Dynamic Response of Steel Tower of Cable-Stayed Bridge with Construction Imperfection under Great Earthquake Ground Motion

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1. Introduction

The recent advancement in manufacturing to produce high strength steels have led to the use of thin plates in structural elements fabrication by hot rolling and welding processes, thus have resulted in a high strength to weight ratio. For significant savings, a great number of long span steel bridge box girders, towers and piers with hollow rectangular box sections are widely used in Japanese highways. Seismic design of such steel bridge is very important for the urban transportation network, which became much clearer after the 1995 Hyogoken-Nanbu Earthquake. As is well known, a lot of steel bridges collapsed during this earthquake[1].

The ability of such structures to survive severe earthquakes depends on both the strength and ductility of the structures. The strength and ductility of a thin-walled steel member is particularly sensitive to initial imperfections including geometric imperfection and residual stress. The initial imperfection of these structures that have not been subjected to damage usually results from the fabrication process. Hence, the initial imperfection can significantly affect its dynamic behavior of thin-walled steel structures. Some studies[2] have been carried out on welded box columns made of rolled steel plates, it was reported that the detrimental effects of geometric imperfections and welding residual stresses, will lead to an appreciable reduction in load carrying capacity. However, a study of the effect of initial imperfection on the dynamic behavior of thin-walled structures is not available in literature.

The purpose of the present study is to investigate the seismic response of steel tower of cable-stayed bridge with hot rolled hollow rectangular cross section and welded stiffeners under initial construction imperfection using the finite element method. A theoretical study is carried out to investigate the effects of initial geometric imperfection of steel tower fundamental mode of vibration pattern with different amplitude and the effects of longitudinal residual stresses on seismic response of steel tower. As a result, a great deal of insight has been obtained on the dynamic response of steel tower. The results indicate that both initial geometric imperfection the welding induced residual stresses significantly affects on the tower seismic response. The residual stress effects are due to decreasing plastic deformation capability, consequently promoting brittle behavior.

2. Nonlinear Equilibrium Equation Solution Technique

Based on the total incremental equilibrium equations, large displacement three-dimensional beam-column element formulation is carried out, where the tangent stiffness matrix and nodal point force vectors considering both geometrical and material nonlinearities can be determined. The initial state of stress (residual stresses) effects on both tangent stiffness and force vector is considered. In this study, the implicit Newmark step-by-step integration method is used to directly integrate the equation of motion, since it has been experienced that the Newmark method is the most suitable for nonlinear analysis; it has the lowest period elongation and has no amplitude decay. In addition, the stability concern is not a problem with the variable ratio of time increment to natural period. The equation of motion is solved for the incremental displacement using the Newton Raphson iteration method where the stiffness matrix is updated at each increment to consider the geometrical and material nonlinearities and to speed the convergence rate. As the incremental displacement is determined, the response acceleration and velocity components of the tower can be determined. In addition, attenuation of the structure adopted the viscous damping of mass proportional type with damping coefficient to the first fundamental natural vibration mode is 5 % as standard.

3. Outline of Finite Element Analysis

3.1 Finite element model

The steel tower of Iwamizawa cable-stayed bridge located in Hokkaido, Japan is considered in this study. The steel tower is taken out of the cable-stayed bridge and modeled as three-dimensional frame structure characterized by a fiber flexural element. The model invoked large displacement using a Total Lagrangian formulation; the Hermitian cubic interpolation is used to describe bending deformation and linear interpolation to specify axial and torsional displacements. The tangent stiffness considers the material nonlinearities through tri-linear stress strain relation for the beam column element. The material behavior is modeled by a tri-linear elastic-plastic constitutive model incorporating a uniaxial yield surface criteria and kinematic strain hardening flow rule and considering initial state of stress, as shown in Fig. 1, the yield stress and the modulus of elasticity are equal to 353 Mpa and 200 GPa, respectively; the strain hardening in the plastic region is 0.01. The effect of residual stresses on the yield level of the material stress strain relationship is neglected.

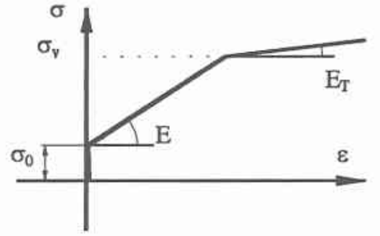
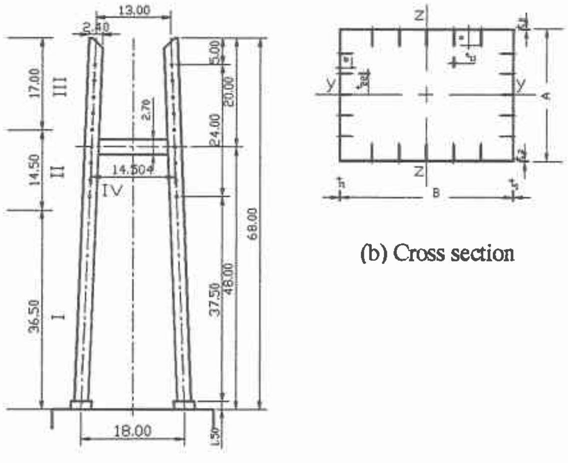


Fig. 1 Tri-linear elastic-plastic constitutive model

The fiber element distributed plasticity formulation is required to provide sufficient degrees of freedom to explicitly model plasticity spread effects. Inelasticity of the flexure element is accounted for by the division of the cross section into a number of fibers zones with uniaxial plasticity defining the normal stress-strain relationship for each zone, the element stress resultants are determined by integration of the fiber zone stresses over the cross section of the element. By tracking the center of the yield region, the evolution of the yield surface is monitored, and a stress update algorithm is implemented to allow accurate integration of the stress-strain constitutive law for strain increments, including full load reversals. To ensure path dependence of the solution, the implementation of the plasticity model for the implicit Newton-Raphson equilibrium iterations employs a stress integration whereby the element stresses are updated from the last fully converged equilibrium state. The nonlinear behavior of cable elements is idealized by using the equivalent modulus approach, in this approach each cable is replaced by a truss element with equivalent tangential modulus of elasticity used to take account of the sag effect.



(a) Tower geometry

(b) Cross section

Fig. 2 Steel tower of Iwamizawa cable-stayed bridge

This cable-stayed bridge has nine cables in both sides of the tower. The dead load of the stiffening girder is considered to be equivalent to the vertical component of the pretension force of cables and acted vertically at the joint of cables. The inertia forces acting on the tower from the stiffening girders is neglected. For the numerical analysis, the geometry and the structural properties of the steel tower is shown in Fig. 2, where this tower has rectangular hollow steel section with internal stiffeners, which has various dimensions along the tower height and its horizontal beam as shown in Table 1.

Table 1 Cross section dimension of different tower region

C. S.		Tower parts of the same cross section			
Dim. (cm)		I	II	III	IV
Outer dim.	A	240	240	240	270
	B	350	350	350	350
	t_1	2.20	2.20	2.20	2.20
	t_2	3.20	3.20	2.80	2.60
Stiffener dim.	a	25.0	22.0	20.0	31.0
	b	22.0	20.0	20.0	22.0
	t_{11}	3.60	3.20	2.80	3.50
	t_{22}	3.00	2.80	2.20	2.40

3.2 Selected ground motions

In the dynamic response analysis, the ground motion recorded during the Hanshin/Awaji earthquake 1995 with the largest intensity of ground acceleration is used as an input to assure the bridges seismic safety. The acceleration time history recoded at JR Takatori Station is suggested for analysis of the steel tower of Iwamizawa cable-stayed bridge at Type II of soil condition, since it was considered to be capable of exciting of this tower into its nonlinear range. The earthquake force of E-W wave was put into the bridge axis direction, and N-S wave to the right angle to the bridge axis.

3.3 Treatments of initial geometric imperfections

Initial imperfections in structures that have not been subjected to damage usually result from the fabrication process and strength of a thin-walled steel member is particularly sensitive to imperfections in the shape of its eigenmodes. The amplitude of imperfections in the lowest eigenmodes of vibration is often sufficient to characterize the influential imperfections. Initial geometric imperfection is applied by modifying the nodal coordinates using a field created by scaling the appropriate vibration eigenvector obtained from an elastic natural vibration analysis of tower model.

3.4 Treatments of residual stresses

The presence of longitudinal residual stresses in stiffened rectangular hollow cross section is mainly attributable to the welding of stiffening members in addition to hot rolling of the hollow cross section itself. The residual stresses in the weld, stiffener and hollow section material in the vicinity of the weld are close to the yield as a result of the contraction of the welds. The magnitude and distribution of the residual stress are governed by welding parameters such as heat input and cooling rate; both are governed by the welding procedure adopted for fabrication. To model the distribution of membrane residual stresses in hot-rolled rectangular hollow section with internal welded stiffeners and the spread of the plasticity, the integration through the division of fiber model of the cross section is considered to be sufficient. A typical residual stress pattern[3] used for analysis is shown in Fig. 3. The tension existing at the hollow section to stiffener junction is balanced by the residual compression. It has been reported that the magnitude of the compressive residual stresses increases as the component plates of hollow section slenderness becomes smaller, values of up to 75% of the yield strength have been measured[4]. An idealized pattern of residual stresses due to welding, as in Fig. 3., is used in the computational model. The maximum residual stress is assumed to be σ_y in tension side and σ_{rc} in compression side. They satisfy the moment equilibrium conditions

$$\int_A y\,\sigma_0\,dA = 0 \quad \text{and} \quad \int_A z\,\sigma_0\,dA = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and the axial force self-equilibrium condition in the whole cross section was used to determine the longitudinal residual stress distribution through the cross section.

$$\int_A \sigma_0\,dA = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

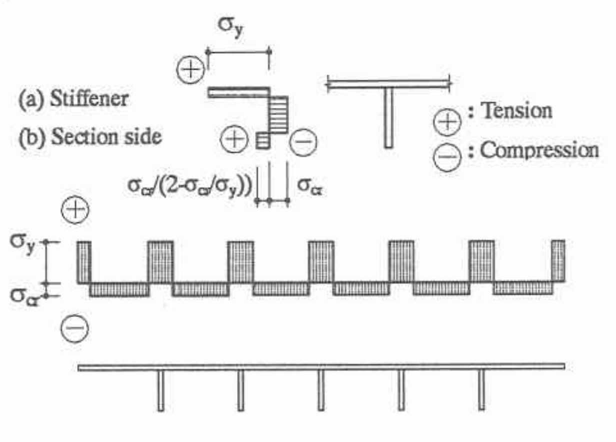


Fig. 3 Residual stresses in each side of hollow rectangular section of steel tower

4. Numerical Results and Discussion

4.1 Geometric imperfection effects

The dynamic response of the steel tower with consideration of initial geometric imperfection is studied for different imperfection amplitude of range from 0.01 up 1% of tower height. The initial geometric imperfection is taken to be the first fundamental mode of vibration pattern. The effect of the magnitude of initial geometric on the extreme values of in-plane displacement of right tower top and in-plane moment at right tower base is presented in Figs. 4 and 5, respectively. It can be concluded that the initial geometric imperfection has significantly effect on the tower seismic response, where the extreme value of in-displacement in the same direction of vibration mode nonlinearly increases as imperfection amplitude increase under the same loading and has value 5% of that of perfect tower at maximum imperfection 1% of tower height, as shown in Fig. 4. Also the extreme value of in-plane bending at tower base, which resisting the vibration mode, slightly increase up to1.5% of that of perfect tower.

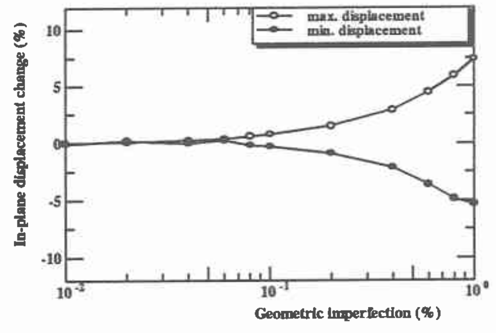


Fig. 4 In-plane displacement &imperfection amplitude at right tower top

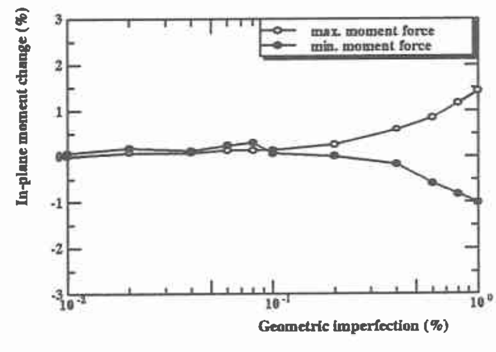


Fig. 5 Moment & imperfection amplitude at right tower base

4.2 Residual stress effects

The formation of residual stresses is inevitable in any welding and hot rolled operation. When superimposed on the externally applied stress fields, the residual stresses can result in significant differences in the performance of welded structures. The effects of residual stresses on the dynamic response of the steel tower is studied by varying the compressive residual stress levels, the compressive residual stresses of 0.0, 0.15, 0.30, 0.45, 0.60 of the yield stress were used to represent the condition of stress relieved, lightly welded to heavily welded stiffener to the cross section. Fig. 6 presents the in-plane displacement change of the tower top with various residual stresses relative to that without residual stresses. It can be observed that residual stresses have a more pronounced effect on the tower dynamic response, which will yield at a lower load stress due to the presence of compressive residual stresses, how the applied membrane stresses used to reflect the removal of residual stresses from the stress picture. Fig. 7 displays brittle behavior due to decreasing of plastic deformation capability, hence the tower top displacement decrease as the residual stress level increase. In addition, the stiffness of tower decreases with an increase in compressive residual stresses. Moreover, the tower strength (moment and shear at tower base) is reduced by the presence of residual stresses and this decreases with increase of compressive residual stress level, as shown in Figs. 8 and 9. From the total, damping and strain energy of the whole tower study, it is appeared the energy decrease as the compression residual stress increases. This reduction can be attributed to decreasing of tower plastic deformation capability and gradual decreasing of tower stiffness due to presence of compression residual stresses, as shown in Fig. 10.

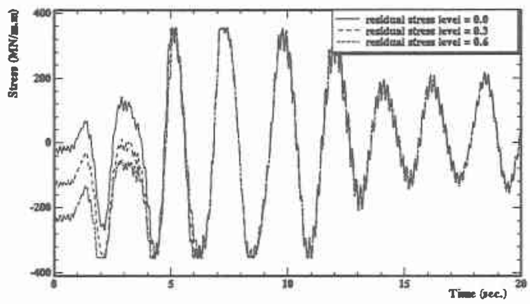


Fig. 6 Stress time history of compression residual stress fiber at tower base

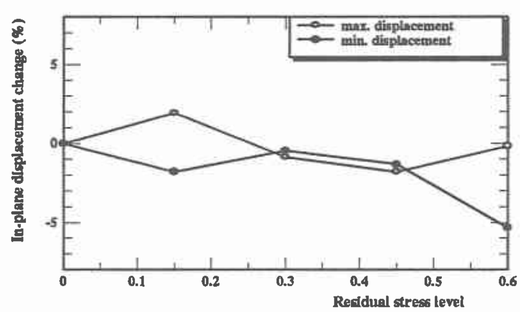


Fig. 7 In-plane displacement &residual stress level at right tower top

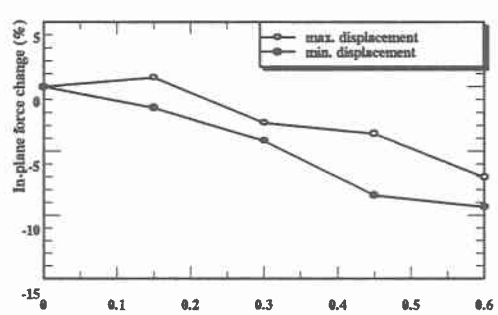


Fig. 8 In-plane force &residual stress level at right tower base

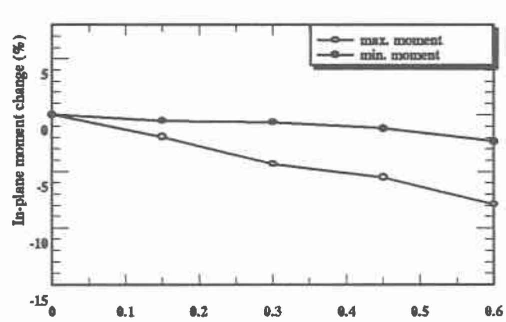


Fig 9 In-plane moment &residual stress level at right tower base

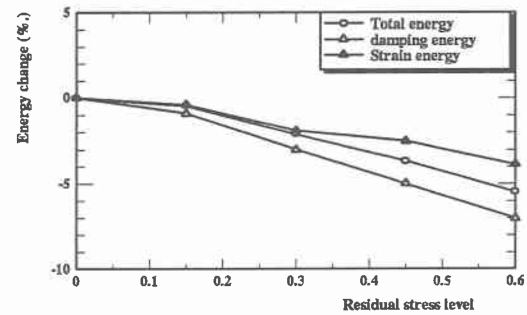


Fig. 10 Total, damping and strain energies of the whole tower & residual stress level relationship

5. Concluding Remarks

Theoretical studies of the steel tower of Iwamizawa cable-stayed bridge have been conducted to investigate the dynamic behavior considering construction imperfection, including geometric and residual stresses imperfection is studied. Using nonlinear finite element dynamic analysis demonstrates how geometric imperfection magnitude and residual stresses all influence tower seismic response. From this study, the following conclusions can be drawn as follow:

(1) Initial geometric imperfection of tower fundamental mode of vibration pattern is found to have significantly effects on the tower seismic response, which can be considered due to fabrication error.

(2) The longitudinal normal stress distribution is affected by the longitudinal residual stress; as a result, the tower moment capacities decease as the residual stress level increase.

(3) The tower stiffness decreases with an increase in compressive residual stresses level. Moreover, the extreme load carrying capacity (moment and shear at tower base) is reduced by the presence of residual stresses and this decreases with the increase of compressive residual stresses.

(4) The residual stress has detrimental effects on tower structural performance, which can be characterized by decreasing plastic deformation capability, consequently promoting brittle behavior.

References

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