An Evaluation of Inelastic Behavior of a Cable Supported Bridge under Earthquake Load
지진하중을 겪는 케이블 지지 교량의 비탄성 거동 평가

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ABSTRACT

케이블 장대 교량의 해석에 있어서 기하적인 비선형만을 고려한 해석이 보편적이었다. 하지만 이 연구에서는 내진 해석시 케이블지지 교량이 비탄성적으로 거동 할 수 있기 때문에, 기하적인 비선형 이외에 재료적인 비선형을 고려할 필요가 있음을 보이고자 한다. 극한 하중 상태를 모사하기 위하여 사하중에 하중계수를 곱하여 하중을 증가시켜 중력방향으로 하중을 가하였고, 지진에 대한 하중 상태를 모사하기 위하여 교축방향의 지진 하중에 대한 등가의 등분포 하중과 이의 0.3배에 해당하는 수직 방향 하중을 동시에 가하였다. 이러한 해석을 통하여 자중의 2배 이상의 하중이 가해지면 거더가 비탄성적으로 거동 할 수 있고, 또한 교축 방향과 수직 방향의 설계지진하중을 고려할 경우 수평방향의 구속이 모두 풀리면 주탑이 비탄성적으로 거동 할 수 있음을 알 수 있다. 따라서 케이블지지 교량의 지진 해석시 특정한 경우에 있어서는 비탄성 거동을 고려해야 할 필요가 있을 것으로 보인다.

1. Introduction

The progress of the design techniques of cable supported bridges has rapidly been made over the last thirty years; this progress is largely developed due to the use of electronic computers, the development of box-girders with orthotropic plate deck, and the manufacturing of high strength wires that can be used for cables. Cable-stayed bridges, in which the deck is elastically supported at points along its length by inclined cable stays, are now entering a new era, reaching to medium and long span with a range of 500m to 1500m of center span.

The increase in span length combined with the trend to more shallow or slender stiffening girders in cable supported bridges has raised concern about their behaviors under both service and environmental dynamic loading, such as traffic, wind and especially earthquake loadings. Because of the fact that these long span, cable supported structures constitute complex

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structural with mainly nonlinearity that is essential to understand and realistically predict there structural response to these loadings. Accordingly, it is highly desirable in bridge engineering to develop and validate accurate procedures that can lead to a thorough understanding of the static, dynamic, seismic, and wind problems of cable supported bridges.

This paper is focused on the occasion of inelasticity of structural members under a severe live and a seismic load. The geometric nonlinearities are considered in the analysis, but the material nonlinearity is not considered. This paper just concentrates on the proportion of inelastic area of the whole structure, which can show that the material nonlinearity which is mainly neglected in the analysis procedure is how important factor in special case.

2. Sources of Nonlinearity in Cable Supported Bridges

2.1 Nonlinear behavior of cables

When a cable is suspended from its end and subjected to its own weight and externally applied axial tensile force, it sags into the shape of catenary. The nonlinear behavior of the individual cables in a cable supported bridge results from this sag phenomenon. The axial stiffness of the cable varies nonlinearly as a function of end displacements, since part of the end movement occurs due to material deformation and another part occurs due to change in sag. As the axial tension increases in the cable stays, this latter part, i. e. the change in the cable sag, becomes smaller and smaller, and the end movement occurs mainly due to material deformation. Accordingly, the apparent axial stiffness of the cable increases as its tensile stress increase.

A convenient method to account for this variation in the cable axial stiffness is to consider an equivalent straight chord member with an equivalent modulus of elasticity which combines both the effects of material and geometric deformations such that the axial stiffness of the equivalent chord member becomes equal to the apparent axial stiffness of the actual curved cable. This equivalent cable modulus of elasticity is given by

$$E_{eq} = \frac{E}{1 + \left[\frac{(wL)^2 AE}{12T^3} \right]} \tag{1}$$

in which E_{eq}=equivalent modulus; E = cable material effective modulus, L = horizontal projected length of the cable; w = weight per unit length of the cable; A = cross-sectional area of the cable; and T = cable tension.

2.2 Nonlinear behavior of bending members

When assuming small deformation in any structural system, the axial and flexural stiffness of bending members are usually considered to be uncoupled. However, when deformations are no longer small, there is an interaction between axial and flexural deformations in such members, under the combined effect of axial force and bending moment. The additional bending moment developed in a laterally deflected, or bent, member when subjected to a simultaneously applied axial force either increases or reduces the original bending moment in the member. The result of this axial force-bending deformation interaction is that the effective bending stiffness of the member decreases for a compressive axial force and increases for a tensile force. In a similar manner, the presence of bending moments will affect the axial stiffness of the member due to and apparent shortening of the member caused by the bending deformations. In most conventional linear structures, this interaction or coupling effect is negligible. However, due to the large deformations that may occur in a cable supported bridge, as a flexible structures, this interaction can be significant and should be considered in any nonlinear analysis.

2.3 Geometry change due to large displacement

In linear structural analysis, it is assumed that the joint displacements of the structure under the applied loads are negligible with respect to the original joint coordinates. Thus, the geometric changes in the structure can be ignored and the overall stiffness of the structure in the deformed shape can be assumed to equal the stiffness of the undeformed structure. However, in cable supported bridges, large displacements can occur under normal design loads, and accordingly, significant changes in the bridge geometry can occur. In such a case, the stiffness of the bridge in the deformed shape should be computed from the new geometry of the structures.

2.4 Material nonlinearity

A cable supported bridge is usually composed of three main structural parts (or bridge components), namely, cables, superstructure, and towers. These structural parts may be made of different materials. The material nonlinear analysis of a long span cable supported bridge depends on the nonlinear stress-strain behavior of individual materials for the structural components.

The assumption of elastic material is only available in a case that the member strain is limited to elastic range. It can be violated in an ultimate load condition or a certain special loading case such as an earthquake. This material nonlinearity should be considered to evaluate the overall safety of a structure.

Many researchers studied the cable supported structure as a structure which has a geometric nonlinearity. However, it is considered that the material nonlinearity is not negligible any more as the bridge has longer span lengths. Therefore, this paper will show the amount of inelastic range under the static equivalent earthquake load and the necessity of a consideration of inelastic behavior for the seismic analysis.

3. Models

The target bridge of this study is Seohae Grand Bridge that has the longest span length (470m) of cable stayed bridges in Korea. The deck of this bridge is a composite type of steel girders and concrete slabs. The analysis is performed with a planar model which considers the geometric nonlinearities. The model of Seohae Grand Bridge is depicted in Fig 1, and the boundary conditions are shown in Table 1.

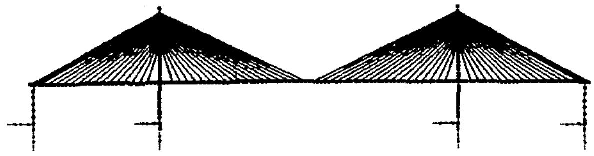

Figure 1 Seohae Grand Bridge plane analysis model

Table 1 Boundary conditions of the analysis model

Location	Type	Vertical Spring Stiffness
Between left pier and left end	Roller Hinge	-
Between left pylon and base	Fixed	-
Between left pylon and girder	Roller Hinge	2,000 tonf/m
Between right pylon and base	Fixed	-
Between right pylon and girder	Hinge	2,000 tonf/m
Between right pier and right end	Hinge Roller	-

4. Inelasticity of the Stiffening Girder

4.1 Applied Load

A load factor [λ] is introduced in this paper that is the multiplication of the dead load of the target bridge. Total dead load of Seohae Grand Bridge is 247,393.25 tonf; 28 tonf/m as a distributed load over the whole span. The factored load is obtained from the multiple of load factor(λ) and distributed dead load(28tonf/m). Table 2 shows the applied loads in this analysis.

The factored load is applied over the whole span after the final construction step which means that the bridge is already loaded by design dead loads and cable tension forces.

These loading conditions can show the ultimate behavior of the bridge which can be important to apply the limit state design method which is gets in the spotlight for the future design method.

Table 2 Factored load

Load factor (λ)	Applied uniform load
1.0	28 tonf/m
2.0	56 tonf/m
2.5	70 tonf/m

4.2 The estimation of elastic limit stress

Modulus of elasticity of girder is $2\times10^{7}\mathrm{tonf/m^2}$, and the elastic limit strain of normal steel is

0.0015. The elastic limit stress is obtained as 30,000tonf/m^2. If a stress of a part of girder exceeds this elastic limit stress, it is regarded that the element is in the inelastic zone. It means that the adopted modulus of elasticity (2×10^7tonf/m^2) is no more valid at this loading condition for this element. Therefore, the approximation of elasticity should be reconsidered. Figure 2 depicts the stress-strain curve for a normal steel.

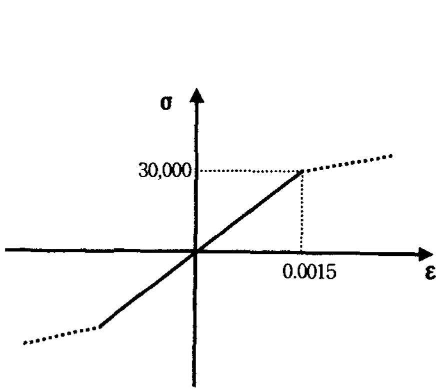

Figure 2 Stress-strain curve of steel

4.3 Analysis results

The load is the load factor (λ) times of the dead load of the bridge and the direction is a gravity direction, namely a vertical one. Figure 3 depicts stress diagrams of the stiffening girder according to the analysis result. The stresses in this graph are the maximum of the absolute value of four stress components which are top and bottom cover stresses of each y and z axis of the elements. The shaded areas of the fig 3 mean the inelastic stress ranges. The vertical axis of these graphs is absolute stresses whose unit is tonf/m^2. The horizontal axis represents the location of the stiffening girder. The most important thing is that the shaded areas show the inelastic range. As estimated above, the elastic limit is 30,000 tonf/m^2 and if the stress graph is the shade zone, the stress level of the element exceeds the elastic limit.

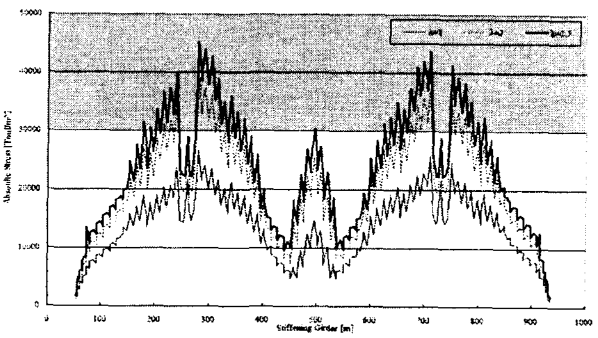

Figure 3 Maximum stress diagram of girder

If a stress level of a structure member is in the shaded area, the assumption of elasticity is violated and the result of the analysis of the member is not reliable any more. When the load factor(λ) is one, not any element of the structure is in the inelastic state. Therefore, it can be thought that the elasticity assumption is satisfied within this load level. However, as the load factor is increased, the more elements are located in the inelastic zone, and most of inelastic areas occurs near the pylons as a compression stress. When the imposed load factor in two and a half, many of the girder elements in the inelastic including tension members as well as compression.

The proportion of materials whose stress state is in the inelastic zone about the total girder length is introduced as the inelasticity index; IEI. Table 3 shows the sum of material length which might behave inelastically and the IEI index.

$$IEI = \frac{\text{sum of element length that might behave inelastically}}{\text{total element length}} \% \qquad (3)$$

Load factor (λ)	Loading the whole span IEI
1	N/A
2	5.57 %
2.5	16.72 %

Table 3 The distribution of inelastic zone of stiffening girder

5. Inelasticity of Pylons

5.1 The estimation of equivalent static force of earthquake

An equivalent static force is estimated to simulate the earthquake condition. The procedure to be adopted is referenced in Korean Highway Bridge Design Specification (here after KHBDS). The elastic seismic response coefficient, or lateral design force coefficient C_s, is a function of the seismic zone, the fundamental period of the bridge, and the site soil conditions. The value of the lateral design force coefficient for structures in which any T_m exceed 4.0 is given by KHBDS Formula (6.5.3) and the equivalent static seismic loading is given by KHBDS Expression (6.5.4) as follows.

$$C_s = \frac{3AS}{T^{4/3}} \qquad \begin{aligned} V &= P_e L \\ &= wLC_s \\ &= WC_s \end{aligned} \qquad P_e(x) = wC_s \qquad (4)$$

where A = acceleration coefficient, T = fundamental period of the bridge.

S = site coefficient or amplification factor for a specific soil profile

V = the total seismic shear force, P_e = the equivalent static seismic load

w = dead weight per unit length, W = total dead weight

L = total length of bridge

In the example Seohae Grand Bridge the fundamental period is 4 seconds as result from the dynamic analysis. The acceleration coefficient A is 0.154 as multiplication of seismic zone factor 0.11 for first Zone 1 and risk factor 1.4 for 1000 years mean return period according to the Korea Road Bridge Specification. The site coefficient is selected as type II, 1.2, which is a profile with stiff clay or the soil type overlying rock ares table deposits of sands, gravels, or stiff clays. In this site type the shear velocity at the 30m depth is from 360m/s to 760m/s. As followed by the equation (4) the lateral design force coefficient[C_s] is 0.0873 for the example bridge. The total dead weight of the superstructure and tributary substructure is 24739.25 tonf. Therefore the equivalent static seismic load [P_e] is 2.444 tonf/m and the total seismic shear force [V] is 2126.28 tonf

5.2 Connections between girder and pylon

The connection type of girder and pylon seems of great importance because it has an great influence on the behavior of a pylon. Consequently, three type of constraints are compared in this paper.

Table 4 the connection types of pylon and girder for an analysis of pylon inelasticity

	Between left pylon and girder			Between right pylon and girder		
	longitudinal	vertical	rotation	longitudinal	vertical	rotation
Case 1	*Free*	2,000 tonf/m	Free	*Free*	2,000 tonf/m	Free
Case 2	*Free*	2,000 tonf/m	Free	*Fix*	2,000 tonf/m	Free
Case 3	*Fix*	2,000 tonf/m	Free	*Fix*	2,000 tonf/m	Free

5.3 The estimation of elastic limit stress

Pylons are made of concrete whose modulus of elasticity is 2.8×10^6 tonf/m^2. The elastic limit strain of concrete is 0.0005 for compression and 0.000086 for tension. Hence, the elastic limit stress for concrete is 1,400 tonf/m^2 for compression and 240 tonf/m^2 for tension.

5.4 Analysis results

Figure 5 and 6 show the analysis result. The shaded areas represent the inelastic range as same as above. Inelasticity happens only in Case 1 according to these results. It means that the inelastic behavior does not appear in a pylon unless the longitudinal constraints between girder and pylons are free to move. In addition, inelastic part of a pylon is not the base part but the upper part of the connection of a girder. It seems that the behavior of a pylon is different from the other bridge pier, because the inelastic hinge exits not at the base like as a usual pier. However, it is not certain if the real earthquake excitation to the base can result the same response.

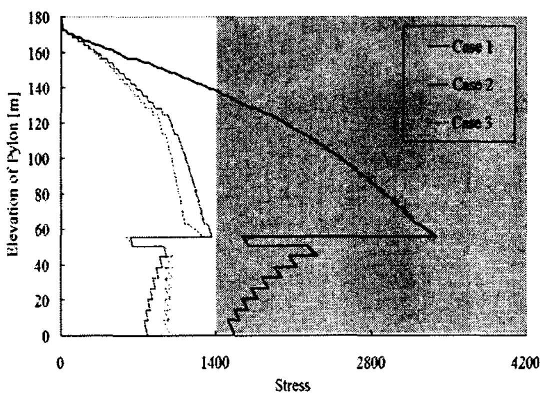

Figure 4 Stress diagram of left pylon

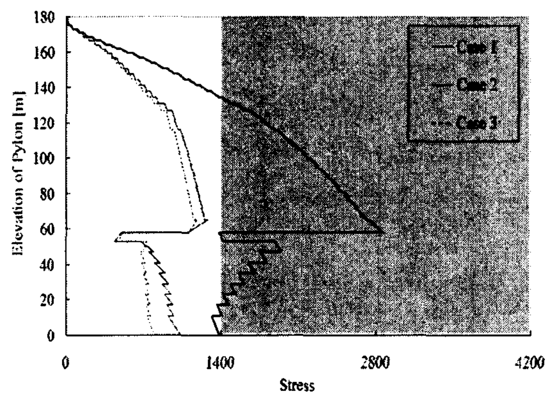

Figure 5 Stress diagram of right pylon

The proportion of inelastic element to the whole pylon is described at Table 5. Inelasticity index (IEI) is also introduced same as above.

Table 5 The distribution of inelastic zone of pylons

	Left Pylon IEI	Right Pylon IEI
Case 1	78.98 %	64.89 %
Case 2, 3	N/A	N/A

7. Conclusions

The stresses of the stiffening girder exceeds the elastic limit when the factor load whose factor is bigger or equal than two(λ=2). It is thought that the inelasticity of stiffening girders should be considered to apply the limit state design method for a future design. In addition, none of the pylon elements does not go over the elastic limit.

The equivalent static force to the longitudinal direction of bridge is adopted to model the longitudinal earthquake ground motion. Also, the 0.3 times of this equivalent force is applied to the vertical direction to simulate the vertical ground motion. In accordance with the analysis result, pylons behave quite differently according to the constraint condition between girder and pylon. The stresses of pylon elements exceed the elastic limit of concrete material when the longitudinal motion is free to move(Case 1). It means that the less constraints can result in higher possibility to behave inelastically. Also, the constraint condition is a very important factor to guarantee the safety of bridge under the earthquake loading condition.

Seismic analysis of cable supported bridge is performed with consideration of geometrical nonlinearity until now. However, it is proved that some of the elements in a cable supported bridge can behave inelastically in this paper. Hence, analysis of seismic behavior of a cable supported bridge should consider not only geometrical nonlinearity but also the material nonlinearity. This characteristic might be more affective at the longer span cable supported bridge than one of modern times.

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