PERFORMANCE BASED DESIGN OF LONG-SPAN CABLE STAYED BRIDGE TOWERS

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ABSTRACT:

Over the last forty years, there is a considerable increase in design and construction of cable-stayed bridges. The seismic requirements of the standard bridge specifications do not typically apply to the analysis and design of long-span cable stayed bridges. The focus of this study can be divided into two phases as (1) to investigate the common characteristics of existing long-span cable stayed bridges and (2) to determine the seismic performance of a typical cable-stayed bridge. In this research, bridge tower damage levels are tried to be predicted using a displacement based design approach. Research program is still in progress.

INTRODUCTION

A number of cable-stayed bridges are listed at different references (Podolony and Goodyear, 2006, Delegation 2001, and Tang 1999). Cable-stayed bridges usually have a main span ranging from 180 to 1018 meters. A typical ratio of the tower height to main span length is around 1/4 to 1/5. If the main span is less than 180 meters typically an extradosed bridge is preffered. The ratio of tower height to main span is around 1/12 to 1/13. In all cases, tower height is measured from the superstructure level to the peak point of the tower. Cable-stayed towers are also called pylons.

Most of the cable-stayed bridges have a tower either in the shape of an A-frame or a H-frame with a cross beam at top or a diamond shape frame. The cable-stayed bridge towers are constructed using hollow high strength reinforced concrete columns. Hollow concrete columns can maximize the structural efficiency in terms of strength/mass and stiffness/mass ratios (Mo and Nien 2002; Lignola et. al. 2007). The cross beams can be post-tensioned to avoid brittle failure of towers during an extreme event.

Damage levels after an earthquake are typically determined through by evaluating the concrete and steel strains within the member. The evaluation of strains is very complex since the computations are highly non-linear. Furthermore, investigation of literature is indicated that there is no consensus on defining damage levels by certain strain values. The focus of this paper is given to evaluate target damage levels in terms of a ratio of displacement capacity to displacement demand based on recent researches rather than strain values.

Displacement-based seismic design of structures and seismic performance of bridges are studied by Priestley et al (2007), Nielsen and DesRoches (2007), Lu et al (2005), Lehman et. al. (2004), Mo and Nien (2002), Park, S. W. et al (2001), Whittaker et al (1998), Williams and Sexsmith (1995), Moehle

(1992), and Floren and Mohammadi (2001). The performance based design approaches increases the cost of design and construction while minimizing the cost of repair after the expected earthquake (Floren and Mohammadi, 2001). The total damage cost after the 1994 Northridge earthquake, USA, was reported to be around 25 billion US dollars. Similar damage was encountered at the 1999 Izmıt earthquake, Turkey.

Moehle (1992) summarized the displacement and ductility based design approaches. The ductility based design approach is suggested to be used at structures with uniform inelastic response characteristics. In this case, local demand-capacity ratios provide a reasonable accurate picture of ductility distribution and magnitude. If the structure does not have uniformity in inelastic response, displacement-based design approach is recommended to be used in design.

Williams and Sexsmith (1995) summarized different damage indices for concrete structure. Some indices take into account the cumulative damages as well. Damage indices are named as deformation-based cumulative indices, energy based cumulative indices and combined indices. The damage levels are defined in five levels as: 1) none, 2) minor, 3) moderate, 4) severe and 5) collapse.

In 1996, Japanese seismic design specifications started to utilize the performance based design (Floren and Mohammadi, 2001). At 1995 Hyogo-Ken Nanbu earthquake, Japan, most of the damaged structures were designed based on the 1964 specifications using force design. In the new specification, lateral load carrying capacity and ductility of the columns were typically increased by having additional longitudinal rebars and reinforcement ties. Consideration was given to seismic isolation devices, dampers and unseating prevention devices.

Three damage levels are presented in ATC-32 (1996) as follows:

Minimal damage: Damage is limited to minor flexural cracking, and minor inelastic response is permitted to develop at structural elements.

Repairable damage: Concrete cracking, reinforcement yielding and minor spalling is allowed to avoid closure of the structure during minor repair stage.

Significant damage: Similar to repairable damage except during repair the structure needs to be closed for the major repair work.

In direct displacement based design approach proposed by Priestley et al (2007) three structural limit states were described similar to the ATC-32 (1996) document. These limits were defined by serviceability, damage-control and survival limit state. For building type structures, the excessive damage is likely to happen when the drift ratios reach to 0.025. However, the drift limits for damage-control state for bridges believed to range from 0.03 to 0.045 drift ratio.

The focus of this research is given to tall hollow high strength frames of cable stayed bridge towers to limit the damage levels at the minimum to repairable level during the expected design earthquake with no closures on service. There is very limited experimental data to use in assessment of the possible damage levels and drift ratios. This research still requires additional work especially on experimental side.

EXPERIMENTAL EVALUATION

Based on the damage levels and limit states described at ATC-32(1996) and Priestley et al (2007), the minimal damage is named as the mid-state between longitudinal reinforcement yielding and initial concrete cover spalling, which represents a minor inelastic state in the hollow concrete tests of Mo and Nien (2002). The relationship between the displacement capacity to demand ratio, and the drift ratio is plotted for the tests of Mo and Nien (2002) as shown in Figure 1. The test data of Lehman et al (2004) for circular bridge columns are plotted in also Figure 1.

The limited evaluation of experimental data indicated that the well-confined circular columns perform better than the hollow concrete columns in terms of drift. Based on the Mo and Nien (2002) tests, the hollow concrete will reach to the minimal damage level when displacement capacity to demand ratio reaches to 2.5. This ratio corresponds to a drift of 0.02 or a 2% drift. The limit of repairable damage is corresponding to a 3% drift. Hollow concrete columns with different sizes, reinforcement ratios and ties need to be investigated to draw solid conclusions on how to define the minimal damage level state in terms of displacement capacity to demand ratio and drift percentages.

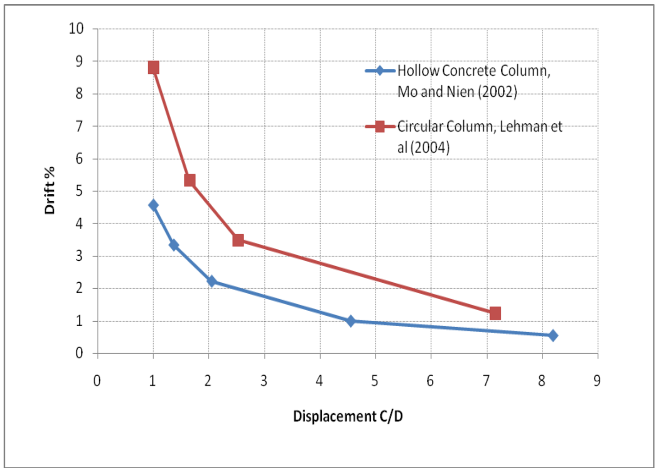

Figure 1. Experimental Evaluation of Displacement Capacity to Demand Ratio and Drift

Pinto et. al (2003) tested scaled models of seismically deficient hollow concrete bridge columns built in Austria in 1975. The deficiencies are: lap splices provided with in the plastic hinge zone; low percentage of longitudinal and transverse reinforcement; and poor reinforcement detailing. The short and the long pier drifted up to 1.5% and 1.6% before failure lower than the 3% drift limit for commonly accepted ductile structures. The prediction of the plastic hinge length using the empirical formulae was in good agreement with the test results for long pier. The empirical formulae for predicting the plastic hinge length is:

$$L_p = 0.08l + 0.022d_bf_y \qquad (1)$$

Where Lp is the plastic hinge length; l is the length distance measured between maximum moment location and contraflexure location; d_b is the diameter of the longitudinal bar and f_y is the yield strength of the rebars.

Lignola et. al. (2007) studied the retrofitting of hollow concrete bridge columns with CFRP. The ductility for standard circular hollow columns can be achieved by having low levels of axial load, moderate longitudinal steel percentage, or a relatively thick wall (wall thickness not less than 15% of the overall thickness of the section). FRP-confined columns have about 15% strength increase compared to a similar unconfined one. The ductility of the confined columns can reach up to 8.

Hollow bridge columns can be prefabricated to reduce on-site construction time. The results of past experiments indicate that the joint opening prior to plastic hinging result in a poor performance during an seismic event compared to a monolithic one. Ou et al (2007) demonstrated that if the precast segments have unbounded post-tensioned tendons, the drift limits at joint opening will be more than 3% and failure may develop around 4% to 8% drift for the specimens they had tested.

Yeh et. al (2002) have tested a prototype and a scaled down hollow concrete column under lateral loads. They have observed that the model column had less ductility and drift capacity compared to the prototype test due to low-cycle fatigue properties of small bars used in the model specimen and an unclear yield plateau of the same small size bars. The maximum drift of prototype specimen was more than 6% but the maximum drift of the scaled-down specimen was around 5%.

ANALYSIS GUIDELINE

Modern codes of practice oriented to new design do not fully cover specific problems related to hollow sections (Lignola et. al. 2007). The tests and analytical studies conducted by different researches indicated that for hollow high strength concrete columns indicated that the damage levels can be interpreted in terms of ratio of displacement capacity to displacement demands as presented in Table 1. Damage levels are mainly function of concrete and steel strains within the member.

The cable-stayed bridge design is allowed to have some minor damage at the base location of the tower for an earthquake with 475 year return period and repairable damage at higher level earthquakes or at near-fault. The following design analysis guideline is used

- Step 1: Perform a response spectra analysis and determine the elastic displacement demand.
- Step 2: Design the tower using forces determined from the response spectra analysis and reduce the moments by

selected response modification factor. Indirect interpretation of AASHTO-LRFD (2007) indicates that the response modification factor for large wall type piers is around 1.5.

- Step 3: Perform a pushover analysis of the tower to determine the displacement capacity of the tower.

- Step 4: Determine the ratio of displacement capacity to displacement demand to evaluate the damage level.

The target damage levels used in this study are mostly based on the literature review of a range of concrete sections including the hollow sections. In this study, minimal damage levels will be limited at displacement capacity to demand ratio (C/D) of 2.5 and repairable damage level will be limited at C/D of 1.5. It shall be noted that these limits need to be verified by analysis of more data. The investigated tests are performed for H/D ratios of around 4. However, towers of a cable stay bridge are usually slender with H/D ratios around 10. The H is the height of the tower and D can be taken as the width of the section.

It may be beneficial to verify the results of this guideline with addition of a time-history analysis step. This part of the research is still in progress.

CASE STUDY

The example cable-stayed bridge used in this research is an alternative design to a new highway bridge design over the Sakarya River in Turkey. The new highway bridge design utilized composite steel girders as the superstructure supported over piers spaced by 50 meters. As an alternative, a cable-stayed bridge with a 300 meter main span carrying the same load of traffic is considered in analysis. The cables are spaced at 14 meters at each edge of the superstructure not to have big jumps at the moment diagrams of the superstructure. At the tower location, a support system simulating a fake cable is provided. Each back span is about 150 meters long and the tower height from the superstructure level is about 75 meters. The superstructure is selected to be a box steel section, which will be expensive compared to a steel composite one.

The tower is selected to be an A-frame as shown in Figure 2. In all cases, the base of the tower is selected to have 6 meter width, 12 meter long hollow reinforced concrete section with 0.8 meter concrete thickness. The cross-sectional length of the tower in longitudinal direction decreases to 6 meters gradually by the height of the tower. The thickness of the concrete is 0.6 meters after the superstructure level. The 28-day concrete strength is selected to be 50 MPa for the tower.

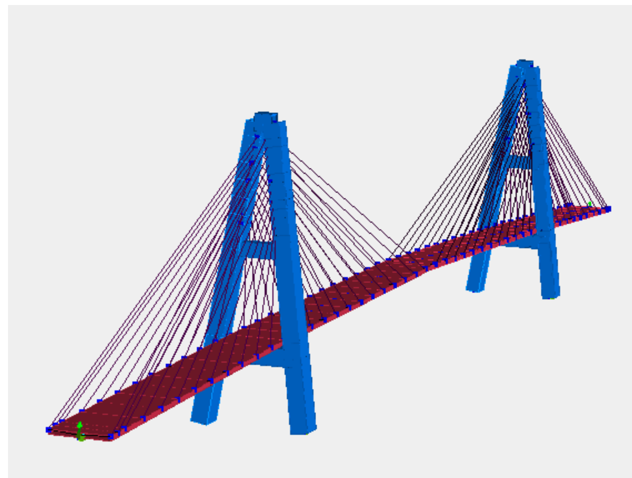

Figure 2. A-Frame

The response spectra curve for the region was supplied by the Yuksel Project International, Turkey as shown in Figure 3. The design spectra curve includes near fault effects of the North Anatolian Fault Line.

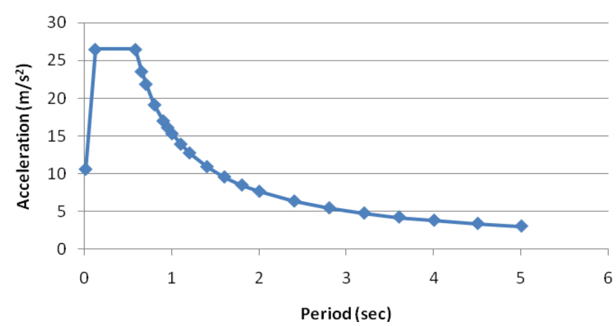

Figure 3. Design Response Spectrum Curve

LARSA 4D program version 7.01.57 is used in this research to model the bridge. The superstructure is modeled as a fishbone where cables are attached to the edge of the superstructure in the model. At foundation level, equivalent springs simulating the soil-structure interaction are provided. Anchor piers are also simulated with equivalent springs. The structural model is shown in Figure 4.

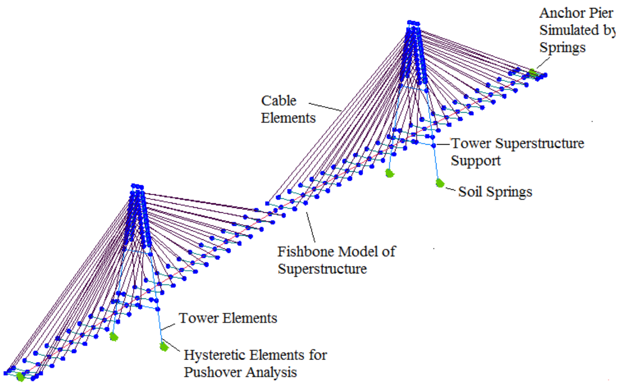

Figure 4. Structural Model

The towers are designed using conventional design methods per AASHTO (2007). A response modification factor of 1.5 is used for the bending effects induced by the seismic activity. The dead load reaction per leg of an A-Frame is around 80,000 kN. The design longitudinal moment is determined to be 2.44e6 kN-m about transverse axis and the design transverse moment is determined to be 3.89e5 kN-m about longitudinal axis. The first fundamental mode is developed in longitudinal direction of the bridge as shown in Figure 5.

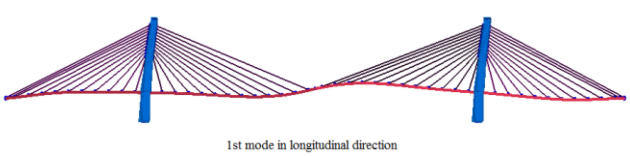

Figure 5. Fundamental Mode of the Bridge

The critical direction of the bridge tower is the longitudinal direction in terms of stability. The A-frame is providing a better stability in transverse direction. The elastic displacements and drift ratios are presented in Table 1. The effective moment of inertia is used in the computations. The effective moment of inertia of the towers is determined from the moment-curvature analysis. It shall be noted that the total height of the tower from the foundation level is 106 meters.

Table 1. Elastic Seismic Displacements and Drift Ratios

Top of Tower Longitudinal Displacement (mm)	Top of Tower Transverse Displacement (mm)	Longitudinal Drift (%)	Transverse Drift (%)
2220	215	2.07	0.20

A pushover analysis in longitudinal direction is performed with different amounts of longitudinal reinforcement in the hollow reinforced concrete tower. The damage levels at hollow concrete can be improved by increasing the amount of longitudinal reinforcement.

Table 2. Displacement Capacity to Demand Ratios and Drift %

Longitudinal Reinforcement Ratio	Top of Tower Longitudinal Displacement Capacity to Demand Ratio	Longitudinal Drift Capacity (%)
0.03	1.36	2.85
0.04	1.62	3.43

The displacement capacity to demand ratio is above the 1.5 threshold value, which is the low limit for the repairable damage. The next option to improve the seismic performance may be to have larger sections.

The near fault location of the bridge adversely affected the seismic performance of the bridge. The peak ground acceleration of the response spectra curve was corresponding to the 0.96g, which is typically more than two times of a high seismic risk zone peak ground acceleration away from a fault line.

CONCLUSIONS

The following conclusions can be drawn from this research in progress:

- Tall hollow high strength column tests are needed to refine target damage levels in terms of concrete strains, displacement capacity to demand ratios or drift ratios.
- The design can improve by increasing the amount of steel or enlarging the sections, which will increase the construction cost. The aim is to minimize the cost of damage and keep the bridge in service after the expected earthquake.

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