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Direct Design of Steel Bridge Using Nonlinear Inelastic Analysis

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***Abstract*—**In this paper, a direct design using a nonlinear inelastic analysis is suggested. Also, this paper compares the load carrying capacity obtained by a nonlinear inelastic analysis with experiment results to verify the accuracy of the results. The allowable stress design results of a railroad through a plate girder bridge and the safety factor of the nonlinear inelastic analysis were compared to examine the safety performance. As a result, the load safety factor for the nonlinear inelastic analysis was twice as high as the required safety factor under the allowable stress design standard specified in the civil engineering structure design standards for urban magnetic levitation railways, which further verified the advantages of the proposed direct design method.

***Keywords*—**Direct design, nonlinear inelastic analysis, residual stress, initial geometric imperfection.

I. Introduction

ACCORDING to current allowable stress design(ASD) and load-resistance factor design(LRFD), a linear-elastic analysis is applied to structures and a nonlinear inelastic analysis is then considered implicitly in the design equation. When undertaking an analysis on the assumption that a structural system has linear-elastic behavior although it in fact has nonlinear inelastic behavior and then considering the nonlinear inelastic behavior implicitly in the capacity check equation of a separate member, a discrepancy arises between the analysis and the design. To overcome this problem, a nonlinear inelastic analysis should be used to analyze the overall structural system. It was necessary to use conventional analysis and design methods before the development of computers. However, it has recently become relatively easy to carry out nonlinear inelastic analyses with the development of the computer and structural analysis theory. Accordingly, the design standards of many countries recommend the utilization of a nonlinear inelastic analysis, and its application to design practices has been on the rise AISC [1], AASHTO-LRFD [2], JSSC [3].

The design time required for a separate member capacity check can be dramatically reduced by replacing the ASD and LRFD components currently being used in design practices with a direct design method using a nonlinear inelastic analysis. In addition, this method allows the efficient design of structures, as it can reduce the size of a member section due to the redistribution of the inelastic moment and can ensure the safety of structures owing to an accurate estimation of the steel girder bridge behavior.

Accordingly, this paper presents the direct design of a railway through a plate girder bridge based on a nonlinear inelastic analysis considering the residual stress and the initial geometric imperfection. Also presented is a method of evaluating the load resistance capacity. Also, the allowable stress design results of a railroad through a plate girder bridge and the safety factor of the nonlinear inelastic analysis were compared to investigate the economic feasibility and safety.

II. Nonlinear Inelastic Analysis Using a Direct Design

A. Design Formula

The formula of the direct design presented here is identical to that of the LRFD design:

$$\phi R_n \geq \sum_{i=1}^{m} \eta_i \gamma_i Q_{ni} \tag{1}$$

In this equation, the following notations are defined: R_n = nominal resistance of a structural member; Q_{ni} = nominal load effect (e.g., shear force, bending moment); ϕ = resistance factor ($\leq$ 1.0) (e.g., 0.9 for beams, 0.85 for columns); η_i = load modifier; γ_i = applied load factor (usually > 1.0) corresponding to $Q_{ni;}$ i = type of load (e.g., D=dead load, L=live load, S=snow load); m = number of load types

However, in the case of the LRFD design, ϕR_n refers to the member strength; and η_i , γ_i , and Q_{ni} denote the member force. In contrast, for the direct design, ϕR_n refers to the load resistance capacity of the structural system and η_i , γ_i , and Q_{ni} represent the load imposed on the structural system.

III. Items to Be Considered at the Time of Nonlinear-Inelastic Analysis

To estimate the behavior of a steel structure accurately, the residual stress and initial geometric imperfection should be considered when the nonlinear inelastic analysis modeling is done.

A. Residual Stress

The residual stress distribution of the welded I section in Fig. 1, as is recommended by the ECCS Technology Committee 8 [9], can be used.

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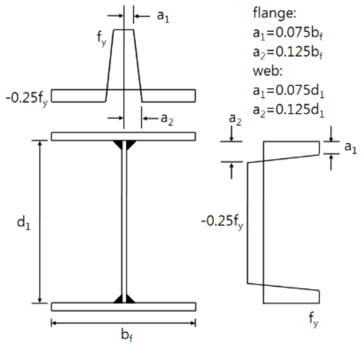


Fig. 1 Residual stress distribution diagram of welded I section

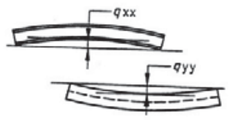
B. Initial Geometric Imperfection

The initial geometric imperfection includes the overall geometric imperfection and the local geometric imperfection. Mode superposition with the following two methods is used during the modeling process [4].

1. Overall Geometric Imperfection

Regarding the largest of the geometric imperfections, the values in Table I, coming from EN 10034 of the European Standard [5], can be used.

TABLE I
ALLOWABLE TOLERANCES

Item	Sectional height(mm)	Tolerance	Remarks
Curve	80 < h < 180	Below member length×0.30%	
	180 < h ≤ 360	Below member length×0.15%	
	h > 360	Below member length×0.10%	

2. Local Geometric Imperfections

For the largest local geometric imperfections, the values in Table II and Fig. 2, sourced from Eurocode 3 [6]-[8], can be used.

TABLE II
SIZE OF THE LOCAL GEOMETRIC IMPERFECTION

Initial imperfection types	Element	Shape	Size
Local	Panel or sub-panel	bucklin g shape	Minimum (a/200, b/200)

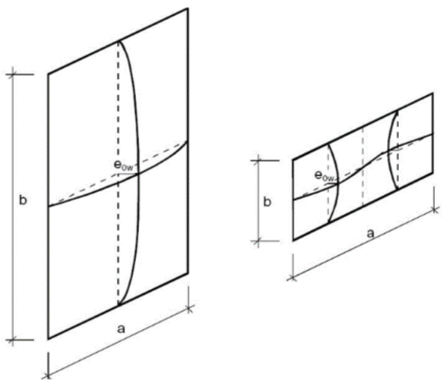


Fig. 2 Local geometric imperfection shape

IV. Nonlinear Inelastic Analysis

A. Through-Plate Girder Bridge Modeling

The length of the through-plate girder bridge is 26,567 mm in the member longitudinal direction. Grade SM490 steel was used for the main girder, cross beams, string beams, and stiffening members, while SS400 was used for the bottom bracing. The geometric shapes and finite element model is displayed in Fig. 3. In the modeling of all structural members, including the main girder, cross beams, string beams, as well as bracing and stiffening members, S4R, ABAQUS's shell element, was used. In this analysis, a total of 37,834 elements with dimensions of 95mm×95mm were used so that a structure similar to the actual shape could be used in the modeling. A combination of the proper load conditions should be selected for a railroad bridge, but it is not necessary to consider all types of loads at the same time. In this study, Case 1 was applied to the modeling analysis, representing the most basic load in railroad bridges, as well as Case 2, which includes a wind load and traction braking load, as shown in Table III. In the primary load cases, a dead load and a live load were applied, while in the special load case, a snow load was applied.

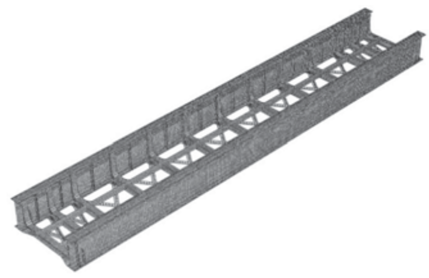


Fig. 3 Finite element modeling

TABLE III
LOAD COMBINATION IN A STEEL BRIDGE

CASE	Load combination
1	primary load + special load equivalent to the primary load
2	primary load + special load equivalent to the primary load + wind load + traction or braking load

B. Allowable Stress Design

The applied stress and allowable stress in the center of the main girder were calculated regarding the working load, in this case a dead load, live load, wind load, and traction braking load. Additionally, Table IV shows the sectional force, including the bending moment, shear force, and torsional moment, in the center of the main girder of a railroad through a plate girder bridge as calculated based on the working load. The applied stress for the working load given in consideration of bending and shear was calculated by dividing the stress into the upper edge stress under compression and the lower edge stress under tension. The calculated applied stress and allowable stress values for bending, shear, and composite stress were reviewed, and they were all found to be satisfactory.

C. Direct Design Using a Nonlinear Inelastic Analysis

A nonlinear inelastic analysis was conducted for the through-plate girder bridge in consideration of the residual stress and initial geometric imperfection. During this modeling process, the residual stress distribution recommended by ECCS Technology Committee 8 [9] in Fig. 1 was used. A maximum residual stress of 320MPa was used as well, like the yield stress, and the *INITIAL CONDITIONS option of the ABAQUS program was used in the modeling of the residual stress distribution.

TABLE IV
APPLIED SECTIONAL FORCE IN THE CENTER OF THE MAIN GIRDER

Item (Unit)	dead load	live load	Wind Load 1	wind load 2	traction or braking load
M (kN·m)	1402.02	1165.65	143.61	315.73	120.16
V (kN)	13.66	59.55	4.71	18.68	1.96
Mt (kN·m)	0.00	0.26	0.00	0.02	0.00

For the application of the initial geometric imperfection, the *BUCKLE option was used to obtain the Eigenmode, and the *IMPERFECTION option was then used to calculate the scale. Eigenmode performed an analysis using mode superposition with the overall geometric imperfection and local geometric imperfection. Based on Table I, the maximum size of the overall geometric imperfection, which was 26.5 mm, equivalent to 0.1% of the member length, was applied mid-span in the X direction. The local geometric imperfection, the applied maximum size of the local geometric imperfection was 8.5 mm, which is equivalent to 1/200 of the web height based on Table II.

A perfectly elastic-plastic model was applied to the steel material model. The *STATIC, RIKS option was selected, which can identify post-buckling behavior in a structure via a nonlinear inelastic analysis. Regarding two cases in which the residual stress and initial geometric imperfection of a through-plate girder bridge are either considered at the same time, Load cases 1 and 2 were compared. The results are given in Fig. 4.

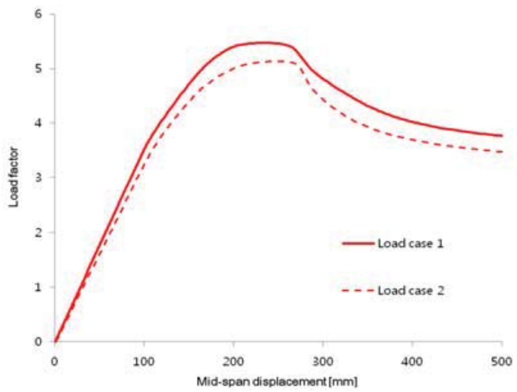


Fig. 4 Load factor-displacement relationship at the center of the right span

D.Design Results

To evaluate the load carrying capacity of a railroad through a plate girder bridge, the load factor from the nonlinear inelastic analysis and the load amplification factor (safety factor) from the allowable stress design were compared. These results are summarized in Table V. The load factor obtained by the nonlinear inelastic analysis of the structural system, as calculated in the table above, corresponds to the safety factor of the structural system in (F.S):

$$F.S. = \frac{R_n}{\sum_{i=1}^{m} Q_{ni}} \tag{2}$$

According to the ASD specified in Civil Engineering Structure Design Standard for an urban magnetic levitation railway [10], the safety factors of separate member yielding points are between 2.50 and 3.11. The nonlinear inelastic analysis found that the safety factors of the structural system for each load combination ranged from 5.13 to 5.47, which is approximately two times higher than the safety factor required by the allowable stress design standard. Thus, the structural system of this plate girder bridge is shown to reduce the section, indicating that the proposed design method is economically advantageous.

TABLE V
LOAD CARRYING CAPACITY AND SAFETY EVALUATION OF THE OVERALL STRUCTURAL SYSTEM

Load case	Load factor at the time of the nonlinear inelastic analysis	Safety factor at the time of the allowable stress design
1	5.47	3.11
2	5.13	2.50

V.CONCLUSION

This paper analyzed the items to be considered at the time of a finite element analysis of the results of a nonlinear inelastic analysis to predict the behavior and strength of a structure feasibly. After a direct design using nonlinear inelastic analysis was carried out considering the residual stress and initial geometric imperfection of railroad through a plate girder bridge, the following conclusions were drawn.

The load safety factor calculated by the direct design using a nonlinear inelastic analysis of a railroad through a plate girder bridge was 5.13~5.47, which is two times higher than the safety factor (2.50~3.11) required in the allowable stress design standard. Therefore, the direct design using the nonlinear inelastic analysis as presented here is verified as an economically advantageous method.

ACKNOWLEDGMENT

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