Flexural Reliability of NU Girders

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ABSTRACT

The NU I-girder series, developed by the University of Nebraska's Center for Infrastructure Research in cooperation with the Nebraska Department of Roads, spans farther than any other standard I-girder shape available today. The new girders have depths ranging from 750 to 2400 mm (30 to 95 in.), with constant top and bottom flange dimensions. The girder's cross section provides several advantages. The wide top flange allows for better worker-platform and shorter deck-slab spans. The wide and thick bottom flange enables increased strand capacity for simple spans and provides increased negative moment capacity for continuous spans. The bottom flange is increased stability in shipping and handling. These unique characteristics significantly affect the flexure and s hear resistance of NU girders, and consequently, influence their reliability when the uncertainty of loads, material properties, and fabrication parameters are considered. This paper presents the reliability analysis of NU girders designed using the AASHTO LRFD bridge design specifications. Three types of NU girders (NU1600, NU1800, and NU2000) with different ratio of dead load to live load are considered. Load and resistance parameters are treated as random variables. The statistical parameters are based on the available literature, available and actual test data and load surveys. Considering the fact that target reliability index is 3.5, these girders showed to have a reliability index slightly over target reliability for the practical range of D/(D+L) ratio with a rather uniform distribution.

Key words: Reliability, NU girders, Load and Resistance Model, Bridges

BACKGROUND

Reliability-based bridge design codes are developed to provide bridge engineers with rational criteria to evaluate the adequacy of the design using the theory of probability and statistics. Loads and resistance parameters are treated as random variables and the adequacy of the design is quantified in terms of the reliability index (β) or probability of failure. Examples of these codes are: AASHTO LRFD Bridge Design Specifications, Canadian Highway Bridge Design Code, and Euro code. The basic design formula in reliability-based bridge design codes is:

$$\Sigma \gamma_i Q_i < \phi R_n \tag{1}$$

Where:

γ_i = load factor

Q_i = nominal (design) load component

ϕ = resistance factor

R_n = nominal (design) resistance

Load and resistance factors are obtained through the code calibration procedures (Nowak 1995). In this procedure, load models are developed to account for uncertainty in different types of loads using a statistical database that includes the results of surveys and observations of load values and frequencies. Resistance models are developed to account for the uncertainty in material properties and fabrication parameters using a statistical database that includes the results of material tests and field measurements. Reliability analysis is then performed using simulation techniques to estimate structural reliability (in terms of β), or probability of failure (i.e. probability of having load effects greater than the resistance). Finally, load factors (γ_i) are calculated so that factored loads have the same probability of being exceeded, and resistance factors (ϕ) are calculated so that designed components have a consistent reliability index that is close to a predefined target reliability.

For the reliability analysis of any structure, two statistical parameters are needed for each load and resistance random variable: bias factor (λ) and coefficient of variation (V). Bias factor is the ratio of mean to nominal value, and coefficient of variation is the ratio of standard deviation and the mean. Extensive research has been conducted to estimate these parameters for bridge loads and component resistance. Examples are: Nowak and Szerszen (1998) for load parameters; Nowak et al. (1994) and Nowak (1995) for the flexure and shear resistance of reinforced concrete T-beam, precast prestressed concrete AASHTO girder, and steel girder; and Nowak and Szerszen (2003) for the flexure and shear resistance of reinforced/prestressed concrete beams, slabs, and columns. Although these statistical parameters cover a wide range of structural components, further analysis needs to be done when new components or material are encountered.

The University of Nebraska’s I-girders, known as NU girders, are precast prestressed concrete girders that have unique characteristics. Figure 1 shows the cross section of NU girders versus the cross section of standard AASHTO girders. The wide top flange of NU girders provides an adequate platform for workers and shorter span for decks. The wide and thick bottom flange of NU girders accommodates a large number of prestressing strands that improve the section capacity, in addition to increasing girder stability during construction. The curved fillets of NU girders reduce stress concentration and improve the flow of concrete during casting. Table 1 lists the 6 standard sections of NU girders along with their geometric properties. These sections allow the girders to span from 90 ft to 210 ft with a high economic competency. NU girders are typically manufactured using high strength self-consolidating concrete (f_c' ranges from 8 to 12 ksi), 0.5 or 0.6 in 7-wire uncoated low-relaxation strands, and steel reinforcement of 60 or 75 ksi yield strength.

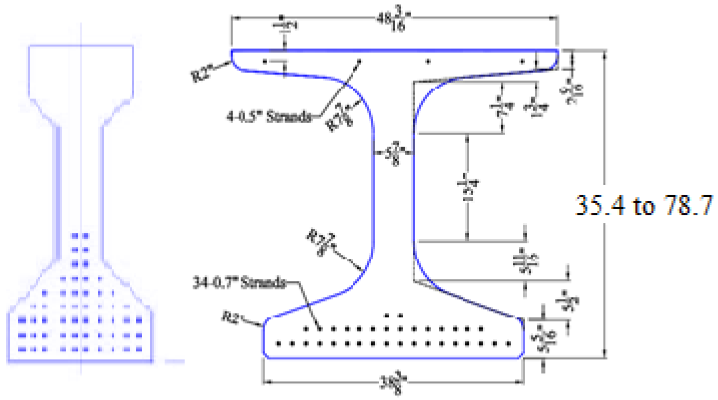

Fig. 1 The cross section of NU girders (right) and standard AASHTO girders (left)

Table 1: Section properties of different NU girders (NDOR 2005)

Girder	Height in.	Web Width in	Top Flange in.	Bottom Flange in.	Area in^2	Yb in.	Inertia in^4	Weight Lbs/Ft
NU 900	35.4	5.9	48.2	38.4	64B.1	16.1	110,262	680
NU 1100	43.3	5.9	48.2	38.4	694.6	19.6	182,279	724
NU 1350	53.1	5.9	48.2	38.4	752.7	24.0	302,334	785
NU 1600	63.0	5.9	48.2	38.4	810.8	28.4	458,482	840
NU 1800	70.9	5.9	48.2	38.4	857.3	32.0	611,328	894
NU 2000	78.7	5.9	48.2	38.4	903.8	35.7	790,592	942

The objective of this paper is to evaluate the reliability of NU girders designed using the AASHTO LRFD bridge design specifications. To achieve this objective, reliability analysis is performed for the flexure capacity of three NU girders. The next section presents the approach used to investigate the reliability of NU girders. The following sections present the limit state function, load model, resistant model which developed for several types of NU girders and considering different ratios of dead to live loads. The variability of flexure is calculated based on the uncertainty of material properties and fabrication parameters. The last sections present the results and conclusion of the study.

APPROACH

In this study, reliability of NU prestressed girders in moment will be assessed in bridges which are designed according to "AASHTO LRFD Bridge Design Specifications, 4th edition, 2007". The girders were assumed to be in simple spans. Three different NU girders with different heights were simulated in the project. The girders are known as NU 1600, NU 1800 and NU 2000. The reliability of the models is calculated on the basis of reliability theory which is provided in the literature (Nowak and Collins 2000). It is a technique that can be used when there is no closed form solution for the problem. Using it, all parameters in the limit state function will be generated randomly considering their uncertanity factors and distribution types. Then the value of limit state function will be calculated to see if any failure has occured. This process repeats until a number of failures occur. The accuracy of the method increases as the number of failures increases. In the current study, all the analyses were continued until after 30 failures occured. Having the total number of similations and failures, the probability of failure and reliability index are estimated as :

$$P_f = N_f / N \tag{2}$$

Where
pf = probability of failure;
Nf = number of failures;
N = total number of simulatios;
The relationship between the reliability index and the probability of failure is:

$$\beta = \varphi^{-1}(-P_f) \tag{3}$$

Where β is reliability index, and φ^{-1} is the standard normal inverse function. In this study, reliability analysis was carried out using Monte Carlo simulation. Different ratios of dead load to total load (D/(D+L)) employed to account for various loading conditions that a member may undergo during the service. Because of different uncertainity facors for live and dead loads, this parameter effect needs to to be investigated.

LIMIT STATE FUNCTION

Limit state function which is defined as the difference between the resistance of, R, and the load effect acting on the member, Q. Ultimate limit state function relates to the loss of load carrying capacity of the member. Assuming R as the capacity of the member and Q as the load effect, the limit state function can be defined as :

$$g(R,Q)=R-Q \tag{4}$$

where g is the limit state function. If g>0, the member is in the safe margin. It means that the capacity is greater than the load effect. In contrast, if g<0, the member fails because the load effect is greater than the capacity. The probability of occurence of this event is called probability of failure.

In the current study, the nominal moment capacity of the section is calculated based on the determined section properties including dimensions, concrete strength and strands grade and numbers. The moment capacity was calculated using strain compatibility method. This is a repetitive formula which needs some repetitions to reach the exact moment capacity of the section. Based on the nominal moment capacity of the section the nominal loads were assumed to be the same as the nominal capacity. Then, the load ratios were used to obtain dead and live loads. Wearing dead load and dynamic load were also considered in dead and live loads, respectively.

LOAD MODEL

A two lane roadway bridge is considered in the study. Dead load and live load are taken into account for the analyses. Load factors (γ_i) in the AASHTO LRFD Bridge Design Specifications are equal to 1.25 and 1.75 for dead and live load respectively:

$$Q = 1.25\,D + 1.75\,L\,(1 + I) \tag{5}$$

In this study, latest load models based on existing statistical data are used. A summary of collected data and observations is provided in Calibration of LRFD Bridge Design Code – NCHRP 368. It should be noted that current load factors in AASHTO Bridge Design Code, are based on a comprehensive reliability study for design of steel and concrete girder bridges as the most common bridge systems. Table 2 shows the bias factors and coefficients of variation for dead load and live load considered for the simulations, offered for maximum dead and live loads. It is of note that dead load is assumed having normal distribution with a mean value and variation of 1.05 and 0.10, respectively (Ellingwood et al 1980).

Table 2: Uncertainty parameters for load models

Load Type	Distribution	Arbitrary point in time		75 year maximum load	
		λ	V	λ	V
Dead Load	Normal	1.05	0.10	1.05	0.10
Live Load + Dynamic (I)	Lognormal	<1	0.2 -0.25	1.21	0.18

RESISTANCE MODEL

The actual shear resistance (R) is defined as the product of nominal shear resistance (R_n), and the factors considering the uncertainity in materail (M), geometry (F), and model error (P). Thus , the mathemathical model of resistance is of the form:

$$R = R_n M F P \tag{6}$$

The bias factor and coefficient of variation of resistance can also be calculated as :

$$\lambda_R = \lambda_M \times \lambda_F \times \lambda_P \tag{7}$$

$$V_R = \sqrt{{V_M}^2 + {V_F}^2 + {V_P}^2} \tag{8}$$

Where λ and V are the bias factor and coefficient of variation of the parameters specified by the subscription. Material factor also accounts for the variation in the material strength. Variation in member dimension is included in geometry factor. Professional factor deals with the errors of the model used in the prediction of the resistance of the member.

Ellingwood et al (1980) suggested uncertainty factors for resistance of different types of structural elements, applying M, F and P factor parameters. The uncertainty parameters are shown in Table 3.

Material factors for concrete and strand is considered in the resistance. Concrete compressive strength uncertainty parameters are shown in Table 3. In the table the bias factor of this parameter decreases from 1.24 to 1.08 as the nominal compressive strength increases from 4 ksi to 12 ksi. Also, the coefficient of variation decreases from 0.125 to 0.11 as the nominal strength increases from 4 ksi to 12 ksi. Strand material uncertainty parameters are also shown in the table. All strands are of 0.6 in. diameter and grade 270. The bias factor and coefficient of variation of the ultimate tensile strength of these strand were assumed 1.02 and 0.015, respectively.

Fabrication and professional uncertainties considered in the resistance formula are shown in Table 4. The effective depth of girders and slab thickness uncertainty parameters were only considered in the simulations. These factors have the greatest effect on resistance among all dimension parameters. In all cases the nominal depth and width of the concrete slab were 8” and 10’, respectively. The area of the strands was deterministic in analyses. Professional factor is a factor that considers the uncertainty of the design formula. This parameter is assumed to have a bias factor of 1.01 with a variation of 0.06.

Table 3: Uncertainty parameters for materials

Material factors	Distribution	Nominal Strength (ksi)	Uncertainty parameters	
			λ	**V**
Concrete	Normal	4	1.24	0.125
		6	1.15	0.11
		8	1.11	0.11
		10	1. 09	0.11
		12	1.08	0.11
Strand	Normal	270	1.02	0.015

Table 4: Fabrication and professional factors

Uncertainty factor		**λ (+in)**	**V (in)**
Fabrication	Slab thickness	0.21	0.26
	Girder depth	0.81	0.55
Professional		1.01	0.06

It is of note that strain compatibility method was used for calculating flexural resistance of girders.

RESULTS

The reliability index is defined as the inverse of the coefficient of variation of the function g = resistance (R) – load (Q), when R and Q are uncorrelated. This can be expressed mathematically by the following equation:

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \qquad (9)$$

Where:

β	= reliability index.
μ_R	= mean value of resistance
μ_Q	= mean value of load effect
σ_R	= standard deviation of resistance
σ_Q	= standard deviation of load effect

Figures 2 to 10 present results of Monte Carlo simulations and reliability indices in various cases. Figures 2 to 4 represents the reliability curves for NU1600 in different levels of strand in the girder section. Also, Figures 5 to 7 and 8 to 10 represents the reliability curves for NU1800 and 2000, respectively. Moreover, in each figure the four diagrams stand for different concrete strengths used in the girder. It is of note that all the diagrams are rather similar in terms of the amount of reliability index and the trend. Reliability indices show to be close to 3.5 for loading ratio of 0.6 and less and decrease to a value close to 3.0 thereafter.

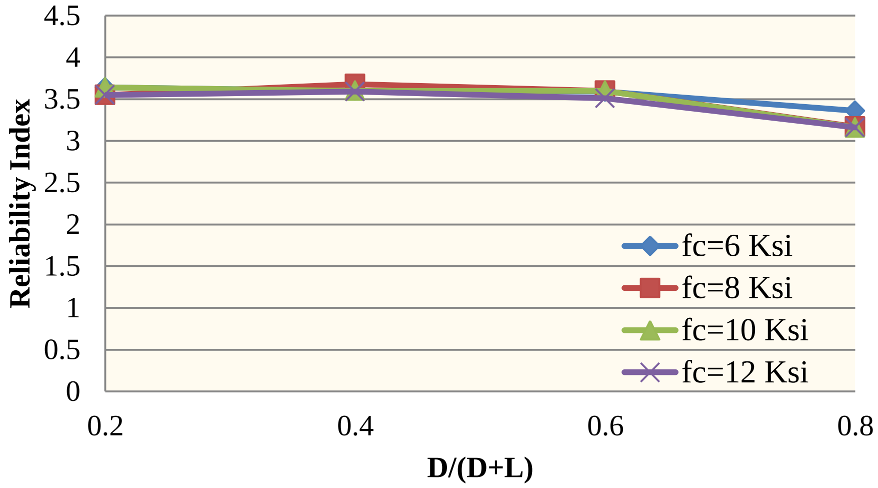

Fig. 2 Reliability indices, simple span moments in NU-1600 girders (Low strand)

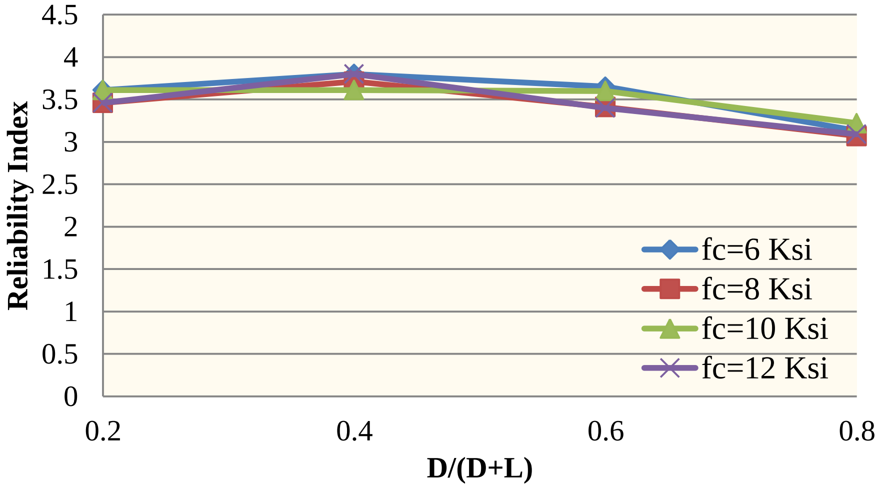

Fig. 3 Reliability indices, simple span moments in NU-1600 girders (Avg. strand)

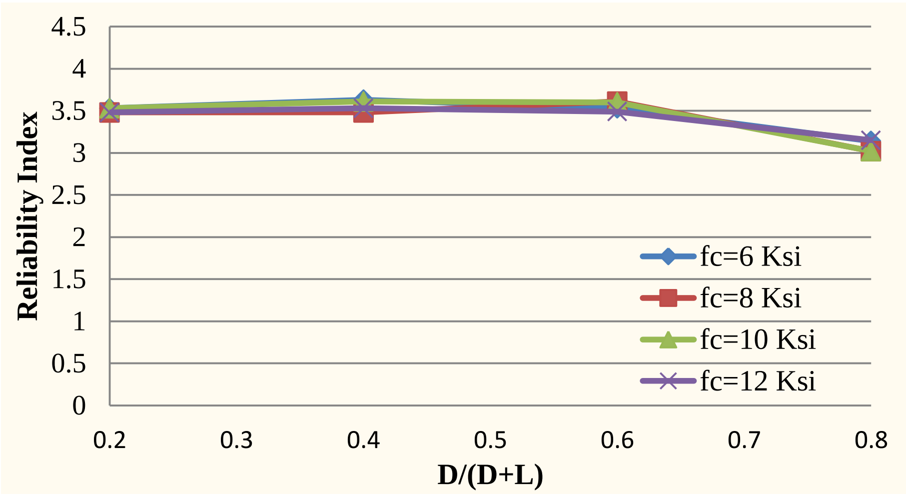

Fig. 4 Reliability indices, simple span moments in NU-1600 girders (High strand)

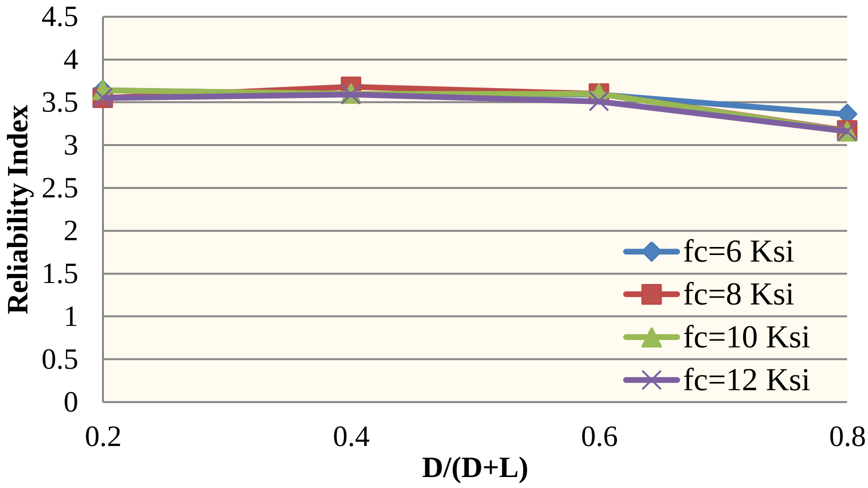

Fig. 5 Reliability indices, simple span moments in NU-1800 girders (Low strand)

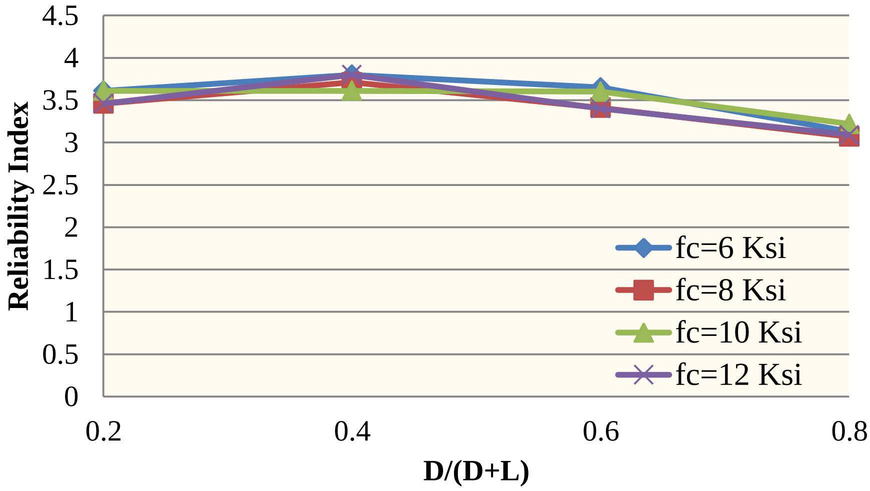

Fig. 6 Reliability indices, simple span moments in NU-1800 girders (Avg. strand)

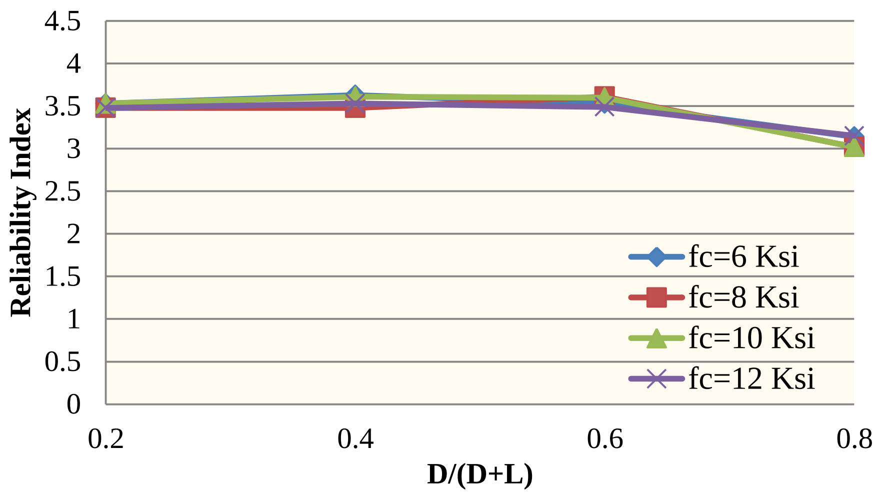

Fig. 7 Reliability indices, simple span moments in NU-1800 girders (High strand)

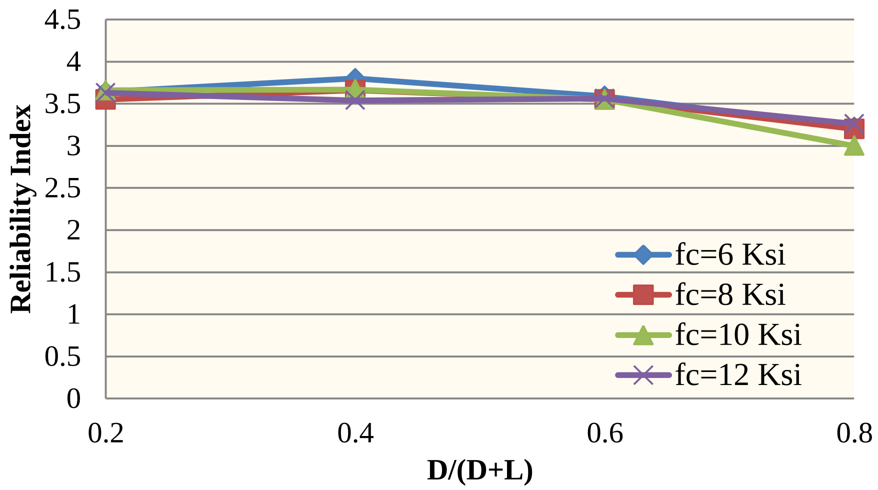

Fig. 8 Reliability indices, simple span moments in NU-2000 girders (Low strand)

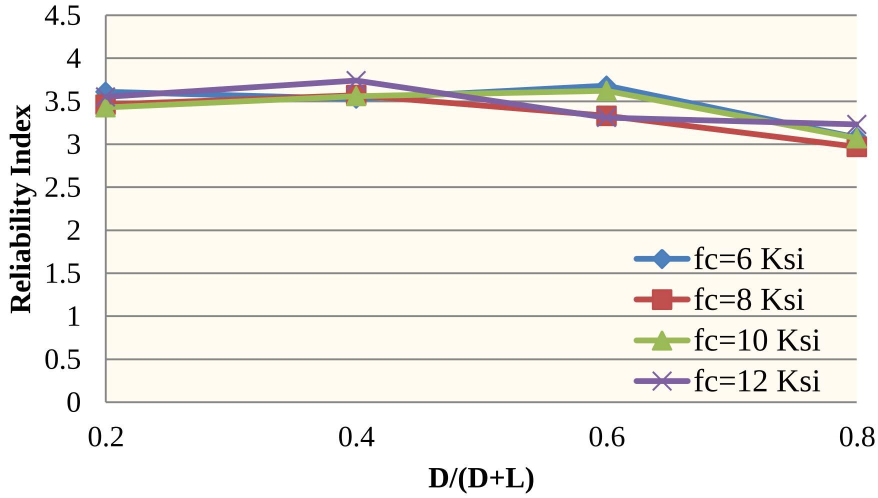

Fig. 9 Reliability indices, simple span moments in NU-2000 girders (Avg. strand)

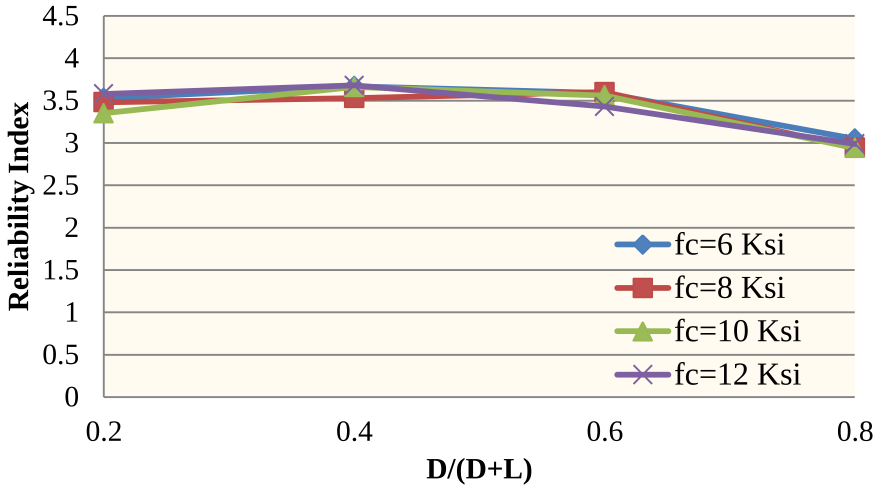

Fig. 10 Reliability indices, simple span moments in NU-2000 girders (High strand)

CONCLUSION

According to analysis results for reliability, it can be seen that for these girders the reliability indices is mainly over 3.5 when D/(D+L) ratio is between 0.2 and 0.6 and decreases to a value of 3.0 when load ratio is 0.8. This range covers from very short to long span length which, In practice, small values for this ratio mean a very short span bridge which can have a large amount of live load with respect to its dead load. On the other hand, high values of D/(D+L) might be expected for very long span bridges which is not is covered by AASHTO code and even these girders . Considering the fact that target reliability index is 3.5, it appears that the reliability of these girders are very close to target reliability index. For practical span length where load ratio is between 0.4 and 0.6 the reliability indices are slightly over 3.5. Thus making these girders have a nearly uniform reliability under various loading conditions. Based on the results no change is needed in the current design procedure unless different uncertainty parameters are introduced in the future.

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