Monte Carlo Simulation on Ultimate Bending Capacity of Hybrid Composite Girder

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ABSTRACT

In the past, most bridges in Japan were designed using the ASD method (Allowable Stress Design Method). On the other hand, other countries, like America and China, take the LSD method (Limit State Design Method) as the main design method of bridges and other structures. Moreover, After the Japanese economic bubble burst, the Japanese economy came to the great depression, and the Japanese construction industry also faced great challenges, which required to save material using and low construction cost. Since the ASD method requires that all sections meet the maximum stress requirements, it will cost more material.

Due to the external environment of the construction industry and the domestic economic situation, in order to solve above problems, a new structure called Hybrid composite girder (HCGs) is developed to design bridge structures. Hybrid composite girder is a kind of composite girder (NCGs) which contains a hybrid steel girder and a concrete slab. The hybrid steel girder is composed of a lower flange made by high-strength steel while the upper flange and web are made of normal steel. The objective of this study is to find a method to value the structural reliability and assess the structural applicability by comparing different cases. This study does numerical experiments based on Monte Carlo Method contributing the statistic models. Furthermore, Fiber Method and FEM (Finite Element Method) are adopted to value the structural reliability.

Firstly, in the Chapter 2, the basic contend of computing method is introduced, including the calculation procedure of Fiber Method, the basic ideas of Finite Element Method and the application of the Monte Carlo Method. The usage method of Fiber Method and Finite Element Method are imported in this section, and this section also discusses the relation variable numbers and convergency about Monte Carlo Method.

The numerical models for simulation are showed in the Chapter 3. Three different section dimensions (slender, standard and compact) used in Fiber Method calculation and the two kinds of element type (solid element and shell element) used in Finite Element Method are introduced.

Chapter 4 is the most important section in this thesis. In Chapter 4, due to the statistical calculation, the random variables of material and section dimensions are generated to increase the reliability of the results. The influence of section dimensions on structural reliability is discussed in this part, and the factors, which influence the structural reliability, are tried to be found.

Chapter 5 shows the results of simulation based on Fiber method and Finite Element method. The result of bending ultimate calculation is outlined in Chapter 5.2 and the buckling ultimate calculation is outlined in Chapter 5.3. Based on the statistical results and reliability calculation, the reliability of bending capacity about each model could be compared quantitatively. It could be concluded that buckling is an important factor affecting structural stability. Buckling analysis should be considered for structures which contain thin shells part. And the compact model shows to have a better performance to restrain buckling, and compact NCGs shows to have a higher reliability among those situations.

Conclusions and future work are outlined in Chapter 6.

FIGURE INDEX

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Chapter 1. INTRODUCTION

1.1 Background

Civil engineering, as the basis of living and acting of humanbeings, its safety is an eter nal topic. As an empirical discipline, civil engineering has a long history and is widely used, but there is a lack of systematic theoretical system for itself. Since modern times, numerous scientific research achievements have provided theoretical basis for civil engineering discipline. Due to the development and application of new materials, the span, height and strength of constructions have been greatly improved. Therefore, higher requirements are put forward for structural safety, rationality and reliability in the construction industry.

In the past, ASD method (Allowable Stress Design Method) was used for the structure design in the elastic design of steel bridges in Japan based on the yield point. ASD method, which is a kind of stress-based design method, stipulated that the maximum stress on the section of structural member in service stage shall not exceed the allowable stress of material. This method does not consider the nonlinear properties of materials, and ignores the difference between the actual bearing capacity of the structure and the results calculated by elastic method. The values of load and allowable stress of materials are determined by experience and lack of scientific basis. On the other hand, overseas countries take LSD method (Limit State Design Method) as the mainstream design. LSD method is a kind of performance-based design method, which divided the limit state of structure into bearing capacity limit state and serviceability limit state. The former requires that the minimum bearing capacity of the structure should not be less than the section internal force caused by the possible maximum external load. The latter limited the deformation and crack formation or cracking degree of structure. It could be referred that LSD method is showed to be a more reasonable design method. It is known that "the Road Bridge Specifications[1)]" is widely used to design bridge structures in Japan. In 2017, it imported LSD method of the background of widely using LSD method in international bridge design. The LSD method considers one or more than one safety factors, like the variation of load, material properties and working conditions.

On the other hand, after the Japanese economic bubble burst, the Japanese economy came to the great depression, and the Japanese construction industry also faced great challenges, which required to save material using and low construction cost, there are lots of attempts for the new material exploitation and reducing costs. According to different countries, however, the allowable values of steel performance and manufacturing error are also different, there is no reference from other situations. Hence, it's necessary to do the research independently based on existing design, fabrication standard and material characteristics to find a new design method.

Therefore, due to the external technical conditions of innovation environment and the plight of the internal construction industry, in order to solve above problems, a new structure called Hybrid composite girder (HCGs) [2), 3)] is developed to design bridge structures. Hybrid composite girder is a kind of composite girder (NCGs) which contains a hybrid steel girder and a concrete slab. The hybrid steel girder is composed of a lower flange made by high-performance steel while the upper flange and web are made of normal steel. Since the lower flange is regarded as the most important member for restraining tension stress, and high-performance steel has a higher yield stress, HCGs could be considered as a rational structure. And the two models are widely used in the bridge structures design. Both of two structures are showed in **Figure** 1.1.

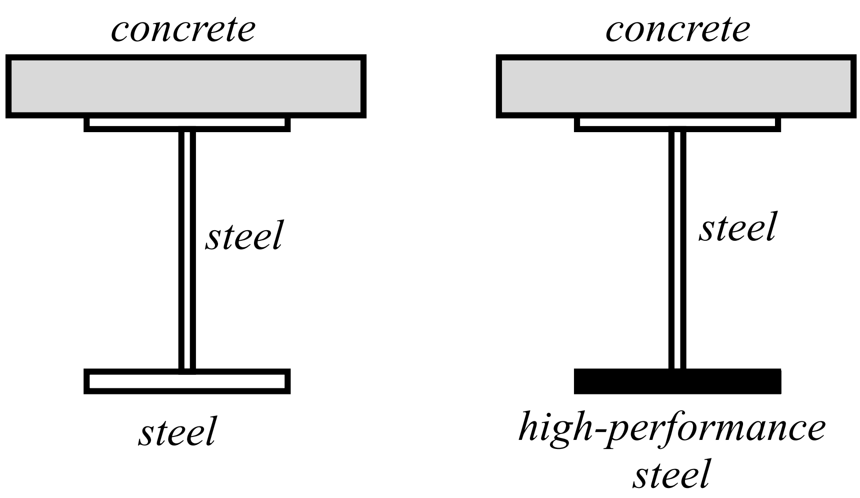


Figure 1.1 *Conceptual model of HCGs and NCGs*

For these situations, the applicability and rationality of HCGs and NCGs need to be fully and scientifically discussed.

1.2 Previous researches on HCGs

The research on Hybrid composite girder has been doing for decades and it comes out primary result. An existing study showed that HCGs are feasible and practical structures.[4)]. In that study, excellent bending resistance performance is shown at the end of the hybrid composite girders. In the experiment, local buckling, lateral buckling, and shear buckling of web in shear span are showed to do not occur. And the failure is confirmed as the crush of upper edge about concrete slab. The full plastic bending moment, defined as M_p, (**Figure** 1.2) is regarded as the simple design basis to value the bending capacity.

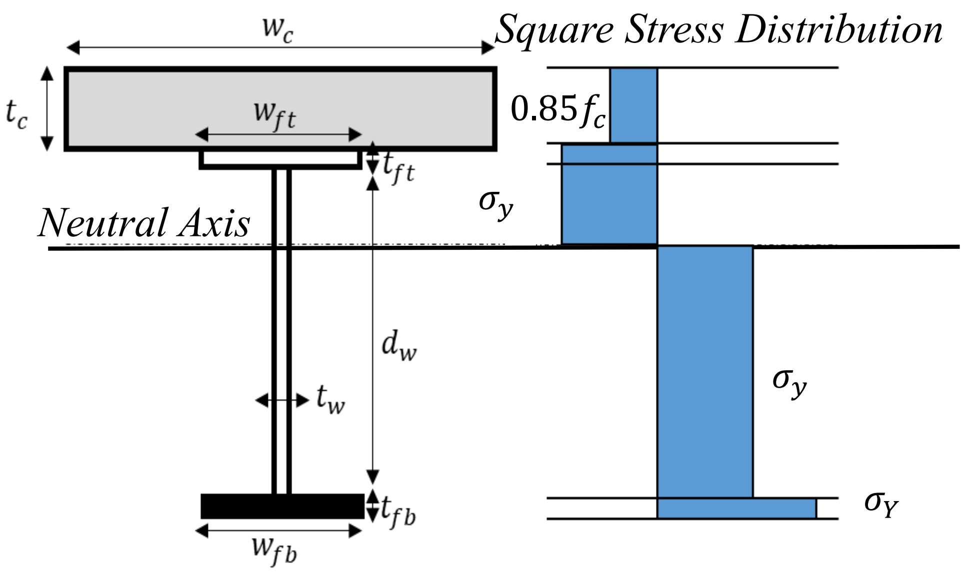


Figure 1.2 *Rectangular stress distribution during full plastic bending moment*

All of the previous research are about to do the simulation calculate the bending capacity to value the structural reliability.

1.3 Objective

It is said that hybrid composite girders could reduce 10% use of steel whereas normal composite girders could reduce 20% use of steel[5)]. Thus, it's difficult to conclude that the HCGs have superiority than the NCGs in all factors. Both of hybrid composite girders and normal composite girders are suggested to be reasonable structures. It's difficult to predict the load carrying behavior of structures with nonlinear regions of materials. Additionally, there is uncertainty in the material and the members.

In order to take advantage of LSD method, the extent of the tolerance to the load carrying behavior of the structure should be clarified theoretically. The method of obtaining the probability distribution of the bending capacity is necessary. For the composite girder, there are some examples[6-8)] of the bending capacity distribution based on the variation of the material parameters.

In the reference 6), there is a new design method for calculating structural bending capacity with the probability variations of material parameters by using Monte Carlo simulation. In this study, three kinds of typical probability density function, like Normal Distribution, Log-normal Distribution and Weibull Distribution, are selected to set variations. Using statistic calculation to evaluate the bending capacity. Thus, there are still several problems need to be solved and the objectives of this research are to find a method to calculate the reliability of composite girder and search the relation between the section dimension and structural reliability.

1.4 Overview

Chapter 1 is a general introduction of this research. Relevant previous studies and objective of this thesis are introduced after the background.

In Chapter 2, the basic contend of computing method is introduced, including the calculation procedure of Fiber Method, the basic ideas of Finite Element Method. Both two simulation method are used combined with Monte Carlo Method.

In Chapter 3 introduces the numerical models for simulation. Three different section dimensions (slender, standard and compact) used in Fiber Method calculation and the two kinds of element type (solid element and shell element) used for Finite Element Method modeling.

In Chapter 4, due to the statistical calculation, the random variables of material and section dimensions are generated to make the results more reliable. The influence of section dimensions on structural reliability is discussed in this part, and the factors, which influence the structural reliability, are tried to be found.

The simulation results and discussions are showed in Chapter 5. Statistical calculations are adopted to do the simulation, and the structural reliability could be quantitative analysis by calculating the reliability index and failure probability.

Conclusions of above work and future work are presented in Chapter 8.

Chapter 2. COMPUTING METHOD

2.1 Introduction

To quantitatively analyze the structural reliability, numerical experiment should be done to calculate the bending capacity of each model. This chapter provides computing methods to calculate structural bending capacity. During the analysis of the linear elastic stage, the Fiber Method is adopted to calculate the bending capacity. Furthermore, the Finite Element Method (FEM) is a numerical method for modeling continuum with finite element and is accepted to do the simulation in the nonlinear and elastoplastic stage.

The structure of this chapter is showed as following. In section.2, it will state the application of both two simulation method for the numerical experiment. Section.3 introduce the Monte Carlo Method and the combination between the Monte Carlo Method and both two computing method.

2.2 Bending Capacity

2.2.1 Fiber Method

The Fiber Method is adopted to calculate the bending capacity of hybrid composite girders (HCGs) and normal composite girders (NCGs).

The Fiber Method is a method to calculate the axial force and the bending moment by cutting the total length into small parts. First, the crushing of the upper edge of concrete slab is defined as the ultimate limit state. For the fixed ultimate strain, the position, where the axial force ($d_y \times S \times \sigma_y$) equals zero, is regarded as the neutral axis of the structure (**Figure** 2.1). Then calculate the bending moment, which is defined as ultimate bending moment M_u.

However, the plane holding assumption remains true, and the steel girders are regarded as not occurring local buckling or lateral buckling and other brittle failure phenomenon. And the bending capacity could be calculated as M_u/M_p. By dimensionless the capacity, it can be used to quantitatively analyze the structural reliability, so that the difference of section dimensions of structures can be neglected. The larger M_u/M_p is, the more the strength of the material could be fully utilized. The computing process[6)] of the Fiber Method is showed in **Figure** 2.2.

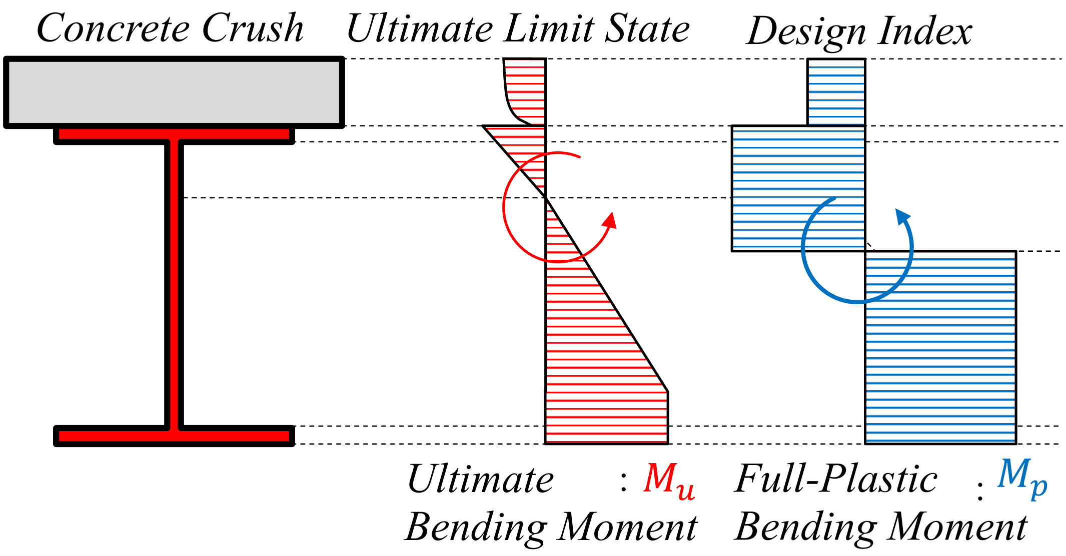


Figure 2.1 *Conceptual picture of calculating bending capacity*

The rectangular stress distribution during full plastic bending moment M_p which is showed in **Figure** 1.2, does not necessarily represent the bending moment, but because it is easy to determine by calculating, it is used as an approximation formula for computing ultimate bending capacity. In this thesis, the mean value of each material parameter is calculated in the calculation of full plastic bending moment. For the composite girders, M_p is calculated to normalize the M_u distribution.

2.2.2 Finite Element Method

FEM is an efficient and commonly used calculation method. It discretizes a continuum into a set of finite elements to solve continuum mechanics problems. In 1956, when the plane element method was successfully applied to the plane stress analysis of plane frame by Turner, Clough et al.

In recent years, the application of finite element method in plastic processing has been developed rapidly. The emergence of elastic finite element, rigid plastic finite element, elastic-plastic finite element, viscoplastic finite element and other related theories and methods provide powerful tools for analyzing various levels of problems in plastic processing, including macro quantity, distribution quantity and micro quantity. However, for complex structures and some nonlinear problems, there are still many studies on the finite element method, which have not yet been standardized and standardized.

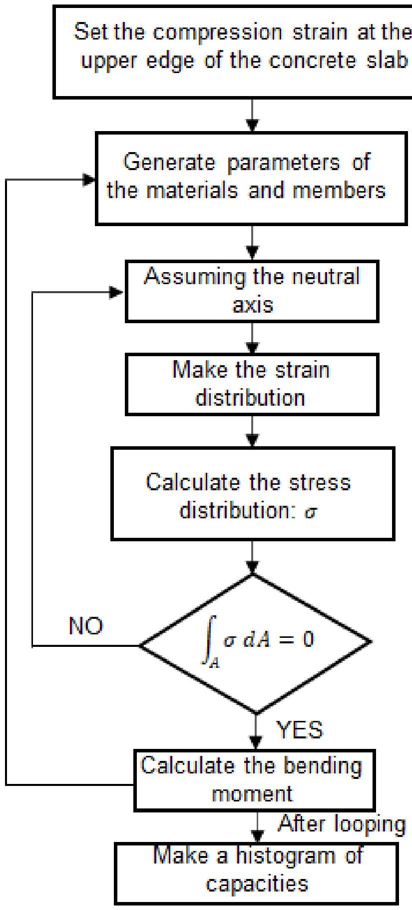


Figure 2.2 *The computing process of Fiber Method*[6)]

For the composite girders, in order to reduce the cost, inevitable, the usage of steel should be cut. This means that the girder will be construct thinner, thus, the structure is more likely to occur buckling. Since the Fiber Method only considers the structural changes in the linear stage, the nonlinear stage of structural instability has not been discussed. Large deformation and buckling analysis of structures need to be discussed by FEM.

Buckling mainly refers to the phenomenon of structural instability under specific loads. It always accompanies with the plastic deformation, and the structure will suddenly jump to another random balance state. Nonlinear buckling analysis and linear (or eigenvalue) buckling analysis are two techniques available in the finite element program for predicting the buckling mode shape of a structure.[9)] And it is said that the linear buckling analysis (by linearized eigenvalue) is generally overestimated. On the contrary, the nonlinear buckling analysis is showed to be more stable and more accurate[10)]. Because of the complexity and uncertainty of constitutive equation, the calculation of FEM part is taken by ANSYS program.

2.3 Monte Carlo Method

Monte Carlo method is a very important numerical calculation method guided by probability and statistics theory. It refers to the use of random numbers (or more common pseudo-random numbers) to solve many computational problems. Theoretically, Monte Carlo method needs a lot of experiments. The more experiments, the more accurate the results are.

2.3.1 Probability Density Function

Using Monte Carlo Method could make the results more reliable, modeling probability density functions. When increasing the number of random variations, the results tend to be more accurate. In order to see more possibilities of the results, three typical probability density functions are used. (Normal Distribution, Log-normal Distribution and Weibull Distribution). And the equations[11)] of these functions are outlined in **Eq**. (2.1), (2.2) and (2.3).

$$f(x) = \frac{1}{\sqrt{2\pi}\sigma} \exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right] \tag{2.1}$$

$$f(x) = \frac{1}{\sqrt{2\pi}\zeta x} \exp\left[-\frac{1}{2}\left(\frac{\ln x-\lambda}{\zeta}\right)^2\right] \tag{2.2}$$

$$f(x) = \frac{1}{\gamma^m} m x^{m-1} \exp\left[-\frac{1}{\gamma^m} x^m\right] \tag{2.3}$$

The parameters λ, ζ, of the Log-normal Distribution and m and η of Weibull Distribution are obtained by **Eq**. (2.4), (2.5), (2.6) and (2.7), respectively.

$$\lambda = \ln\mu - \frac{1}{2}\zeta^2 \tag{2.4}$$

$$\zeta = \sqrt{\ln\left(1 + \frac{\sigma^2}{\mu^2}\right)} \tag{2.5}$$

$$\mu = \eta\Gamma\left(\frac{1}{m} + 1\right) \tag{2.6}$$

$$\sigma = \eta\sqrt{\Gamma\left(\frac{2}{m} + 1\right) - \Gamma^2\left(\frac{1}{m} + 1\right)} \tag{2.7}$$

2.3.2 Discussion about Convergency of Simulation Result

It's required to set random variations of Monte Carlo simulation. Therefore, it is necessary to discuss the convergence relationship between random variations and value. Take the concrete compression strength as an example to set random variations (average:30, standard deviation:1.2), Observe the convergence of the numerical value and determine the number of random variations, which are showed in **Figure** 2.3. (average and standard deviation)

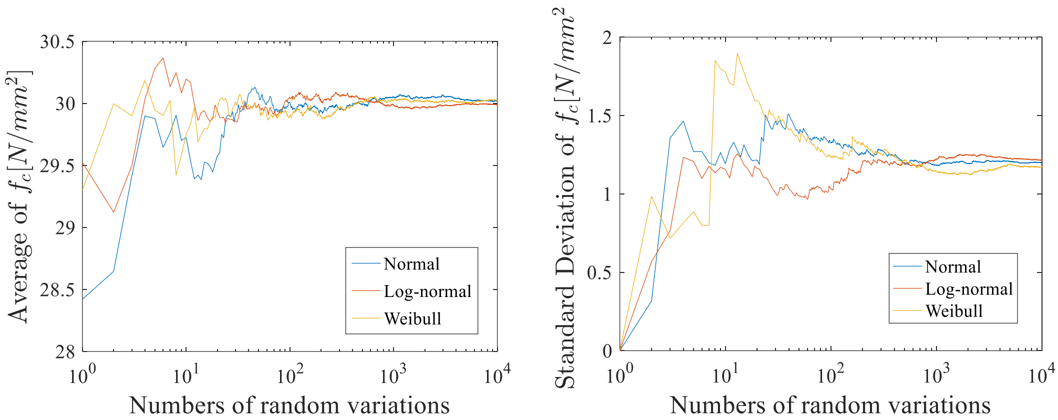


Figure 2.3 *Relation between numbers of random variations and average, standard deviation of* $\boldsymbol{f_c}$

Due to the results showed above, it could be concluded that the average and standard deviation could get convergent when the numbers of random variations reaches about 10,000. It shows that with the increase of random numbers, its value is closer to the design value. And the histograms of the three functions about the example (f_c: cconcrete compression strength) are outlined in **Figure** 2.4, 2.5 and 2.6.

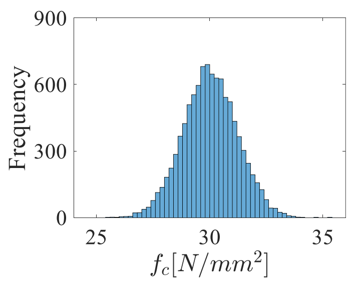


Figure 2.4 *Normal Distribution*

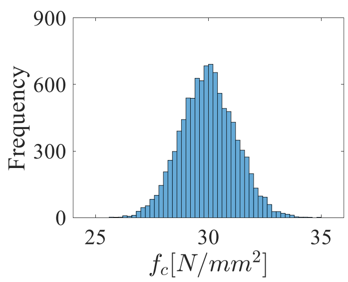


Figure 2.5 *Log-normal Distribution*

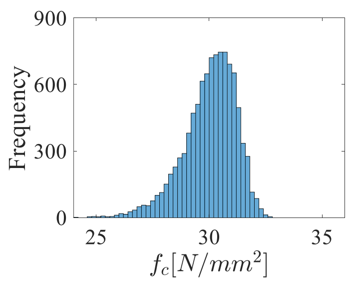


Figure 2.6 *Weibull Distribution*

Chapter 3. NUMERICAL MODEL

In this chapter, we will introduce the numerical model of the simulation, including simulation model of Fiber Method (slender, standard and compact) and the process for modeling FEM model.

3.1 Fiber Method

Based on the previous researches, in order to make the results of numerical simulation more reliable, more cases need to be considered to discuss the influence of various parameters on structural reliability. In order to find a more appropriate model for calculating, the influence of the section dimensions on structural reliability is considered in the section. Three kinds of section dimensions of both structures (NCGs and HCGs), a slender one, a standard one and a compact one (showed in **Figure** 3.1) are selected to modeling simulation models.

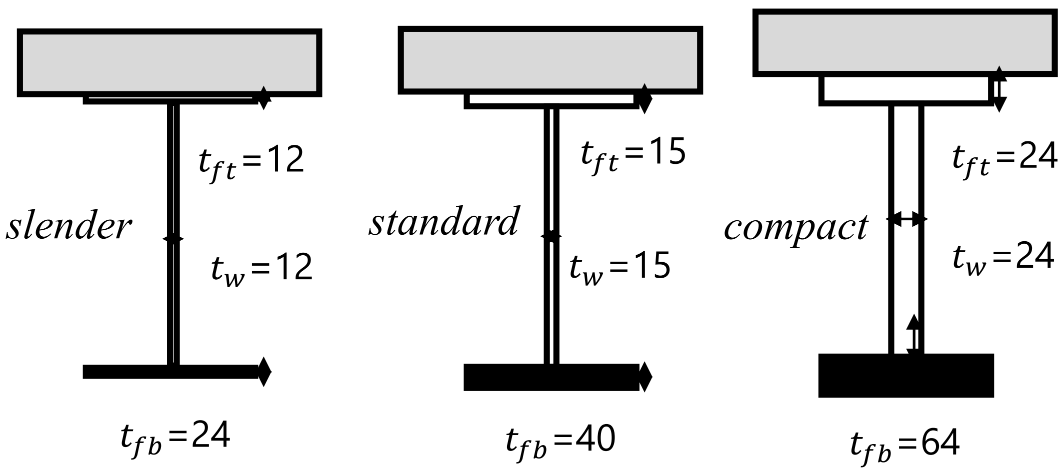


Figure 3.1 *Three models of different section dimensions on composite girder*

Based the three section dimensions showed above for modeling, using Fiber Method to do the statistical calculation. Compare the results to get the structure reliability and a more reasonable structure.

3.2 Finite Element Method

From the reference 10), we could know that linear buckling analysis generally yields unconservative results. Hence, it's necessary to do the nonlinear buckling analysis. A nonlinear buckling analysis is based of linear buckling analysis. With the increase of the load, the structure occurs the nonlinear deformation and get its ultimate state.

3.2.1 Defined Element Type

For FEM modeling, there are two kinds of element types adopted for define structure members. The concrete slab and composite girder are modeled separately.

For the concrete slab, solid element is accepted for modeling. In ANSYS, element SOILD65 is a common element type used to modeling concrete. This element is used for 3D modeling of solids with or without rebar. The solid is capable of cracking in tension and crushing in compression. **Figure** 3.2 shows the geometry of SOLID65.

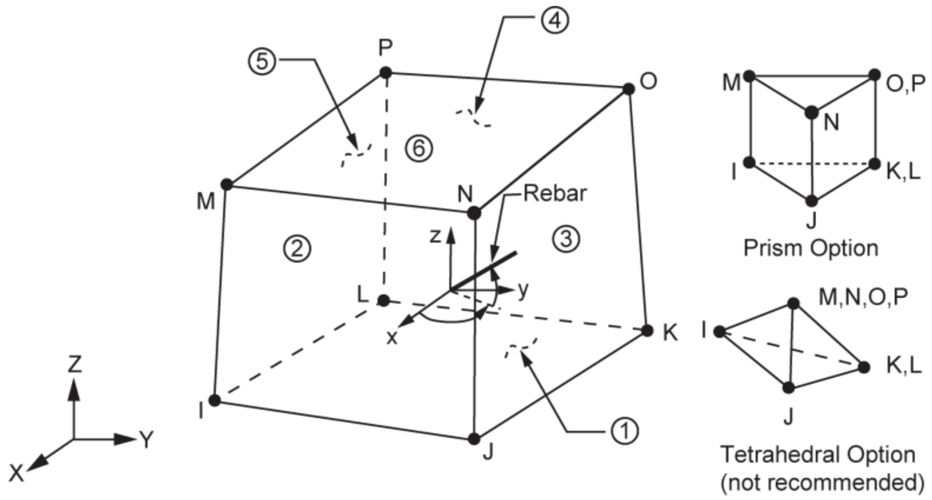


Figure 3.2 *Element SOILD65 geometry (ANSYS, 2012)*[12)]

On the other hand, for the steel members, shell element is considered to modeling the steel girder. In ANSYS, element SHELL181 is a common element type used to modeling thin members. Because SHELL181 is first order shear deformation shell and has finite strain capability. Hence, it is considered as the model for bending and shearing calculation. Furthermore, SHELL181 can have all applicable nonlinear materials, with both membrane stresses, enhanced strain effects, and transverse shear stresses, accounted for. And SHELL181 has a more advanced warping correction algorithm, making the element stiffness less sensitive to warping, so it is regarded as an appropriate element type for steel material. **Figure** 3.3 shows the geometry of SHELL181.

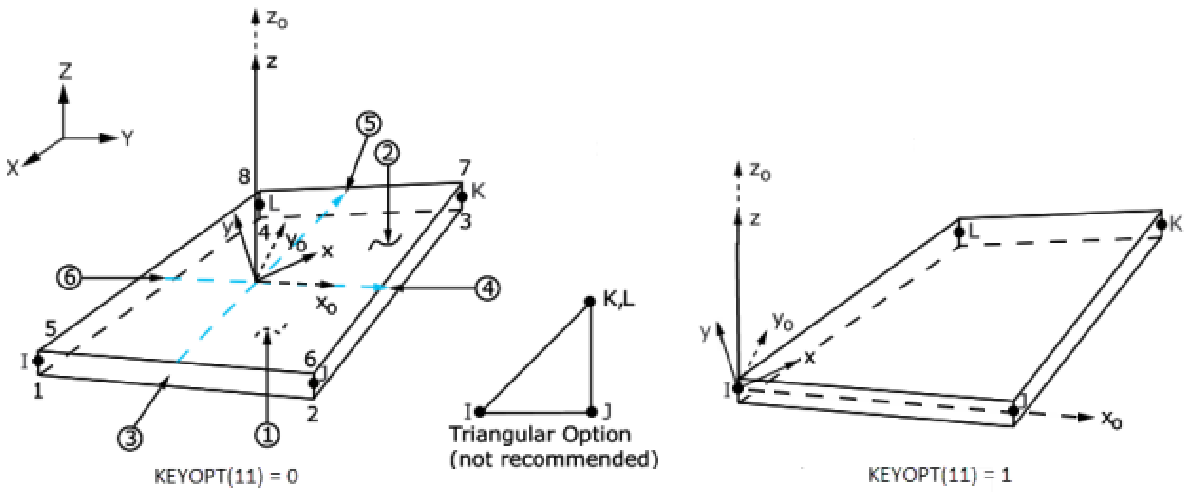


Figure 3.3 *Element SHELL181 geometry (ANSYS, 2012)* [12)]

3.2.2 Modeling FEM Model

At first, we modeling a one-span girder as FEM model. The total length of the structure is set as 30 meters. Using the solid elements to modeling the concrete slab, and the shell elements are applied to the composite girder modeling. The total structure is set up with more than 20,000 elements, set constraints on the two edge of bottom flange to avoid lateral displacement (One end constrained omnidirectional displacement and The other end constrains longitudinal and lateral displacements), then set the vertical distributed load at one third and two third of the total length, which is easy for structure deformation. The concrete slab and the composite girder are connected in the fixed way. And the conceptual model is showed in the **Figure** 3.4.

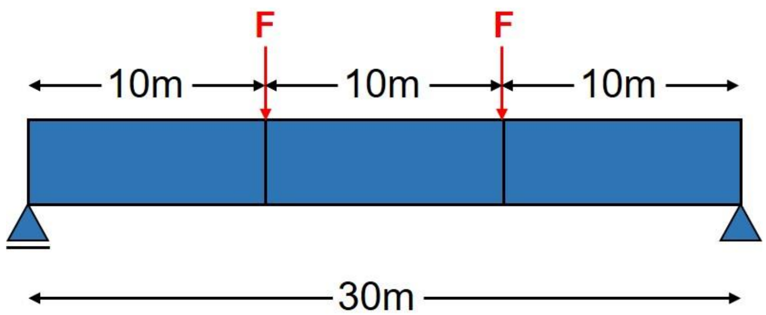


Figure 3.4 *Conceptual model (One-Girder model)*

3.2.3 Verification of ANSYS Program

In order to ensure the accuracy of ANSYS calculation, it is necessary to do the verification about ANSYS program. Because there is no definite theoretical solution in the nonlinear stage, we concentrate on the linear stage of verification.

- Verification: Bending of a composite beam

A beam of length l and width w, made up of two layers of different materials, is subjected to a uniform rise in temperature from T_{ref} to T_o and a bending moment M_y at the free-end. Determine the free-end displacement δ (in the Z-direction) and the X-direction stresses at the top and bottom surfaces of the layered beam. E_i and α_i correspond to the Young's modulus and thermal coefficient of expansion for layer i, respectively. And the conceptual model is showed in **Figure** 3.5, the model properties are showed in **Table** 3.1.

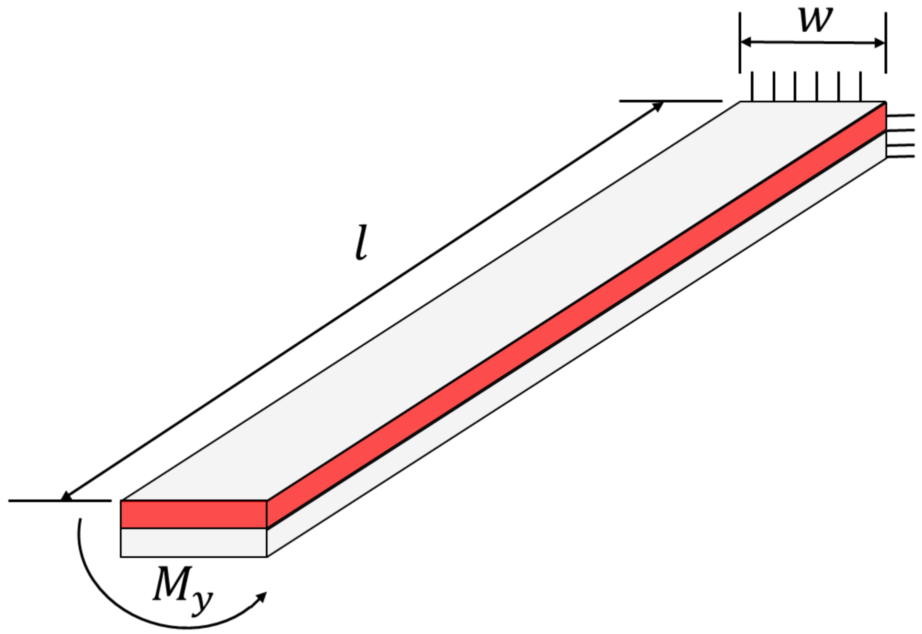


Figure 3.5 *Conceptual model (Composite beam)*[13)]

Table 3.1 *Model properties of Composite beam*

Material Properties	Geometric Properties	Loading

Material Ⅰ (red): $E_1 = 1.2 \times 10^6 psi$ $\alpha_1 = 1.8 \times 10^{-4} in/in/\,^oF$ Material Ⅱ (grey): $E_2 = 0.4 \times 10^6 psi$ $\alpha_2 = 0.6 \times 10^{-4} in/in/\,^oF$	$l = 8in$ $w = 0.5in$ $h_1(red) = 0.2in$ $h_2(grey) = 0.1in$	$T_o = 100\ ^oF$ $T_{ref} = 0\ ^oF$ $M_y = 10in\text{-}lb$

It refers to the reference 11, using SOLID185 to modeling the composite beam, the displacement of Z direction is calculated as $0.832mm$.[13)]

Then, the simulation result by ANSYS Program is showed in **Figure** 3.6. And the comparison and error analysis is stated as following.

Figure 3.6 *Simulation result of displacement about ANSYS about Composite beam (10 times scale)*

It could be observed that the simulation result of maximum displacement is about $0.83181mm$. Thus, the error between the theoretical solution and numerical solution 0.0228%.

Therefore, it can be approximately considered that the results obtained by ANSYS program are reliable due to the verification result.

Chapter 4. NUMERICAL EXPERIMENT

4.1 Introduction

In order to quantitatively calculate the structural reliability and make the results more reliable. Numerical experiment is adopted to do the simulation. To increase the degree of data dispersion, we set variations of section members and material properties by their average and standard deviation.

The structure of this chapter is showed as following. Section.2 gives the values (design value, average and standard deviation) of each parameter. Introduce the relation between stress and strain of materials (concrete and steel). Section.3 introduces the definition of reliability index and failure probability and gives the equation for calculating the reliability index.

4.2 Variables on Computing Model

4.2.1 Variables on Section Members

For the computing model, there are three kinds of section dimensions (slender model, standard model and compact model) and the parameters of section members about each model should be determined.

When determining the section members, it is assumed that the central vertical axis always exists on the steel web to make the concrete bear the compressive force in the ultimate state. In the numerical experiment, there are three kinds of structural section dimensions, and the parameters of section members[14)] are outlined in **Table** 4.1. The data are slender (standard, compact) in turn.

Table 4.1 *Parameters of section members (slender (standard compact))*

Parameters	Average μ(mm)	Standard Variation σ(mm)	Variation Coefficient CV
Slab Width：w_c	1500	6	0.0040
Slab Thickness：t_c	160	6	0.0375
Upper Flange Width：w_{ft}	300	4.38	0.0146
Upper Flange Thickness：t_{ft}	12 (15, 24)	0.175 (0.219, 0.350)	0.0146
Web Thickness：t_w	12 (15, 24)	0.175	0.0146

Web Height：d_w	1500	21.9	0.0146
Bottom Flange Width：w_{fb}	300	4.38	0.0146
Bottom Flange Thickness：t_{fb}	24 (40, 64)	0.350 (0.584, 0.934)	0.0146

Using the parameters outlined above to modeling three different models for evaluating the structural reliability.

4.2.2 Variables on Material Properties

To do the simulation, the equations of strain-stress about materials[15)] should be acknowledged. The strain-stress curve of concrete is showed in the **Figure** 4.1 and the equations are outlined in **Eq.** (4.1).

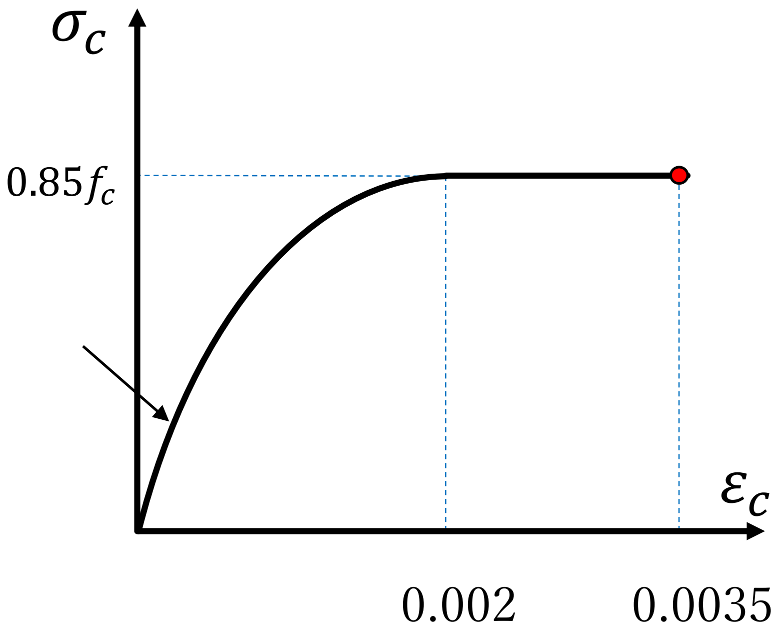


Figure 4.1 *The strain-stress curve of concrete*

$$\sigma_c = 0.85f_c\left(\frac{\varepsilon_c}{0.002}\right)\left(2-\frac{\varepsilon_c}{0.002}\right) \quad , \quad \varepsilon_c \le 0.002$$

$$\sigma_c = 0.85f_c \quad , \quad 0.002 \le \varepsilon_c \le 0.0035 \tag{4.1}$$

where ε_c is the strain of concrete, σ_c is the stress of concrete and f_c is compression strength of concrete.

Additionally, the strain-stress curve of normal steel and high-performance steel is showed in the **Figure** 4.2 and the equations are outlined in **Eq**. (4.2).

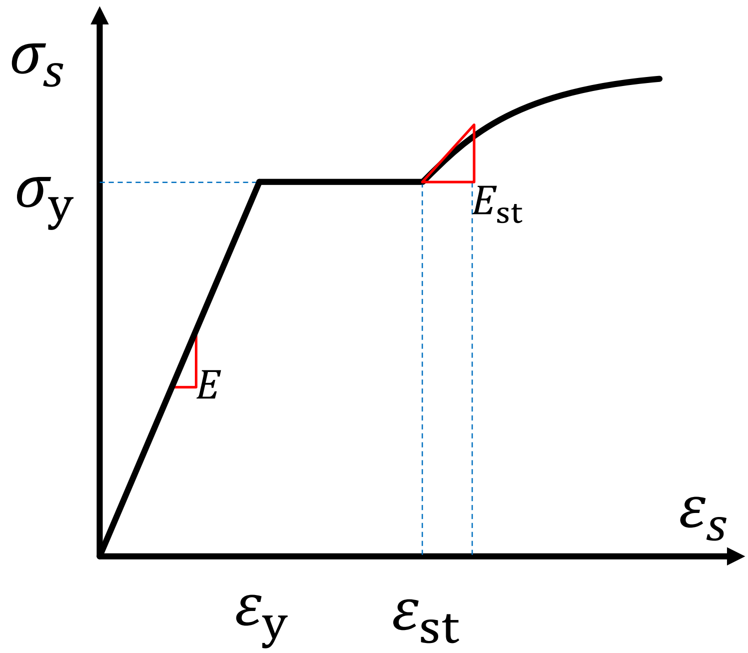


Figure 4.2 *The strain-stress curve of steel*

$$
\begin{aligned}
&\sigma_s = E\varepsilon_s \quad , \quad \varepsilon_s \leq \varepsilon_y \\
&\sigma_s = \sigma_y \quad , \quad \varepsilon_y \leq \varepsilon_s \leq \varepsilon_{st} \\
&\frac{\sigma_s}{\sigma_y} = \frac{1}{\xi}\frac{E_{st}}{E}\left[1 - exp\left\{-\xi\left(\frac{\varepsilon_s}{\varepsilon_y} - \frac{\varepsilon_{st}}{\varepsilon_y}\right)\right\}\right] + 1, \quad \varepsilon_s \geq \varepsilon_{st}
\end{aligned}
\tag{4.2}
$$

where ε_s is the strain of steel, σ_s is the stress of steel, E is the young's modulus of steel, ε_y is the yield strain of steel, σ_y is the yield stress of steel, ε_{st} is the hardening strain of steel, E_{st} is the hardening coefficient of steel and ξ is the hardening curvature of steel.

The statistic parameters of each material are acknowledged from the reference 6). The parameters of concrete are outlined in **Table** 4.2, and the parameters of steel are outlined in **Table** 4.3.

Table 4.2 *Parameters of concrete*

Parameters	Average μ	Standard Variation σ	Variation Coefficient CV
Compression Strength f_c (N/mm^2)	30	1.2	0.04
Crushing Strain ε_c	0.0035		

Table 4.3 *Parameters of steel*

	steel			high-performance steel		
	Average μ	Standard Variation σ	Variation Coefficient CV	Average μ	Standard Variation σ	Variation Coefficient CV
Young's Modulus E (*N/mm^2*)	200000	2000	0.01	200000	2000	0.01
Yield Strength σ_y (*N/mm^2*)	293.75	23.5	0.08	549	36	0.0656
hardening strain ε_{st}	0.0185	0.0049	0.265	0.0082	0.0041	0.5
Hardening Coefficient E_{st} (*N/mm^2*)	4156	1342	0.323	2000	1170	0.585
Hardening Curvature ξ (*N/mm^2*)	0.049	0.027	0.55	0.02	0.025	1.25

4.3 Reliability Index and Failure Probability

When it comes to the structural reliability, it always refers to the reliability index β. The reliability index is divided by the standard deviation to the average value for the designed load. And the conceptual picture is showed in **Figure** 4.3 while the equation of the reliability index is outlined in **Eq**. (4.3).

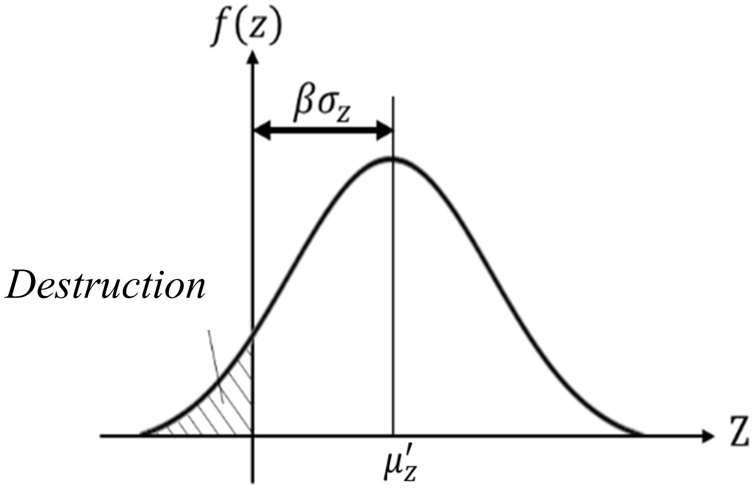


Figure 4.3 *Conceptual picture of reliability index*

$$\beta = \frac{\mu_z - \alpha}{\sigma_z} = \frac{\mu_z'}{\sigma_z} \tag{4.3}$$

Where, μ_z and σ_z are the average and standard deviation of bending capacity, α is the factor of safety. It is said that α ranges from 0.59-1.00[16)].

In this part, the probability distribution of bending capacity is calculated by assuming the normal distribution of all the strength distribution. This means that the probability density functions of material and member parameters are not considered.

Failure probability is the probability of failure in the sample size, which is assumed to occur below a certain bending capacity. Reliability index and failure probability are the criteria for evaluating structural reliability. The structure with high reliability index and low failure probability is considered as high reliability. And a low reliability and a high failure probability indicates a low reliability.

Chapter 5. RESULTS ABOUT SIMULATION

5.1 Introduction

In this chapter, it provides histograms and other results of the hybrid composite girders and normal composite girders. Statistical methods were used to evaluate the reliability of the structures. After statistical calculation, we could get the histograms of structural bending capacity. With those results, structural reliability index and failure probability could be calculated.

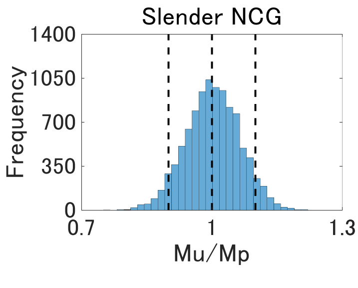
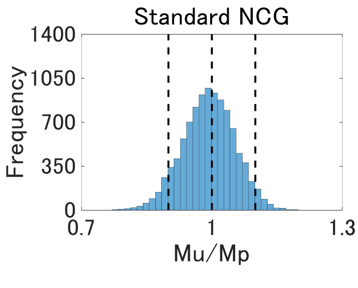
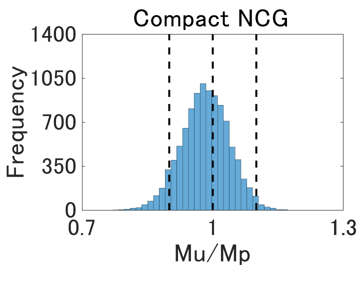
The structure of this chapter is showed as following. Section.2 gives the results of two models (HCGS and NCGs) in three section dimensions (slender, standard and compact) and then discuss and analyze the results showed above. In section.3, the problem and solution of FEM are showed to modeling the FEM model and then the results and discussions of FEM are outlined following.

5.2 Fiber Method

5.2.1 Statistical Calculation

In the calculation part of Fiber Method, 10000 random variations were generated for statistical calculation. And the histograms about bending capacity of each case are generated for comparison, the histograms of Normal Distribution are outlined in **Table** 5.1. Those of Log-normal Distribution are outlined in **Table** 5.2. And those of Weibull Distribution are outlined in **Table** 5.3. All of the histograms are in the same horizontal axis and vertical axis for directly observation. It could be observed that the histograms of HCGs are narrower than those of NCGs, which indicate less discreteness of data.

Table 5.1 *Histograms of bending capacity (Normal Distribution)*

M_u/M_p	Slender	Standard	Compact
NCGs			

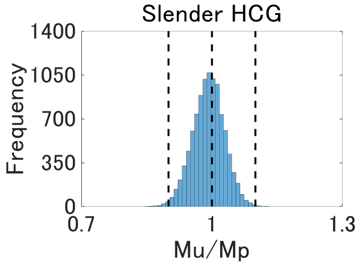
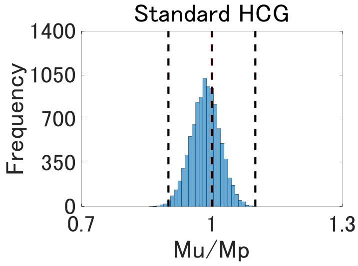
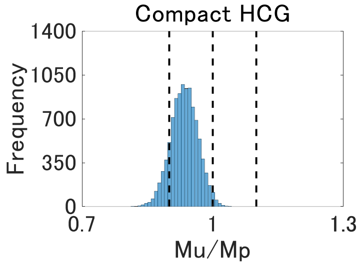
<table>
<tr><td>HCGs</td><td>
</td><td>
</td><td>
</td></tr>
</table>

Table 5.2 *Histograms of bending capacity (Log-normal Distribution)*

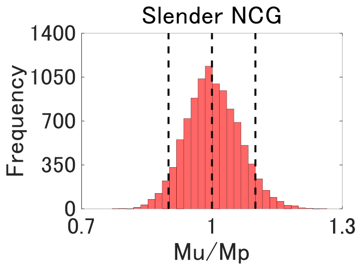
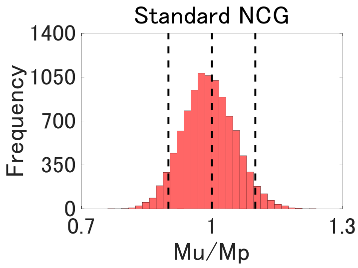
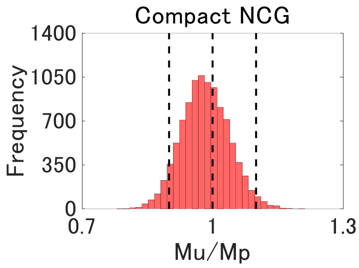
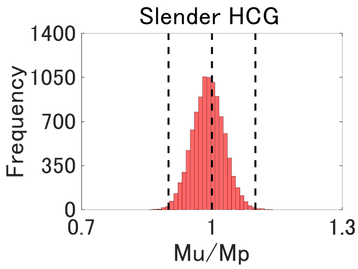
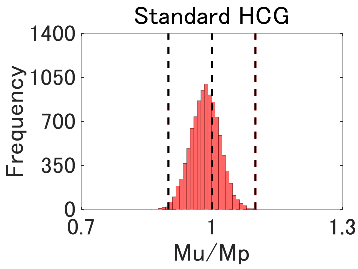
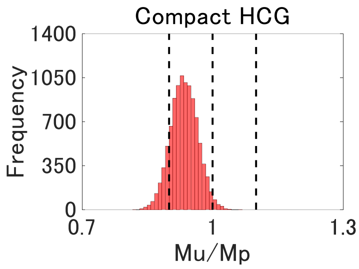
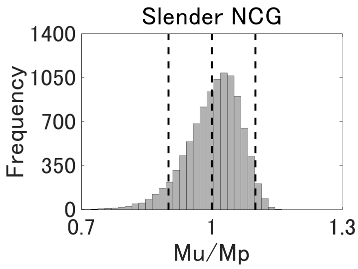
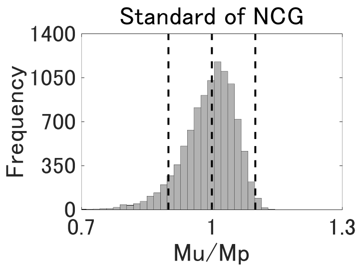
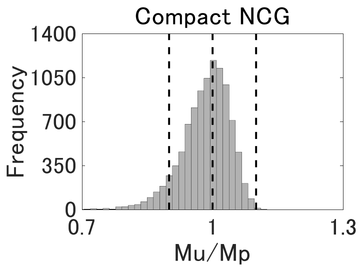
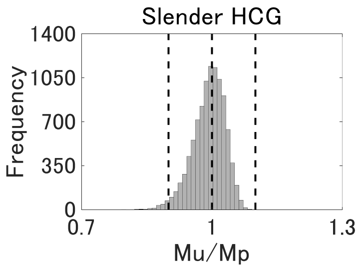
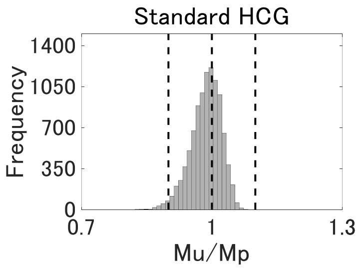
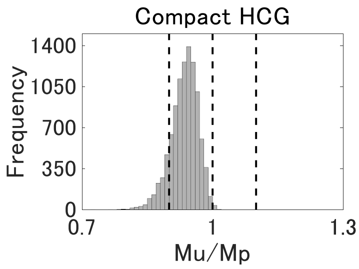
<table>
<tr><th>M_u/M_p</th><th>Slender</th><th>Standard</th><th>Compact</th></tr>
<tr><td>NCGs</td><td>
</td><td>
</td><td>
</td></tr>
<tr><td>HCGs</td><td>
</td><td>
</td><td>
</td></tr>
</table>

Table 5.3 *Histograms of bending capacity (Weibull Distribution)*

<table>
<tr><th>M_u/M_p</th><th>Slender</th><th>Standard</th><th>Compact</th></tr>
<tr><td>NCGs</td><td>
</td><td>
</td><td>
</td></tr>
<tr><td>HCGs</td><td>
</td><td>
</td><td>
</td></tr>
</table>

After statistical calculation, the average, standard deviation, kurtosis and skewness are obtained for analysis and discussion to evaluate the reliability of each case. The statistics result of Normal distribution is showed in the **Table** 5.4, the statistics result of Log-normal distribution is showed in the **Table** 5.5 and the statistics result of Weibull distribution is showed in the **Table** 5.6.

Table 5.4 *Statistics result of Normal Distribution*

	Slender		Standard		Compact	
	NCG	HCG	NCG	HCG	NCG	HCG
Average	1.0021	0.9921	0.9925	0.9853	0.9830	0.9345
Standard deviation	0.0617	0.0360	0.0592	0.0334	0.0541	0.0309
Kurtosis	-0.0573	0.0162	-0.0356	-0.0155	-0.0235	-0.0433
Skewness	-0.0588	-0.0644	-0.1152	-0.0711	-0.0950	-0.0605

Table 5.5 *Statistics result of Log-normal Distribution*

	Slender		Standard		Compact	
	NCG	HCG	NCG	HCG	NCG	HCG
Average	1.0020	0.9918	0.9925	0.9849	0.9830	0.9345
Standard deviation	0.0613	0.0360	0.0592	0.0332	0.0541	0.0310
Kurtosis	0.0287	0.0149	-0.0362	-0.0196	-0.0282	-0.0409
Skewness	0.1708	0.0503	0.0106	-0.0119	0.1211	0.0194

Table 5.6 *Statistics result of Weibull Distribution*

	Slender		Standard		Compact	
	NCG	HCG	NCG	HCG	NCG	HCG
Average	1.0035	0.9927	0.9939	0.9862	0.9842	0.9347
Standard deviation	0.0622	0.0369	0.0602	0.0349	0.0552	0.0314
Kurtosis	0.9493	0.6136	1.2117	0.6255	1.3875	0.9705
Skewness	-0.7785	-0.6468	-0.8731	-0.6314	-0.8914	-0.7074

5.2.2 Statistical Results and Discussions

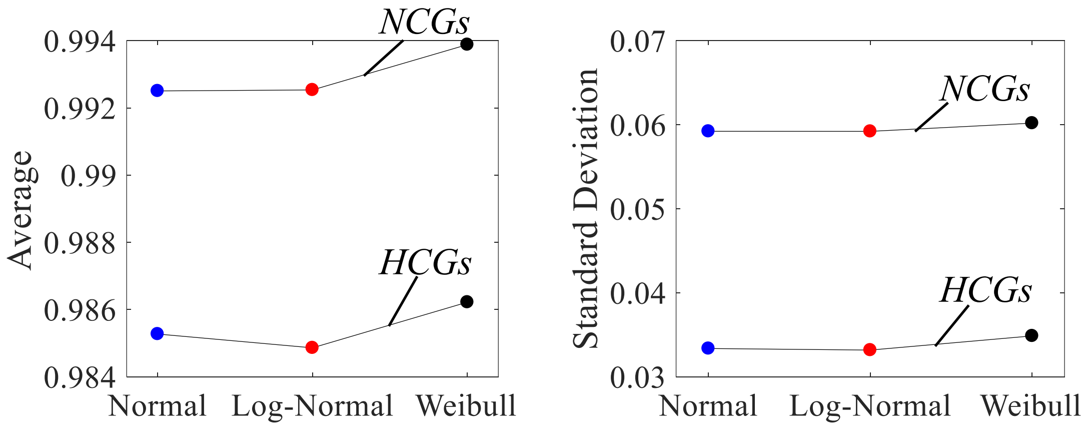


Figure 5.1 *Average and Standard Deviation of NCGs and HCGs (Standard model)*

Take the standard model as an example, the upper lines are NCGs, and the lower lines are HCGs. From the **Figure** 5.1 (left), it could be concluded that with the same section dimensions, the average of bending capacity about NCGs is a little larger than that of HCGs, which means that the NCGs models have a higher bending capacity than those of HCGs models. And from the **Figure** 5.1 (right), it could be concluded that with the same section dimensions, the standard deviation of bending capacity about NCGs is larger than that of HCGs, which means that the HCGs models have fewer disperse samples than those of NCGs models from the calculation by Fiber method.

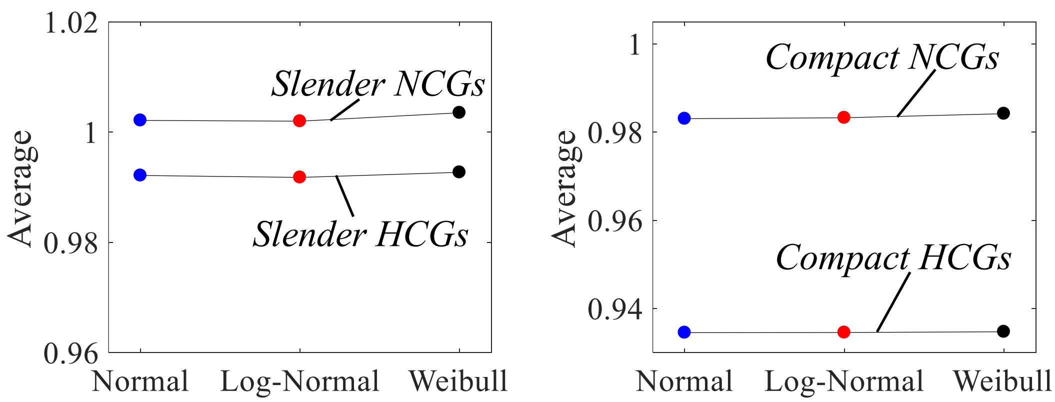


Figure 5.2 *Average of NCGs and HCGs (Slender model and Compact model)*

The comparison charts about average of bending capacity are showed in the **Figure** 5.2. From **Figure** 5.2, it could be concluded that with the same material composing, the average of bending capacity about slender models is a little larger than that of compact models, the larger the section dimensions are, the lower the average of bending capacity is, which means that a thick steel girder could not be made fully use of its bending capacity.

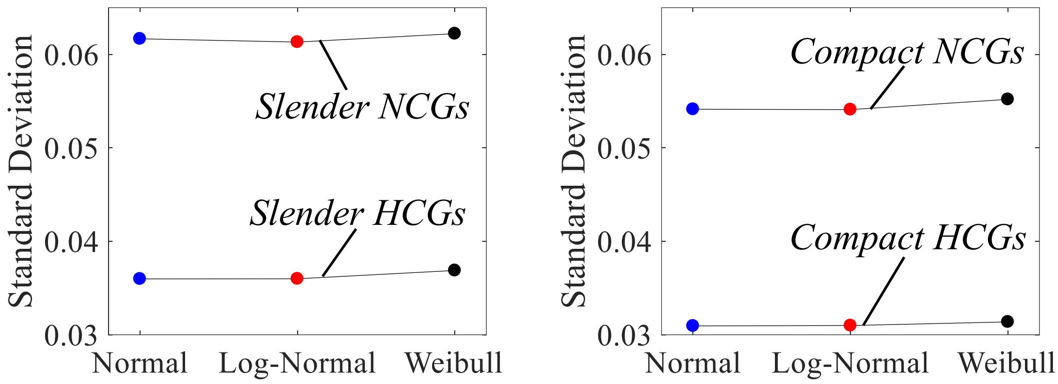


Figure 5.3 *Standard Deviation of NCGs and HCGs (Slender and Compact model)*

The comparison charts about standard deviations of bending capacity are showed in the **Fig** 5.3. From **Fig** 5.3, it could be concluded that with the same material composing, the standard deviation of bending capacity about slender models is a little larger than that of compact models, the larger the section dimensions are, the smaller the standard deviation of bending capacity is, which means that a thick steel girder could provide a higher structural stability and guarantee a higher data concentration.

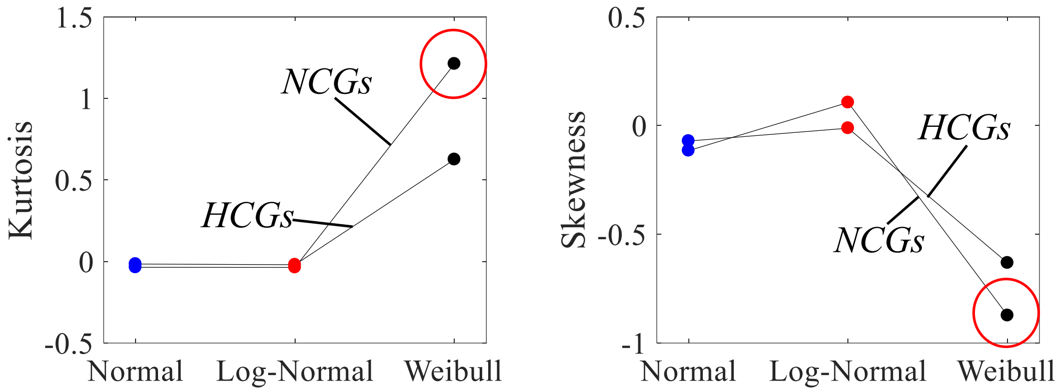


Figure 5.4 *Kurtosis and Skewness of NCGs and HCGs (Standard model)*

Furthermore, kurtosis and skewness are showed in the **Figure** 5.4. Kurtosis is the characteristic number of the peak value of the probability density distribution curve at the average value. Intuitively, kurtosis reflects the sharpness of the peak. Because the kurtosis of a normal distribution is equal to 3, 3 is usually subtracted from the kurtosis calculation. High kurtosis means that the increase of variance is caused by the extreme values of lower frequency greater than or less than the average value. Hence, due to the **Figure** 5.4 (left), it could be concluded that Weibull Distribution has more extreme values than other two kinds of functions, which indicates more abnormal samples.

Skewness represents the degree of asymmetry of probability distribution density curve with respect to the average value. Intuitively, it is the relative length of the tail of the density function curve. When the distribution is symmetrical, the skewness is equal to 0. When the skewness is greater than 0, the heavy tail is on the right, the distribution is right skewed. When the skewness is less than 0, the heavy tail is on the left, the distribution is skewed to the left. Hence, due to the **Figure** 5.4 (right), it could be concluded that Weibull Distribution is skewed to the left, which indicates there are more values are less than the average.

This shows that the Weibull Distribution contains more dangerous samples, the design using Weibull Distribution will make the design safer.

5.2.3 Results and Discussions about Structural Reliability

After the statistical calculation, we could get the reliability index and failure probability of each model. Refer to the Chapter 4, according to the reference 16), in order to make the sample larger, there fixed $\alpha = 0.9$, then the reliability index and failure probability of each case could be calculated.

These data are compared in the form of scatter plotted on the same graphs which are outlined in **Figure** 5.5 and **Figure** 5.6. **Figure** 5.5 is about the comparison of same section dimensions and **Figure** 5.6 is about the comparison of same material properties.

From the **Figure** 5.5, take the compact section dimension as an example, it could be observed that with same section dimensions, HCGs have lower reliability than the NCGs. Because concrete will arrive its ultimate state faster with a larger load, such structure cannot make full use of the bending capacity of steel. On the other hand, the slender model and standard model of HCGs have higher reliability than such structures of NCGs. Hence, if the structure section dimensions are selected as compact one, use NCGs are safer. On the contrary, if the structure section dimensions are selected as slender or standard one, use HCGs are showed to be safer.

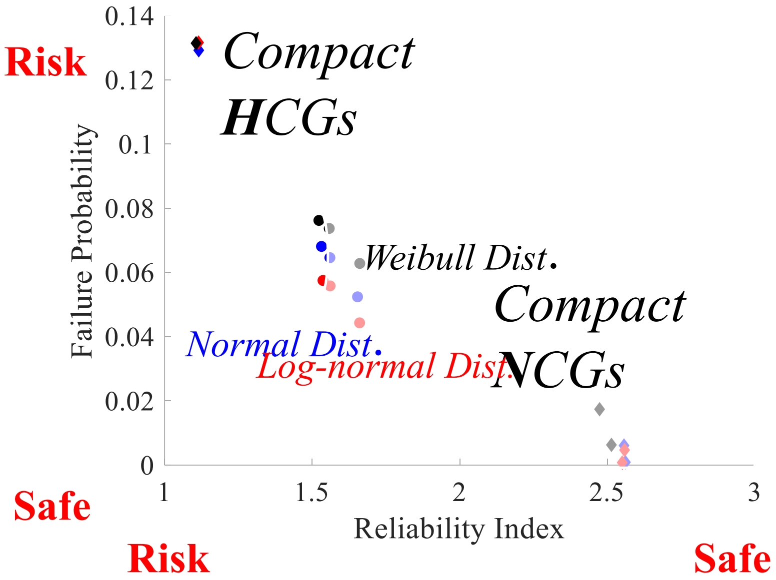


Figure 5.5 *Reliability index and Failure probability (NCGs and HCGs)*

Besides, the failure probability of Weibull Distribution shows to be highest among the three functions, which means that Weibull Distribution has more failure samples and using Weibull Distribution for designing tends to be safest.

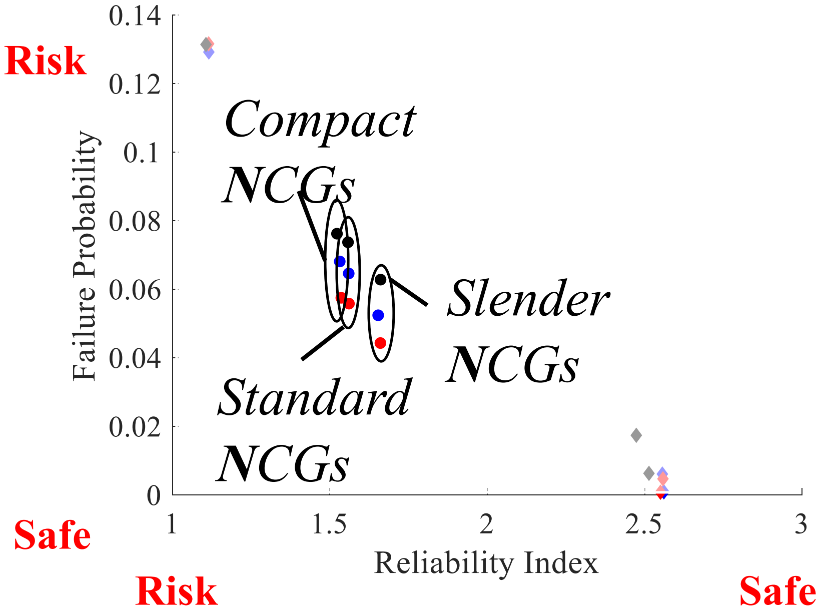


Figure 5.6 *Reliability index and Failure probability (Slender, Standard and Compact)*

From the **Figure** 5.6, it could be observed that with same material properties, the slender model has a higher reliability index and lower failure probability than the standard one with the same probability distribution, also the standard model has a higher reliability index and lower failure than the compact one with the same probability distribution. Hence, it could be concluded that the slender model has the highest reliability among the three section dimensions with same probability distribution.

And the histograms of destruction numbers are outlined in the **Figure** 5.7, 5.8 and 5.9. Using the same horizontal axis (α: factor of safety) and vertical axis to compare each model directly.

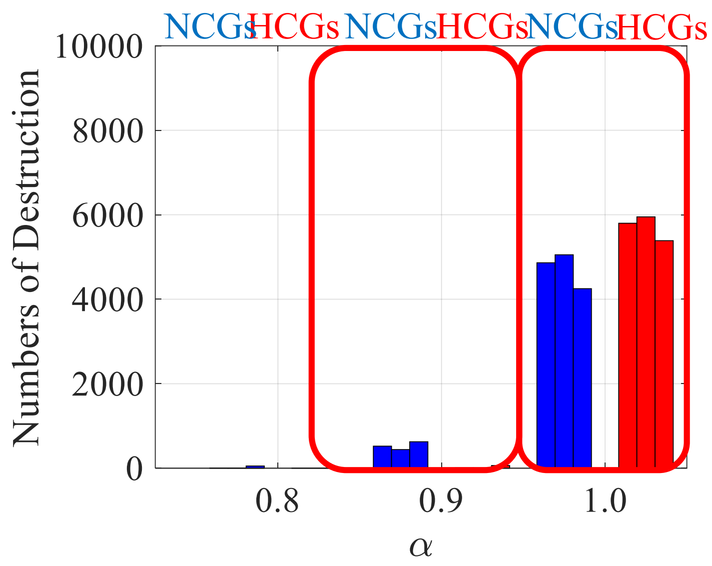


Figure 5.7 *Histograms of destruction numbers (Slender model)*

From the **Figure** 5.7 (slender model), when α is equal to 0.9, for the HCGs, there are nearly no destructions which is much fewer than that of NCGs. And when α is equal to 1.0, the failure number of HCGs is larger than that of NCGs. This means that when the design load is small $(\alpha = 0.9)$, the HCGs have higher reliability with slender section dimension. And when the design load is large $(\alpha = 1.0)$, the NCGs have higher reliability with slender section dimension.

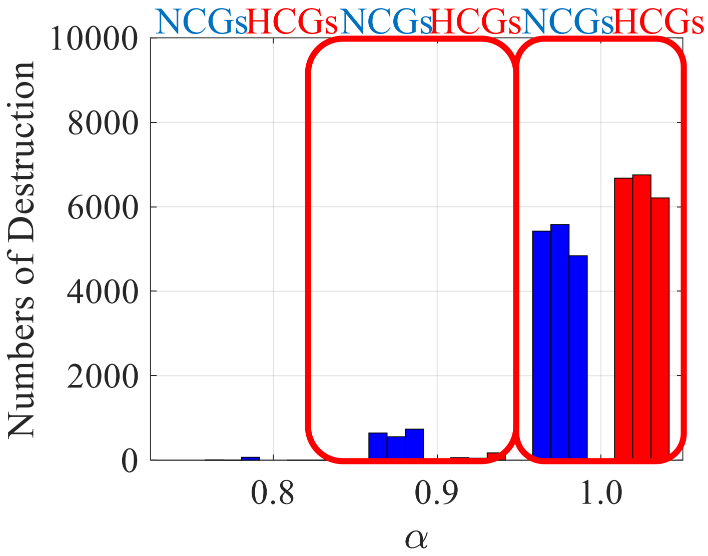


Figure 5.8 *Histograms of destruction numbers (Standard model)*

From the **Figure** 5.8 (standard model), when α is equal to 0.9, the failure number of HCGs is also smaller than that of NCGs. And when α is equal to 1.0, the failure number of HCGs is larger than that of NCGs. This means that when the design load is small $(\alpha = 0.9)$, the HCGs have higher reliability with standard section dimension. And when the design load is large $(\alpha = 1.0)$, the NCGs have higher reliability with slender section dimension.

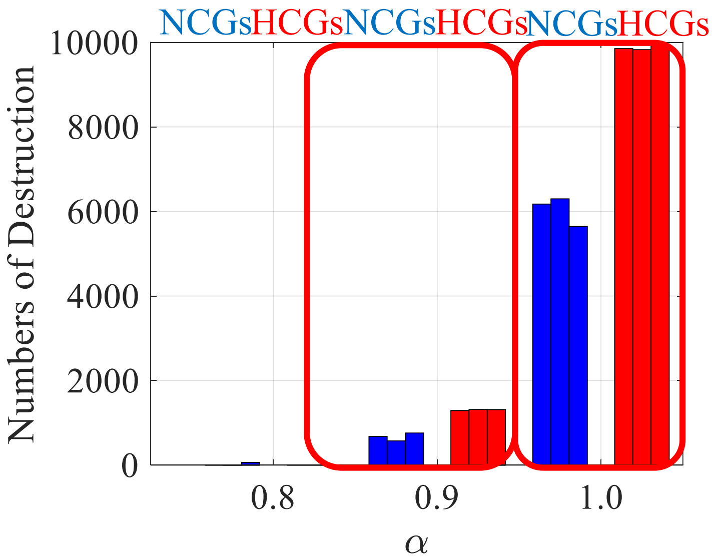


Figure 5.9 *Histograms of destruction numbers (Compact model)*

From the **Figure** 5.9 (compact model), when α is equal to 0.9 and 1.0, the failure number of HCGs is larger than that of NCGs. This means that for the compact model, using NCGs is showed to have fewer failure samples and could be safer.

It could be concluded that with the same section dimension, when $\alpha = 0.9$, which indicated a smaller design load, it's better to design the composite girder as slender and standard for HCGs, compact for NCGs. And when $\alpha = 1.0$, which indicated a larger design load, it's better to design all three kinds of the section dimensions as NCGs.

5.3 Finite Element Method

5.3.1 From One-Girder Model to Two-Girder Model

During the calculation of Fiber Method. one-girder model is adopted as the simulation model. However, in practical engineering applications, one-girder bridges are usually not used. Thus, in order to meet the actual needs of the engineering practice, a two-girder model is accepted for the FEM modeling, the part of steel girder is showed as **Figure** 5.10. Set stiffeners at the web of I-beam every 5 meters to restrain the deformation of buckling, and set cross girder connecting the two span of girders to restrain the lateral buckling under the large load.

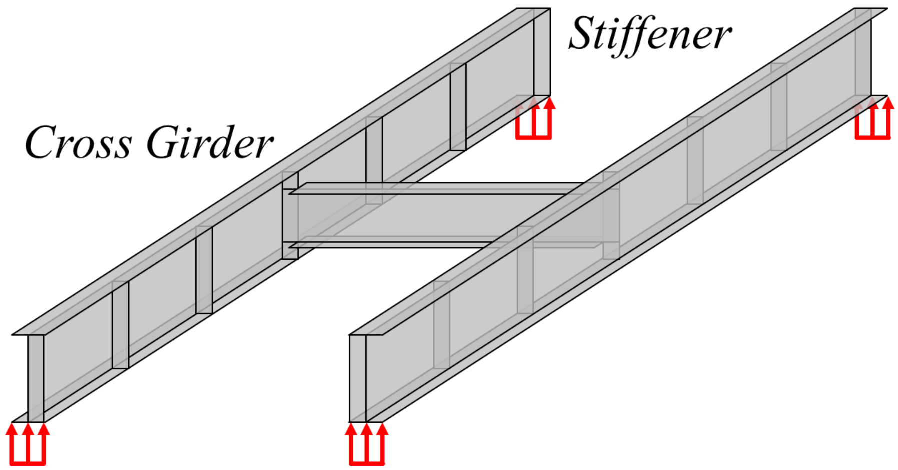


Figure 5.10 *Supplement to One-Girder model (Two-Girder model)*

And the overall conceptual model is showed in the **Figure** 5.11. The distributed loads are set at one third and two third of the top of the concrete slab.

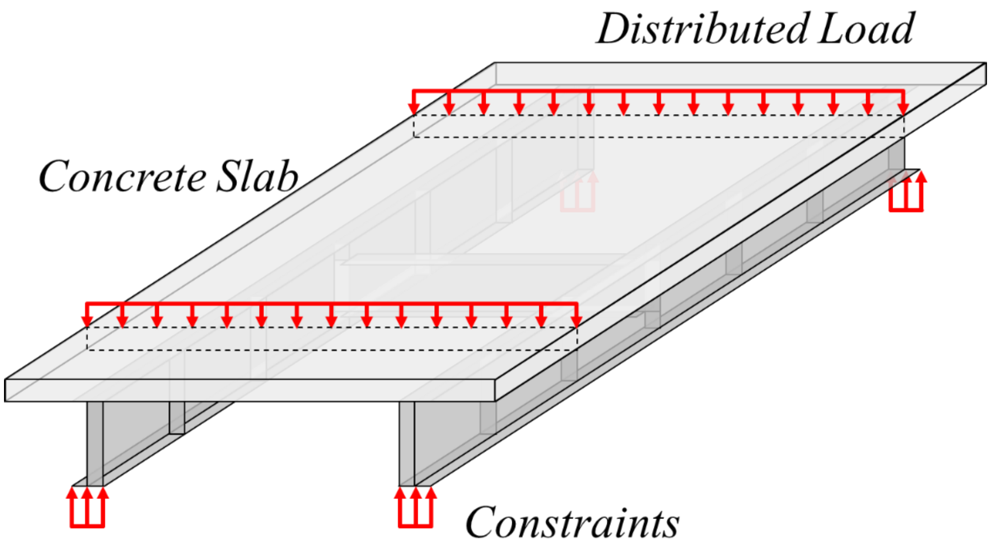


Figure 5.11 *Overall conceptual model (Two-Girder model)*

5.3.2 Statistical Calculation and Results

Refer to the chapter 3.2.1, we modeling the concrete slab with SOLID 65 and the steel girder with SHELL 181.

Based on the Von Mises yield criterion and associate with its' liquidity criteria. Using Multilinear isotropic strengthen criteria to generate material properties. Displacement control is adopted as the load for the calculation and plenty of loading increments are given for iterative calculation. The ultimate load is obtained eventually. The nonlinear calculation follows the criterion of Riks method, the calculation is terminated when the reaction force reaches the peak and the last state is regarded as the limit state of the structure. The characteristic buckling state is used as the initial structural imperfection.

After calculation, the ultimate bending moment M_u should be recorded for statistic calculation. Therefore, it is necessary to determine the limit state of the structure. The ultimate state determined by Fiber method is the destruction of the upper edge of the concrete slab. Therefore, it is necessary to determine whether the upper edge of the concrete slab in the ultimate state obtained by the Riks method is damaged.

The total displacement of standard HCG model due to the simulation result is showed in **Figure** 5.12.

It could be observed that the structure occurs large deformation in the last step. Since the steel girder in the lower part of the compact model has greater strength among these models, the upper part of the concrete slab is more likely to be damaged under the same bearing capacity. Therefore, the compact HCGs model is adopted to value the strain of concrete slab. And the Von Mises total mechanical strain of compact HCGs model is showed in **Figure** 5.13.

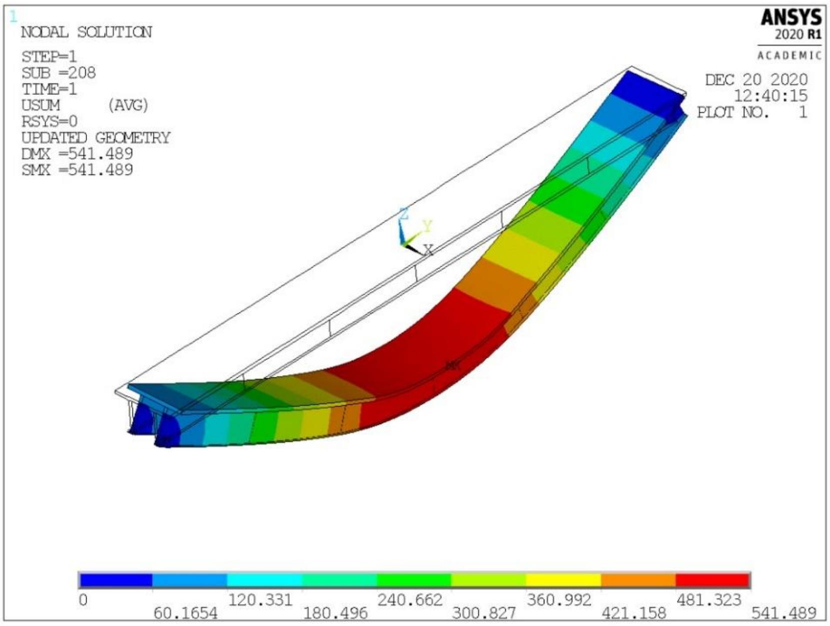


Figure 5.12 *Total displacement of Standard HCGs model (10 times scale)*

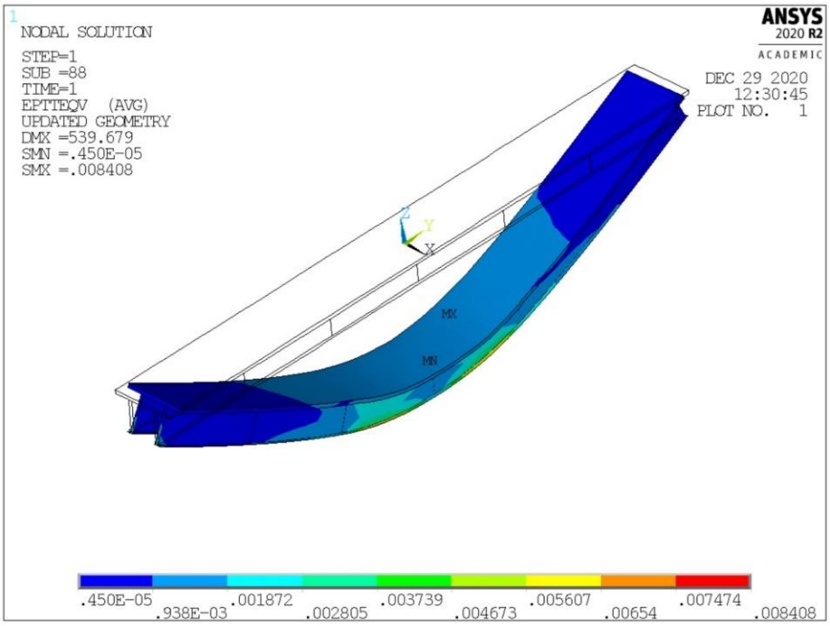


Figure 5.13 *Total mechanical strain of Compact HCGs model (10 times scale)*

According to the **Figure** 5.13, due to the Von Mises total mechanical strain chromatogram, it could be observed that the maximum total mechanical strain of the compact HCGs model is posited at the bottom flange of the steel girder. And the total mechanical strain of the upper edge of concrete slab could be observed to be less than the crushing strain ε_c=0.035.

Thus, buckling limit state is considered as structural limit state. It could be determined that the structural ultimate state is the limit state calculated by the Riks method. The ultimate bending moment should be calculated by the result of last set about the FEM simulation. The graphical illustration of Riks method is showed in the **Figure** 5.14. And the verification of the relation of displacement and reaction force (standard HCGs model) is showed in the **Figure** 5.15.

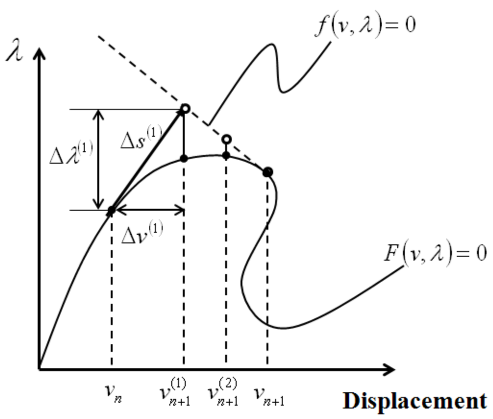


Figure 5.14 *Graphical illustration of Riks method*[17)]

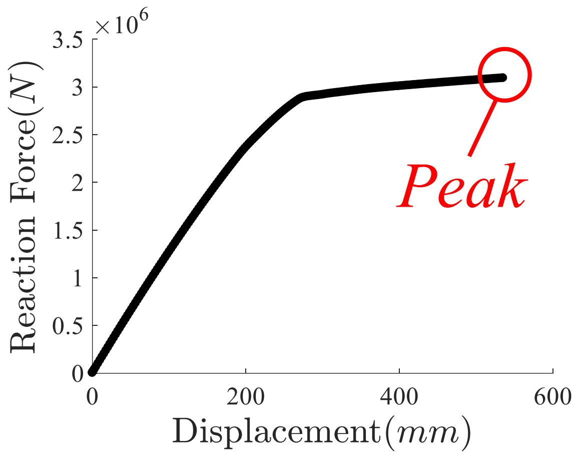
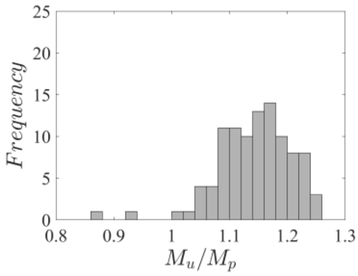
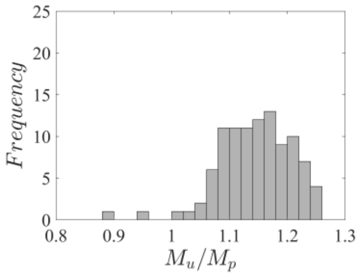
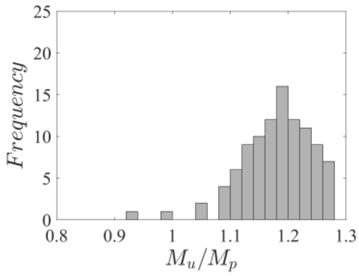
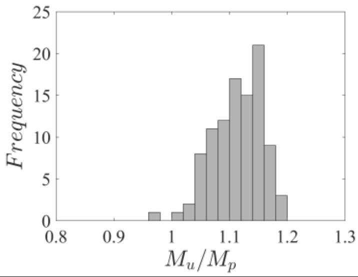
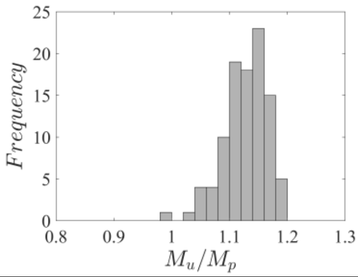
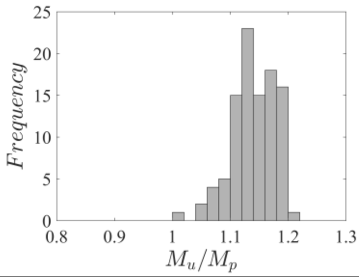


Figure 5.15 *Relation of displacement and reaction force (Standard HCGs model)*

From the **Fig** 5.15, It could be observed that the curve of the displacement and reaction force reach its peak value and its shape is similar to the first half of the **Figure** 5.14. Thus, it could be concluded that the calculation process of ANSYS satisfies the calculation criteria of standard Riks method.

And the statistic calculation is adopted for data processing. Since the Weibull distribution is showed to have a higher design reliability, it is supposed to use Weibull distribution to generate random variables. And due to the complexity and time consumption of FEM calculation, 100 random variables of material properties are set by Weibull distribution. Then the histograms about bending capacity of each case are generated for discussion, which are showed in **Table** 5.7.

Table 5.7 *Histograms of bending capacity for FEM (Weibull Distribution)*

M_u/M_p	Slender	Standard	Compact
NCGs			
HCGs			

Using the same x and y axis for comparing each case. After statistical calculation, the average and standard deviation are obtained for analysis and discussion to evaluate the reliability of each case.

Since there are cross girder and several stiffeners in the two-girder model than the two-span conceptual model, the bending capacity M_u/M_p tends to be slightly greater than 1. The statistics result of FEM simulation is showed in **Table** 5.8

Table 5.8 *Statistics result of FEM simulation (Weibull Distribution)*

M_u/M_p	NCGs			HCGs		
section	slender	standard	compact	slender	standard	compact
average	1.14219	1.1437	1.18028	1.11388	1.12670	1.14051
standard deviation	0.064128	0.063464	0.062536	0.042917	0.038534	0.038894

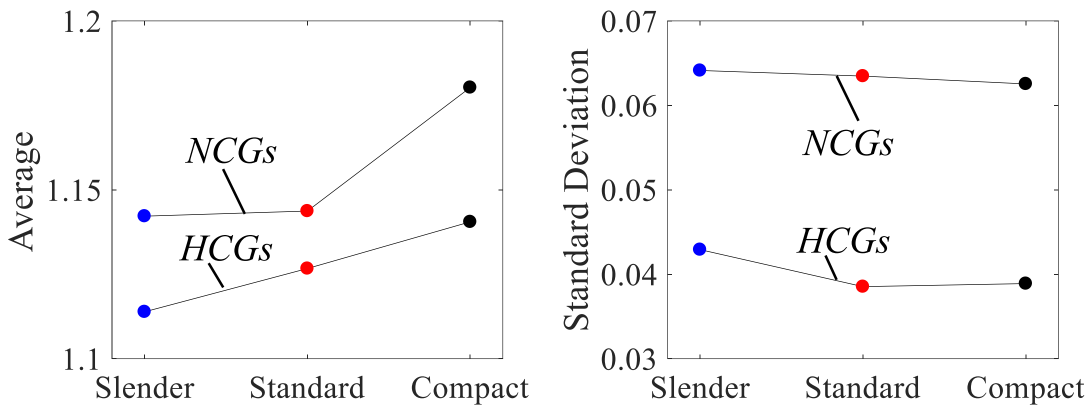


Figure 5.16 *Average and Standard Deviation of NCGs and HCGs (Weibull Distribution)*

The comparison charts about average and standard deviation are showed in the **Figure** 5.16. From **Figure** 5.16 (left), it could be observed that, with the same section dimensions, the average of bending capacity about NCGs shows to be larger than that of HCGs, which means that the NCGs models have a higher bending capacity than those of HCGs models in buckling calculations. Furthermore, with the same material composing, the larger the section dimensions are, the higher the average of bending capacity is, which means that a thick steel girder could give full play to the bending capacity of steel and compressive capacity of concrete.

And from **Figure** 5.16 (right), it could be observed that, with the same section dimensions, the standard deviation of bending capacity about HCGs showed to be lower than that of NCGs, which means that the HCGs models have fewer disperse samples than those of NCGs models in buckling calculations. Besides, with the same material composing, the larger the section dimensions are, the lower the standard deviation of bending capacity is, which means that a thick steel girder could provide a higher structural stability and guarantee a higher data concentration.

5.3.3 Results and Discussions about Structural Reliability

Then, after the statistical calculation, we could get the reliability index and failure probability of each model. Refer to the Chapter 4, according to the reference 16), in order to make the sample larger, there fixed $\alpha = 1.1$, then the reliability index and failure probability of each case could be calculated. These data are compared in the form of scatter plotted on the same graphs which are outlined in **Figure** 5.17, where the circles are about NCGs models and the diamonds are about HCGs models, and the blue, red and black are slender, standard and compact section in turn.

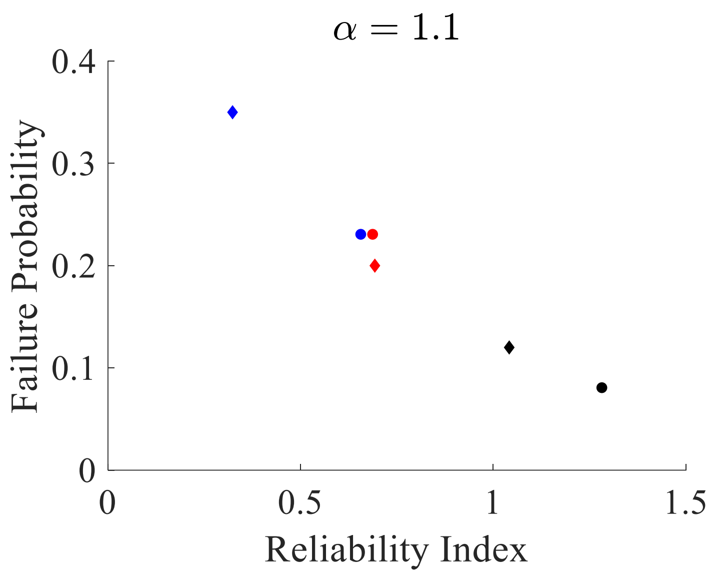


Figure 5.17 *Reliability index and Failure probability of FEM (NCGs and HCGs)*

From the **Figure** 5.17, when $\alpha = 1.1$, it could be observed that the distribution of the scatter diagram about structural reliability is regular. From the Chapter 4 we could know that a large reliability index and a small failure probability tend to have a higher structural reliability.

When $\alpha = 1.1$, it could be observed that, with the same material composing, the large the section dimensions are, the higher the reliability index is and the smaller failure probability is, the compact models have a higher structural reliability among the three different section dimensions.

Furthermore, with the same section dimensions, the NCGs models have a higher reliability than that of HCGs models in slender and compact section dimensions, while the HCGs models have a higher reliability than that of NCGs models in standard section dimensions. It might be caused by the abnormal samples.

Thus, it could be concluded that the compact models are more reliable in the buckling calculation, and the compact NCGs shows to have a higher structural reliability than that of compact HCGs. And the slender models are overvalued for the structural reliability, they are more likely to occur buckling and lost their bearing capacity.

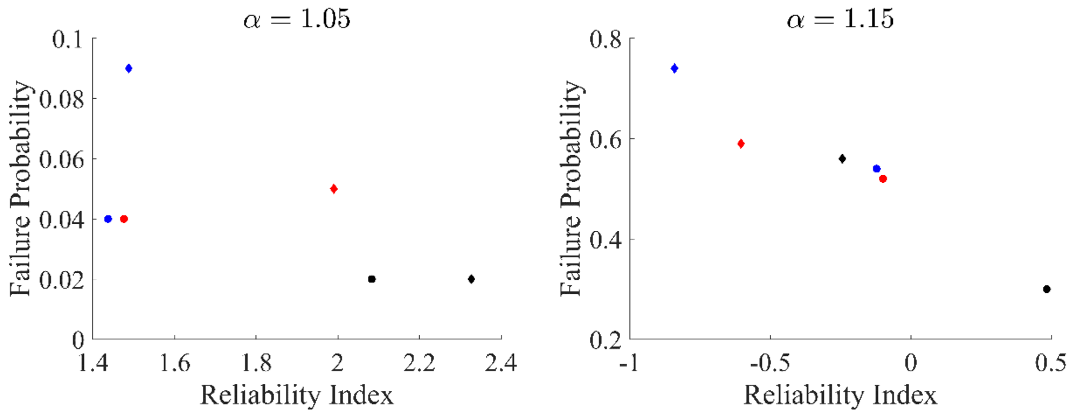


Figure 5.18 *Reliability index and Failure probability* ($\boldsymbol{\alpha = 1.05, 1.15}$)

And the scatter diagrams of structural reliability about $\alpha = \mathbf{1.05}$ and $\mathbf{1.15}$ are showed in **Figure** 5.18. From the **Figure** 5.18, it could be observed that with the increase of design load, buckling resistance performance of NCGs will be more prominent.

Furthermore, the histograms of destruction numbers of FEM are outlined in the **Figure** 5.19, Using the same horizontal axis (α: factor of safety) and vertical axis to compare each model directly. This time, to make the samples larger, we fixes α=1.05, 1.10 and 1.15 to set histograms.

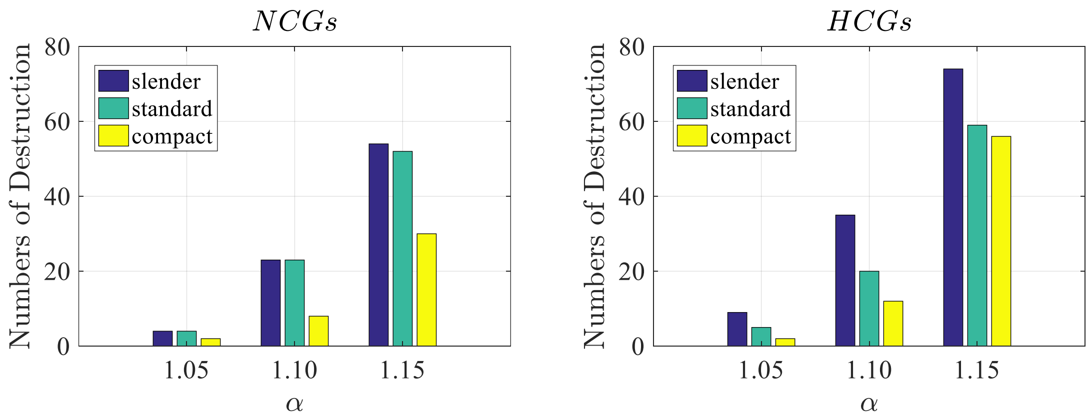


Figure 5.19 *Histograms of destruction numbers about FEM (NCGs and HCGs)*

The comparison charts about numbers of destruction are showed in the **Figure** 5.19, where the left is about NCGs and the right is about HCGs.

From **Figure** 5.19, it could be observed that with the same section dimensions, the destruction numbers of NCGs models are fewer than those of HCGs, which means that

NCGs shows a better performance in buckling analysis while HCGs are more likely to reach the buckling ultimate state.

Furthermore, it could be observed that with the same material composing, the yellow bar chart is the shortest in a set of data, which means that the compact models occur the fewest destruction with same situation. The compact models are more reliable in the buckling calculation, and the compact NCGs shows to have fewest destruction among these cases.

Thus, for FEM part, involving buckling analysis, compact section could be summarized to have a higher reliability and a better performance to restrain buckling.

To synthesize the results of Fiber method and FEM, the results of Fiber method are not completely credible when the structure has a thin shell part, and the buckling calculation should be considered in the simulation. The compact NCGs could be concluded to have a highest structural reliability among these situations and have a better performance to resist buckling.

Chapter 6. CONLUSION AND FUTUER WORK

6.1 Conclusion

The purpose of this study is to clear the stochastic capacity distribution of the ultimate limit state of HCGs and to evaluate them by comparing with those of NCGs. To find an appropriate structure with less material usage and make better use of the bending capacity of steel. Using Monte Carlo Method with Fiber method and FEM to do the simulation and calculate the bending capacity. Using member and material parameters and three kinds of probability functions (Normal Distribution, Log-normal Distribution and Weibull Distribution) to make the results more reliable.

The conclusions could be summarized as follows:

Chapter 1

- The background and objective of this research is introduced. The previous research on HCGs and the significance of the research is introduced.

Chapter 2

- Chapter 2 introduce the main simulation method used in the research. The idea and calculation process of Fiber method and FEM are introduced.
- Three typical probability functions (Normal Distribution, Log-normal Distribution and Weibull Distribution) are adopted as distribution for Monte Carlo method combined the Fiber method and FEM to improve the reliability of the simulation results.

Chapter 3

- The numerical model is introduced in Chapter 3, using MATLAB code to build one-girder model for Fiber method calculation and ANSYS program to build two-girder mode (close to engineering practice) for FEM calculation.
- The element type using for FEM calculation is introduced and a verification (bending of a composite beam) is outlined to verify the accuracy of ANSYS program.

Chapter 4

- In Chapter 4, the calculation process of Monte Carlo method is introduced. The data about variables of materials and section members are outlined for following calculation.
- The reliability index and failure probability, which are quantities for evaluating structural reliability, are introduced for reliability calculation.

Chapter 5

- The simulation results are outlined in Chapter 5. Statistical calculation is adopted to value the bending capacity of each situation.
- For Fiber method, it could be concluded that with the same section dimensions, the average and standard deviation of bending capacity about NCGs is a little larger than that of HCGs, which means that the NCGs models have a higher bending capacity than those of HCGs models and HCGs models have fewer disperse samples than those of NCGs models.
- Furthermore, with the same material composing, the average and standard deviation of bending capacity about slender models is a little larger than that of compact models, the larger the section dimensions are, the lower the average and standard deviation of bending capacity are, which means that a thick steel girder could not be made fully use of its bending capacity and provide a higher structural stability and guarantee a higher data concentration.
- Weibull Distribution contains more dangerous samples due to its high kurtosis and left skewed distribution, which means that using Weibull Distribution considers more unfavorable values which could make the design safer.
- For the reliability calculation, for slender model, when the design load is small ($\alpha = 0.9$), the HCGs have higher reliability with slender model while when the design load is large ($\alpha = 1.0$), the NCGs have higher reliability. For standard model, with the small design load, the HCGs have higher reliability with standard model while with the large design load, the NCGs have higher reliability. And for compact model using NCGs is showed to have fewer failure samples and could be safer. It could be concluded that with the same section dimension, when $\alpha = 0.9$, which indicated a smaller design load, it's better to design the composite girder as slender and standard for HCGs, compact for NCGs while when $\alpha = 1.0$, which indicated a larger design load, it's better to design all three kinds of the section dimensions as NCGs.
- For FEM, using Weibull Distribution (most unfavorable design) to set variables for calculation. Buckling limit state is considered as structural limit state for all situations since the upper edge of concrete slab does not reach ε_c in buckling state.
- From the statistical result, it could be concluded that, with the same section dimensions, the average and standard deviation of bending capacity about NCGs shows to be larger than that of HCGs, which means that the NCGs models have a higher bending capacity than those of HCGs models and the HCGs models have fewer disperse samples than those of NCGs models in buckling calculations.

Furthermore, with the same material composing, the larger the section dimensions are, the higher the average of bending capacity is and the lower the standard deviation of bending capacity is, which means that a thick steel girder could give full play to the bending capacity of steel and compressive capacity of concrete and provide a higher structural stability and guarantee a higher data concentration.

- For the reliability calculation, it could be concluded that, with the same material composing, the large the section dimensions are, the higher the reliability index is and the smaller failure probability is, the compact models have a higher structural reliability among the three different section dimensions. And, with the same section dimensions, the NCGs models have a higher reliability than that of HCGs models in slender and compact section dimensions while a lower reliability in standard section dimensions. And, with the increase of design load, buckling resistance performance of NCGs will be more prominent. Involving buckling analysis, compact section could be summarized to have a higher reliability and a better performance to restrain buckling
- To synthesize the results of Fiber method and FEM, the results of Fiber method are not completely credible when the structure has a thin shell part, and the buckling calculation should be considered in the simulation. The compact NCGs could be concluded to have a highest structural reliability among these situations and have a better performance to resist buckling.

6.2 Future Work

- It is possible to improve the parameters used in numerical simulation. Due to the different statistical data of references, materials and steel manufacturers, it is possible to further rationalize the results by using more realistic values.
- Due to the complexity and time-consuming of the FEM buckling analysis, only 100 variables are generated for simulation in this research. In the future work, we should try to simplify the calculation process and use more variables to make the results more reliable.
- For the structure with thin shell part, Fiber method is not enough for calculation. In order to make the result more reliable, buckling analysis of FEM should be added into calculation. To make fully use of performance of each material, a more appropriate section dimension could be found for the design of composite girder in the future work, which should satisfy that the structure achieves the failure of upper edge of concrete slab and buckling ultimate state simultaneously.

REFERENCES

1) Japan Road Association: Road Bridge Specifications, 2017. (in Japanese)
2) Kyosuke Yamamoto, Hitotaka Kouno, Kuniyuki Sugiura and Yoshinobu Oshima, Effect of Material Properties on Bending Capacity Problem Distribution of Steel-Concrete Hybrid Composite Girders, Journal of concrete engineering, Vol.30, No.3, 2008. (in Japanese)
3) Kyosuke Yamamoto, Hitotaka Kouno, Kuniyuki Sugiura, Yoshinobu Oshima and Tarou Tonegawa, Effect of material plastic properties on ultimate Bending Capacity of Hybrid Composite Girder, 62nd Annual Lecture of Civil Engineering Society, 2007. (in Japanese)
4) Yukio Maeda, Yasuharu Kajikawa and Masao Ishiwata, Bending Behaviors and Maximum Load-Caring Capacity of Hybrid Composite Beams, Kawasaki Technical Report, Vol.10, No.1, pp.86-99, 1978. (in Japanese)
5) Masutsugu Nagai, Takeshi Miyashita, Cuiping Liu, Naofumi Inaba and Atsushi Homma, Design and Application of Steel and Steel-Concrete Plate Girds Bridges with Hybrid Section, Journal of Japan Society of Civil Engineers A1, Vol68, No.1, pp.203-215, 2012. (in Japanese)
6) Katsuki Egashira, Shozo Nakamura, Satoshi Araki and Kazuo Takahashi, Design Formula of Positive Flexible Strategic Strength of Composite I-Sections Conflicting Variation of Material Properties, Journal of Japan Society of Civil Engineers A, Vol63, No.4, pp.576-585, 2007. (in Japanese)
7) Satoshi Araki, Shozo Nakamura, Katsuki Egashira, Kazuo Takahashi and Qingxiong Wu, Influence of steel properties in plastic region on probability distribution of flexural strength of steel-concrete composite girders under positive bending, Journal of structural engineering, 51A, pp.1247-1255, 2005. (in Japanese)
8) Katsuki Egashira, Shozo Nakamura, Kazuo Takahashi and Qingxiong Wu, Influence of steel properties on the positive flexural strength of steel-concrete composite sections, Journal of structural engineering, 49A, pp.791-789, 2003. (in Japanese)
9) M.F Hassanein, A.A. ELkawas, Yong-Bo Shao, M. Elchalakani and A.M. El Hadidy, Lateral-Torsional buckling behavior of mono-symmertric S460 corrugated web bridge girders, Thin-Walled Structures, 153 (2020) 106803.
10) Shun-Fa Hwang, GUU-Huann Liu, Buckling Behavior of Composite laminates with multiple delaminations under uniaxial compression, Composite Structures, 53 (2001) 235-243.

11) Yoshio Miyata, Shozo Nakamura and Kazuo Takahashi, Influence of probability distribution on the failure probability of steel I-girder bridges, Annual Journal report of steel structures, 19, pp.63-68, 2011. (in Japanese)
12) ANSYS Mechanical APDL Connection User's Guide (2012)
13) R. J. Roark, W. C. Young, Formulas for Stress and Strain, McGraw-Hill Book Co., Inc., New York, NY, 1975, pg. 112-114, article 7.2.
14) Mysayoshi Ito, Satoshi Ino, Akira Suginome and Takeyoshi Uematsu, Investment on Construction Accuracy of Trusted Concrete Construction at Trust Construction Sites, Journal of concrete engineering, Vol.16, No.1, 1994. (in Japanese)
15) Satoshi Nara, Shozo Nakamura, Hiromichi Yasunami, Fumimaru Kawabata and Toyaaki Shiwaku, A Statistical Survey on Thickness and Scientific Properties of Structural Steel Plates for Bridges, Journal of Japan Society of Civil Engineers, No.752, I-66, pp.299-310, 2004. (in Japanese)
16) Masatsugu Nagai, Takeshi Miyashita, Guiping Liu, Naofumi Inaba and Atsushi Homma, Design and applicabicability of steel and steel-concrete plate girder bridges with hybrid section, Vol.68, No.1, 203-215,2012. (in Japanese)
17) Andrea Bucchi, Grant E. Hearn, Predictions of aneurysm formation in distensible tubes: Part A—Theoretical background to alternative approaches, International Journal of Mechanical Sciences 71 (2013) 1-20

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