliability index was 3.79, which achieves the requirement of 3.50.

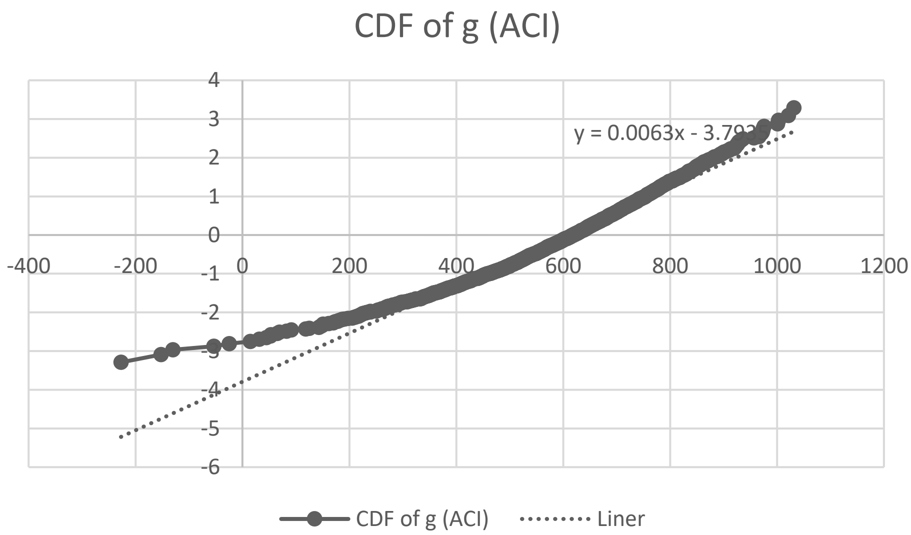

Fig 15. CDF of g for Example 6

The reinforcement ratio of Example 8 was 5.7%, the CDF of limit function is shown in Figure 16. The reliability index was 3.51, which is very close to 3.5.

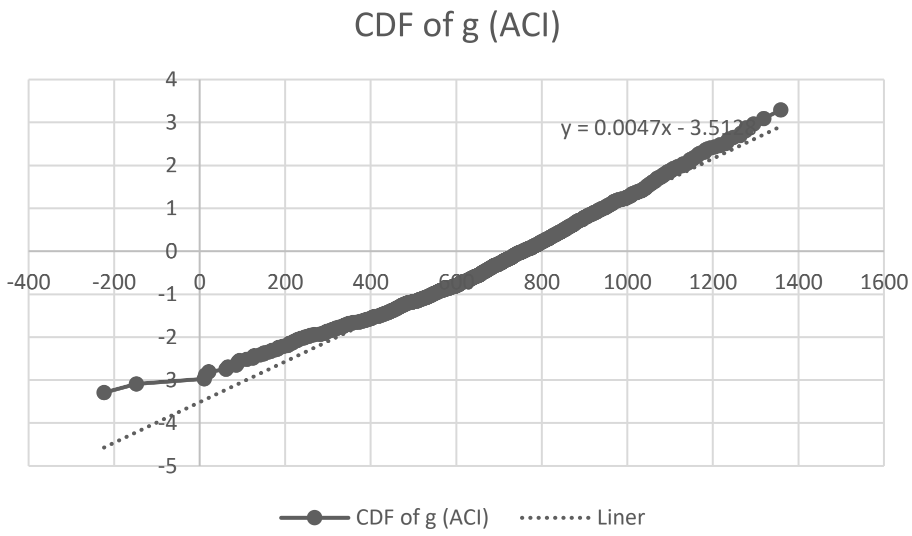

Fig 16. CDF of g for Example 8

6.4. Comparison of sensitive analysis

6.4.1. Sensitive analysis of beams

The sensitivity analysis applied on the beam examples was performed taking into account the following parameters: reinforcement ratio (0.2%, 0.3%, 0.4%, 0.5%, 0.6%, 0.7%, 0.8%, 0.9%, 1.0%), load ratio between live load and dead load plus live load ($L/D+L = 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9$).

The process of applying different reinforcement ratio is to analysis the resistance's varying influence on the reliability. The longitude reinforcement area has a significant contribution to the positive moment capacity of beams, which is presented by reinforcement ratio. In this sensitivity analysis, the nominal value and statistical parameter of loads did not change, the nominal values and statistical parameters of the reinforcement yield strength and concrete compression strength also did not change. The only changing factor was different reinforcement ratio (0.2%, 0.3%, 0.4%, 0.5%, 0.6%, 0.7%, 0.8%, 0.9%, 1.0%). The reliability results applied different reinforcement ratio is shown in Table 16 and Figure 17.

Table 16- Reliability index for different reinforcement ratios

reinforcement ratio	0.002	0.003	0.004	0.005	0.006	0.007	0.008	0.009	0.01
Example 1 (GB)	0.69	2.79	4.16	5.04	5.54	6.06	6.24	6.41	6.68
Example 2 (ACI)	2.04	4.08	5.17	6.01	6.4	6.64	6.93	7.13	7.28

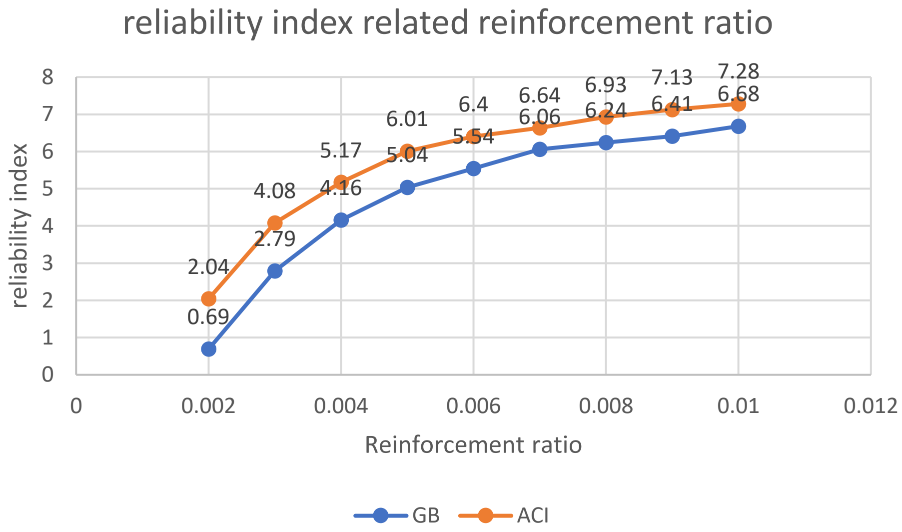

Fig 17. Reliability index for different reinforcement ratios

Comparing the results, the slop of the reliability variety was similar to each other. The design reinforcement ratio of Example 1 was 0.357%, the design reinforcement ratio of Example 2 was 0.2713%. When the reinforcement area was less than the requirements, the reliability decreased rapidly. When the reinforcement area increased upon the requirements, the reliability index did not rise rapidly as it did when below the requirements. Reliability index with proper reinforcement ratio of Example 1 was in the area with a smaller slope compared with reliability index with proper reinforcement ratio of Example 2, which means the design of American codes is more sensitive than the design of Chinese codes on reinforcement area variety in the appropriate range.

Applying different load ratio is used to analyze the load effect influence on the reliability.

Live load and dead load mainly control the required resistance of the beams. Changing the ratio between dead load and live load could show the reliability index reflection on the variety of the load ratio. In this sensitivity analysis, the resistance factors for Example 1 and Example 2 kept the same value as the original analysis and kept the quantity of dead load plus live load constant, then the ratio of live load versus dead load plus live load (ratio equal to 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9) was the only changing factor. The results of reliability indexes applied differently the ratio of live load versus dead load plus live load is shown in Table 17 and Figure 18.

Table 17- Reliability index for ratio of live load versus dead load plus live load

L/D+L	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
Example 1 (GB)	2.87	3.19	3.44	3.66	3.79	4.14	4.31	4.53	4.68
Example 2 (ACI)	2.37	2.78	3.18	3.52	4.03	4.29	4.4	4.61	4.74

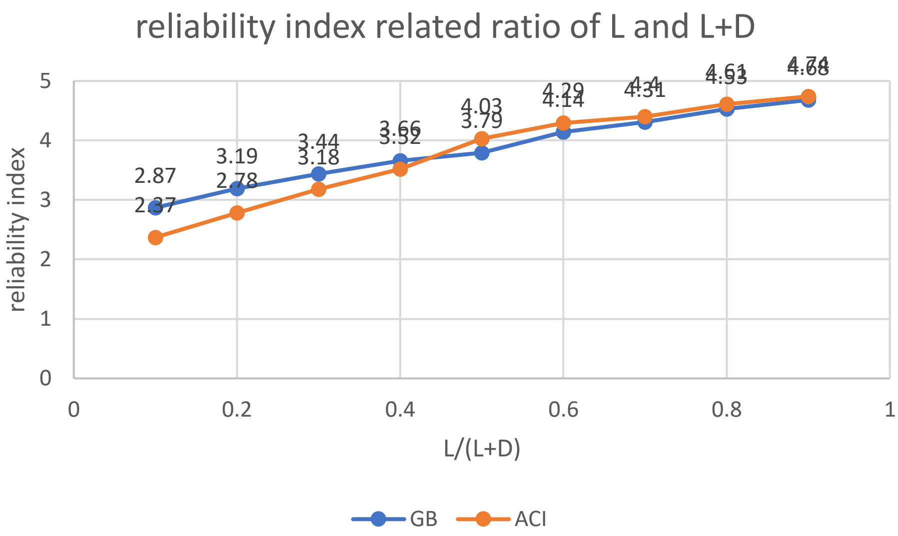

Fig 18. Reliability index for ratio of live load versus dead load plus live load

Overall the Figure 18, the slope of Example 1 was a little flatter than the slope of Example 2. The ratio of live load versus dead load plus live load in the original design was 0.475. The intersection of two lines in Figure 18, was close to the original design point. In the range closed to the actual load ratio, the slope of Example 1 was significant smaller than the slope of Example 2, which means the design based on American codes was more sensitive than the design based on Chinese codes in the real load ratio range.

6.4.2. Sensitive analysis of columns

The sensitivity analysis for columns was performed taking into account the following parameters: concrete strength ($f'_c = 30\ MPa, 40\ MPa, 50\ MPa, 60MPa$), reinforcement ratio ρ (0.5%, 1%, 1.5%, 2%, 3%).

Concrete provides a significant contribution to the axial compressive force resistance. The application of different strength of concrete is to figure out how it influences the reliability. In this sensitivity analysis, Example 3 and Example 4 were chosen as the primary pair of analysis examples, the nominal value and statistical parameter of loads did not change, the nominal values and statistical parameters of the reinforcement yield strength and reinforcement area also did not change. Example 3 and Example 4 applied different concrete strength ($30\ MPa, 40\ MPa, 50\ MPa, 60MPa$). The reliability indexes result due to different concrete strength is shown in Table 18 and Figure 19.

Table 18- Reliability index due to different concrete strength of Example 3 and Example 4

concrete strength (Mpa)	30	40	50	60
Example 3 (GB)	8.76	8.43	8.14	7.36
Example 4 (ACI)	3.87	3.66	3.34	3.15

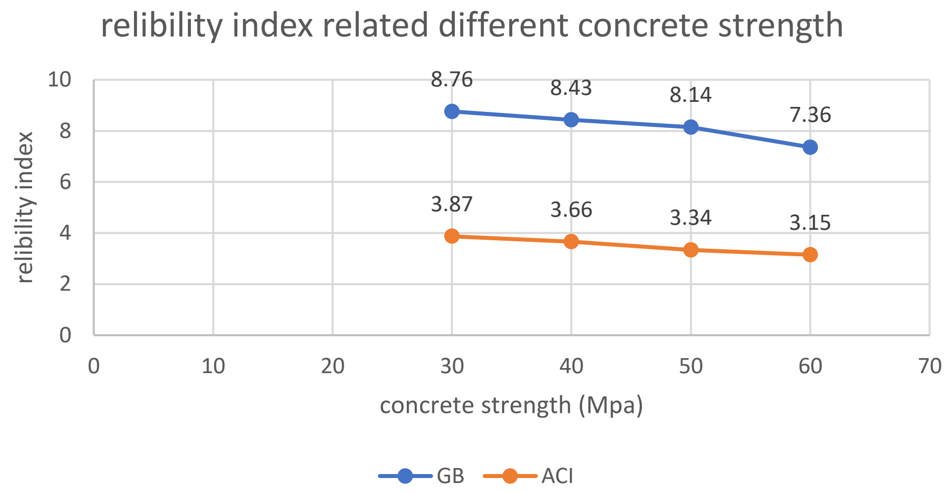

Fig 19. Reliability index due to different concrete strength of Example 3 and Example 4

Overall the Figure 19, the slopes of two examples were similar, and the columns had better reliability when the concrete had a lower concrete strength. It means the column failure with less concrete strength is more closed to area of the tension-controlled, the column failure with higher concrete strength is more closed to the area of compression-controlled.

After sensitive analysis due to concrete strength, the sensitive analysis due to reinforcement area was applied. Reinforcement is another contributor to the capacity. In this sensitivity analysis, Example 3 and Example 4 were chosen for the analysis. The nominal value and statistical parameter of loads kept the constant, the nominal values and statistical parameters of the reinforcement yield strength and concrete strength also did not change. Example 3 and Example 4 applied different reinforcement ratio (0.5%, 1%, 1.5%, 2%, 3%). The reliability results due to different reinforcement ratio are shown in Table 19 and Figure 20.

Table 19- Reliability index due to different reinforcement ratio of Example 3 and Example 4

reinforcement ratio	0.005	0.01	0.015	0.02	0.025
Example 3 (GB)	8.67	8.74	8.89	8.91	8.96
Example 4 (ACI)	3.36	3.52	3.83	3.97	4.1

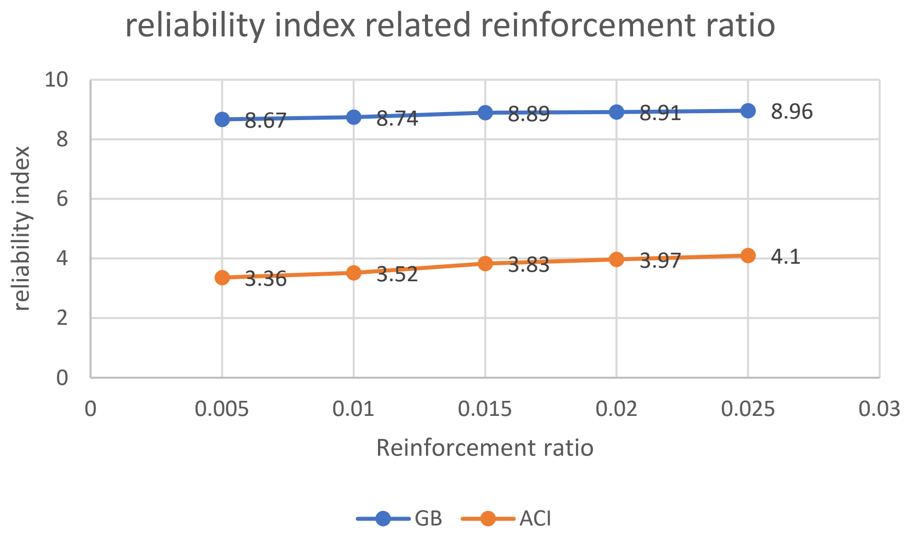

Fig 20. Reliability index due to different reinforcement ratio of Example 3 and Example 4

Overall the Figure 20, the slopes of two codes were similar and flat. The reliability index variation was not significant due to the reinforcement ration increasing. Rather than the sensitive analysis due to the concrete strength, the reliability index is more sensitive to the concrete strength.

7. Conclusions

In this dissertation, beams and columns were the main discussed structures for reliability analysis. For the analysis of beams, the two examples created were designed by different codes. In addition, the sensitivity analysis was applied to the examples with various reinforcement areas and different live load ratios. For the analysis of columns, there were six examples, to ensure coverage the two different design methods in the two different codes (non-sway column design and sway column design; small eccentric compression design and large eccentric compression design). Through the analysis procedure, there were some complications were determined as noted by the following comments:

(1) Both American and Chinese codes could get very excellent performance at the beams' reliability with proper reinforcement ratio. But the American codes are more sensitive to the reliability changing due to load ratio changing.

(2) Chinese code has an entirely different load combination in the design steps. It is also based on Turkstra's Rule, but for each variable, the factor number is not same as ASCE 7-10. Usually the load combination according to Chinese code will get a little larger value.

(3) The most different part about load is live load, the design live load (nominal value) is not same. ASCE's value is larger than GB's value. In addition, in Chinese code live load separates into two categories, permanent live load and temporary live load. These two types of live load have entirely different statistical parameters.

(4) In Chinese code GB 50010-2010, the column design procedure does not consider about the structures are sway frame or non-sway frame. Instead the Chinese code

directly enlarges the eccentricity to gain a larger required moment resistance.

(5) Through the analysis of column models, the reliability index of the columns designed by Chinese code was extremely high due to ϕ factor. But the resistance factor ϕ in two series' codes for resistance is not same. In Chinese code, it depends on the importance of the building, from 0.9 to 1.1. But in American code, it is 0.9 for tension-controlled failure, 0.65 for compression-controlled failure. This difference has a significant influence on the reliability index, especially in the column reliability analysis.

Overview this research; the Chinese code design method is a little more conservative than American code on the column design. For the load, even though the load combination is different, but the results are similar. Only on the live load was there is a significant difference: it is a more detailed category than in the American code. As for the resistance factor ϕ, it should have more research on it for Chinese code.

References

ACI318. *Building Code Requirements for Reinforced Concrete.* Detroit: American Concrete Institute Standard Committee 318, 2014.

ASCE7-10. *Minimum design loads for buildings and other structures.* Reston: American society of civil engineers, 2010.

Bartlett, F.M., H.P. Hong, and W. Zhou. "Load factor calibration for the proposed 2005 edition of the National Building Code of Canada: Statistics of loads and load effects." *Can. J. Civ. Eng.*, 2003: 30(2): 429-439.

Bazovsky, Igor. *Reliability Theory and Practice.* New York: Dover Publications. INC, 1961.

Ellingwood, Bruce R., and Paulos Beraki Tekie. "Wind load statistics for probability-based structural design." *J. Struct. Eng*, 1999: 125(4): 453-463.

GB50009-2012. *Load Code for Deign of Building Structures.* Beijing: Ministry of Housing and Urban-Rural Development of the People's Republic of China, 2012.

GB50010-2010. *Code for Design Concrete Structures.* Beijing: Ministry of Housing and Urban-Rural Development of the People's Republic of China, 2010.

GB50153-2008. *Unified Standard for Reliability Design of Engineering Structure.* Beijing: Ministry of Housing and Urban-Rural Development of the People's Republic of China, 2008.

Hou, Bonian, and Caiang Wei. "Statistical Analysis of Snow Loads in the Unified Standard for Building Structure Design." *Industrial Construction*, 1982: 12 (2): 61-64.

Lee, Kyung Ho, and David V. Rosowsky. "Site-Specific Snow Load Models and Hazard Curves for Probabilistic Design." *Nat. Hazards Rev*, 2005: 6(3): 109-120.

Li, Zhenchang, and Jiayan Wang. "Statistical Analysis of Dead Loads in the Unified Standard for Building Structure Design." *Industrial Building*, 1982: 12 (3) :55-58.

Nowak, A.S., and K.R. Collins. *Reliability of Structures.* New York: CRC Press, 2013.

Nowak, Andrzej S., and Maria M. Szerszen. "Calibration of Design Code for Buildings (ACI 318) Part 1: Statistical Models for Resistance." *Structural Journal* (University of Michigan), 2003: 377-382.

Shao, Xinyan, Chongxi Bai, and Wang Liang. "Statistical Resistance and Reliability Analysis of Steel-Concrete Composite Beams." *Proceedings of the International Conference on Advances in Energy, Environment and Chemical Engineering.* Changsha, China: Atlantis Press, 2015. 545-549.

Szerszen, Maria M., Aleksander Szwed, and Andrzej S. Nowak. "RELIABILITY ANALYSIS FOR ECCENTRICALLY LOADED COLUMNS." *ACI Structural Journal*, 2005: 676-

688.

Tu, Xiangyuan. "Statistical Analysis of Wind Loads in the Unified Standard for Building Structure Design." *Industrial Buildings*, 1982: 12 (2): 57-60.