Transverse seismic failure mechanism and ductility of reinforced concrete pylon for long span cable-stayed bridges: Model test and numerical analysis

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ABSTRACT

Although often designed to behave elastically, seismic damage to reinforced concrete (RC) pylons of cable-stayed bridges have been witnessed in history such as the 1999 Chi-chi earthquake. This paper aims to assess the transverse seismic failure mechanism and ductile properties of typical inverted Y-shape RC pylons for long span cable-stayed bridges using quasi-static model tests and numerical analyses. To facilitate the limited laboratorial loading system, a simplified displacement-controlled two-node load-pattern, one at the bifurcation-node and the other at the crossbeam, is first proposed using numerical analyses. It is found the ratio of displacements at the two loading nodes is correlated generally well with the ground motion parameter, bracketed duration. A displacement ratio of 5.0 is then adopted in the test. Test results indicate a flexural damage mode with considerable ductility; plastic hinges were detected first at bottom of the upper column (i.e., above the crossbeam), then at bottom and top of the lower column, successively; multi-level displacement ductility factors are proposed to associate with numbers of plastic hinges formed in the pylon. Moreover, an experimentally validated numerical model is adopted to study the impact of loading displacement ratios on the failure mechanism and ductility. It is found that the loading displacement ratios may significantly affect them. Smaller displacement ratios tend to transfer the location of first plastic hinge from the bottom of the upper column to that of the lower column.

1. Introduction

During the past few decades, hundreds of cable-stayed bridges have been constructed in the world, mostly in China. Among them, there are over twenty long span cable-stayed bridges with main spans beyond 600 m, serving as pivotal joints in transportation networks. Due to their high requirements for post-earthquake serviceability, seismic design codes often give preference to elastic design strategies for pylons of cable-stayed bridges under design-level earthquakes [1–3], and nearly elastic design strategies even under occasionally strong events (e.g., return periods over 2,000 years) [1]. To this end, uneconomical high demands on reinforcement ratios for columns and crossbeams are imperative for the design of reinforced concrete (RC) pylons. Nevertheless, unexpected large earthquakes may cause damage to such bridges. A documented evidence is the Chi-ju cable-stayed bridge in Taiwan during 1999 Chi-chi Earthquake [4], where the pylon above the deck experienced severe concrete spalling in the transverse direction and formed an approximately 2 m-length plastic region [5]. Therefore, several later constructed long span bridges (e.g., Stonecutters in China [6], Rion-Antirion in Greece [7] and Third Tacoma Narrows in USA [8]) allowed limited damage in pylons to account for occasionally strong earthquakes. On the other hand, seismic isolation devices and viscous fluid dampers (VFDs) have been widely adopted in the longitudinal direction to reduce deck-pylon relative displacements [9–14], which in turn protects the pylons. In the transverse direction, although VFDs and steel dampers have been adopted or experimentally/numerically studied for a few bridges [15–19], the rigid connection between pylon and deck is still the most practical solution to as-built cable-stayed bridges [20], because it can provide sufficient stiffness for resistance of wind and traffic loads. With this configuration, pylons may behave as elastoplastic components under unexpected strong earthquakes, which stimulates the motivation of this study.

Numerical studies on modelling of RC pylons often use elastic elements [21–24]. Few studies focused on their inelastic behavior. Endo et al. [25] studied the seismic yielding process of a H-shape steel pylon with multi-struts (crossbeams) for a long span suspension bridge. Pushover analysis was applied transversally to the fiber/shell-element-based pylon model. Numerical results revealed that the plastic region...
first developed at the lowest strut, and then gradually shifted to upper struts, finally extended to pylon columns. Camara and Astiz [26] proposed a coupled nonlinear static pushover analysis method to estimate elastic and inelastic responses of inverted Y-shape and Diamond-shape pylons subjected to multi-directional excitations. Pang et al. [27] and Zhong et al. [28] assessed the seismic fragility of cable-stayed bridges with diamond-shape pylons modeled using fiber elements. Although sophisticated finite element (FE) models are popular for assessments of structural inelasticity, their reliabilities are highly dependent on related experimental validations [29].

Shake tables have been used extensively because they can provide conditions representing real seismic excitations. Early attempts [30–32] modeled cable-stayed bridge specimens using organic glass in small scales (i.e., 1/200 ~ 1/100). Although these studies can generally capture the elastic behavior of such bridges, they are not capable of revealing the inelastic behavior of the bridges, especially for those with RC pylons, because the inelastic behavior of rebar and concrete cannot be accurately represented by organic glass or other substituted materials. In addition, the mechanical property of rebar or concrete often has a certain level of uncertainty. For these reasons, RC materials tend to be accurately represented by organic glass or other substituted materials. Nevertheless, considering its cost effectiveness as compared to shake-table test, the quasi-static test is still deemed to be an effective way for seismic failure mechanism assessments of RC pylons.

This study presents results of a quasi-static test on a 1/35-scale inverted Y-shape RC pylon model. To facilitate the limited loading equipment in laboratory, a displacement-controlled two-node load-pattern for the single pylon model is first proposed through numerical analyses on the full cable-stayed bridge model. Physical observations recorded in the test are described and interpreted. Meanwhile, the quasi-static test is simulated using FE method. Based on the validated FE model as well as the test records, the seismic failure mechanism of the RC pylon model is revealed. In addition, multi-level displacement ductility factors are proposed to quantify ductile properties of the test model. After that, based on the validated FE model, parametric studies are performed to understand the impact of loading displacement ratios for the two-node load pattern on the seismic failure mechanism and ductility of the RC pylon model. Finally, conclusions and limitations are addressed.

2. Design of the quasi-static model test

2.1. Selection of prototype for long span cable-stayed bridges

Table 1 summarizes long span cable-stayed bridges with RC pylons (main spans beyond 600 m). Fig. 1 shows typical shapes for such pylons. As can be seen from Table 1, the inverted Y- and diamond-shape are the most popular ones. They share the same inverted Y-shape portion above the crossbeam, and the only minor difference is the portion below the crossbeam. In fact, these two shapes have quite similar static and dynamic characterizations [20]. Ratios of pylon height to main span fall into a range of 0.251 ~ 0.339. Among these bridges, the Sutong Bridge in China is the most widely studied case in literature, including areas such as earthquakes, winds and structural health monitoring [21,37–41], which provide sufficient information on numerical modelling of the bridge that can be used to validate the FE model in this study. In this regard, the Sutong Bridge is adopted as the prototype in this study, which is a box-girder cable-stayed bridge with twin 300 m-height inverted Y-shape pylons. More details on configurations of the Sutong Bridge can be found in [18,21,37,39].

2.2. Simplified two-node lateral load-pattern

The Pushover analysis is an increasingly popular and generally direct method for assessing the seismic failure mechanism of structures that are subjected to monotonically increasing loads up to failure [29]. Recalling the basic theory of the Pushover analysis as well as the fact that high order modes of pylons for long span cable-stayed bridges may significantly contribute to the seismic responses of pylons [26], it is best to “push” the pylons with load-patterns imitating shapes of the
predominant modes. However, for the quasi-static pushover test, due to limited loading equipment and technique in laboratory, it is practically impossible to consider all of those modes. In this regard, this study explores a simplified two-node load-pattern for the quasi-static pushover test of the inverted Y-shape pylon, one at the bifurcation-node and the other at the crossbeam. To this end, a full bridge FE model is built and subjected to a series of ground motions using incremental dynamic analyses (IDA) [42]. The simplified two-node load-pattern is then proposed based on the obtained displacement ratios across increasing intensity measures such as PGA. A primary focus of this section is to examine whether the obtained displacement ratios develop to generally stable values from low to high intensities (i.e., from elastic to plastic states).

2.2.1. Full bridge FE model and validation

A three-dimensional FE model of the Sutong Bridge is built in OpenSees [43], an open source platform. Fig. 2 schematically shows the FE model and pylon sections. Plasticity-distributed fiber models are used to represent pylon sections accounting for material nonlinearity and axial force-moment interaction. The Kent-Scott-Park model [44,45] is adopted to model concrete fibers. For the concrete cover, peak strength $f_{c,\text{cover}} = 32.40$ MPa, corresponding to a strain $\varepsilon_{c,\text{cover}} = 0.002$; ultimate strain $\varepsilon_{u,\text{cover}} = 0.004$, corresponding to an ultimate strength $f_{u,\text{cover}} = 22.73$ MPa. As for the concrete core, parameters for the fiber models are listed in Table 2. The rebar is represented by a bilinear model with kinematic hardening, with the following parameters: yielding strength $f_y = 400$ MPa, Young’s modulus $E_y = 200$ GPa and strain-hardening ratio $b_s = 0.01$. It is worth noting that these values are theoretical values from design documents.

![Fig. 1. Typical pylon shapes for long span cable-stayed bridges: (a) A-shape; (b) Inverted Y-shape; (c) Diamond-shape; and (d) H-shape (more struts may be adopted).](image)

![Fig. 2. Schematic illustration of finite element modelling of the Sutong Bridge prototype.](image)
Table 2
Model parameters for the confined concrete of the pylon in the full bridge FE model.

<table>
<thead>
<tr>
<th>Section #</th>
<th>$f_{L,CORE}$ (MPa)</th>
<th>$\varepsilon_{L,CORE}$</th>
<th>$f_{L,STAB}$ (MPa)</th>
<th>$\varepsilon_{L,STAB}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>35.53</td>
<td>0.0022</td>
<td>26.51</td>
<td>0.0134</td>
</tr>
<tr>
<td>II</td>
<td>35.65</td>
<td>0.0022</td>
<td>25.95</td>
<td>0.0137</td>
</tr>
<tr>
<td>III</td>
<td>35.14</td>
<td>0.0022</td>
<td>26.82</td>
<td>0.0122</td>
</tr>
</tbody>
</table>

Displacement-based beam-column elements with fiber sections discretized into 15 m in depth (i.e., approximately one time of the section length; 5 integration points for each element) are used to model inclined columns of the pylon. This modelling technique is empirically deemed to produce a stable result of plastic hinge region [46–48]. The Corotational Transformation command in OpenSees [43] is used to account for the P-Δ effect [49]. Elastic beam-column elements are used to model (1) the crossbeam of pylon, (2) the upper vertical column of pylon, (3) the deck, and (4) piers. Reasons for such modelling are below. The crossbeam is normally designed in elastic behavior (e.g., allocated with high-strength prestressed rebars) to ensure a reliable load-path under earthquakes, while the upper vertical column and the deck are often not controlled by earthquake loads. As for piers, a preliminary analysis by the authors show that their nonlinearities have almost negligible impacts on transverse seismic responses of pylons. Hence, elastic elements are used to model the piers for computational efficiency. A floating deck system is used in the longitudinal direction, while rigid deck-pylon connections are used in the transverse direction due to the application of wind-resistance bearings at the pylons. Bearings for pier-deck connections are transversally fixed for the purpose of transverse stability of the bridge. Cables are modeled as large-displacement truss elements using the Ernst method (modified modulus of elasticity) [50] owing to its convenient usage and the capability to account for the sag effect. To simulate as-built tensional forces in cables, initial strains are assigned in these elements without masses. It is worth noting that local cable dynamics are neglected since Caetano et al. [51] suggested that only a narrow band of earthquake excitation frequencies can cause dynamic interactions between local cables and the global system. Soil-structure interaction effects are neglected for simplicity (i.e., piers and pylons are base-fixed). Before dynamic analyses, a gravity analysis is conducted to verify the as-built condition for the FE model; that is, almost identical cable-forces and deck-deformations between the FE model and prototype (i.e., absolute percentage errors are within 2%).

Modal analyses show first seven modes of the OpenSees FE model with periods of 15.86, 9.89, 5.61, 4.52, 3.53, 3.22 and 2.68 s. On the other hand, Wang et al. [39] used ANSYS [52] to model the Sutong Bridge with the same constraints (i.e., neglecting soil-pile interactions) and predicts the first seven corresponding modes with periods of 16.07, 10.01, 5.50, 4.47, 3.49, 3.08 and 2.68 s. It is seen that very close results are achieved between the two FE models (i.e., absolute percentage errors are within 5%). Ground motions are input uniformly in the transverse direction. Average-acceleration-based Newmark-β integrator [53] and Newton algorithm [54] are used to solve dynamic equilibrium equations.

2.2.2. Adopted ground motions for IDA

Based on the site condition of Sutong Bridge [55], twelve non-pulse-like ground motion records are selected from PEER-NGA strong motion database [56]. Table 3 lists their information, including stations, record sequence number (RSN), moment magnitude ($M_w$) and source distance ($R$). Fig. 3 shows their acceleration time histories scaled to the same PGA, together with their durations. From this figure, it is seen that the adopted ground motions contain a wide range of duration (i.e., 30 – 120 s for the total duration and 2.3 – 50 s for the significant duration [57]). Fig. 4 illustrates pseudo-acceleration spectra of these motions and their mean as well as the site spectrum. In general, the adopted ground motions have a wide range of predominant periods (i.e., 0.15 – 0.54 s), which generally covers the platform segment of the acceleration spectrum for the site of Sutong Bridge [55]. These records are then scaled to 0.1 – 1.0 g with a step of 0.1 g. It is worth noting that the upper boundary of 1.0 g is found to trigger apparently inelastic behavior of the bridge.

2.2.3. Displacement ratios for two-node load-pattern and correlations to motion parameters

Fig. 5 shows the IDA-based development of displacement ratio ($\delta$) between the bifurcation-node and crossbeam when the maximum bending moment along the pylon reached. From this figure, $\delta$ is found to be motion-dependent (i.e., $\delta$ ranges from approximately 1 to 7). Among them, most of the ground motions lead to generally stable results across the increasing PGA (mostly for those beyond 0.3 g). As for the Iwate AOMH13 record (#4) that contains an extremely long duration (recall Fig. 3), $\delta$ keeps almost constant for PGA = 0.1 – 0.5 g and then gradually rises up to nearly 7 (being generally stable for those beyond 0.9 g). This result may be attributed to the fact that the ground motion duration often has a noted impact on inelastic behavior of RC components [58]. Besides, as highlighted in Fig. 5 for the Chi-chi TC0054 record (#1), the stable $\delta$ value of nearly 5 is adopted as the two-node load-pattern in the quasi-static test described later, since this earthquake witnessed the damage of RC pylon for the Chi-ku cable-stayed bridge, which is the unique documentation of recorded damage to RC pylon as yet.

Since $\delta$ is found to be motion-dependent, correlation analyses are performed to formulate $\delta$ with ground motion parameters. Values of $\delta$ in the correlation analyses are those at PGA = 1.0 g in Fig. 5. On the other hand, since IDA are performed, amplitude-related parameters such as PGA and Spectrum Intensity are not appropriate for the correlation analyses. Therefore, this study focuses on frequency- and duration-related ground motion parameters, as listed in Table 4. Fig. 6 shows results of the correlation analyses, including coefficients of determination ($R^2$) and Pearson’s linear correlation coefficients ($p$) [59], with respect to the studied parameters. It is found that the bracketed duration ($D_b$) has the highest correlation, closely followed by the number of effective cycles and the significant and uniform durations, while the frequently used predominant period shows the least correlation. Fig. 7 shows $\delta$-$D_b$ regression models in linear, logarithmic and logarithmically linear forms. These formulations can be used to estimate the displacement ratio of the two-node load-pattern for inverted V-shape RC pylon subjected to ground motions. It should be noted that application ranges of these formulations are limited to the upper and lower boundaries of $D_b$ among the adopted ground motions as well as the studied object. In addition, $D_b$ can be estimated through ground motion prediction equations in [57] or related hazard analyses, which is out of the scope of this study.

Table 3
Information of the adopted PEER-NGA ground motion records for IDA.

<table>
<thead>
<tr>
<th>#</th>
<th>Earthquake, Year</th>
<th>Station</th>
<th>RSN</th>
<th>$M_w$</th>
<th>$R$ (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Chi-Chi, 1999</td>
<td>TCU054</td>
<td>1494</td>
<td>7.62</td>
<td>5.28</td>
</tr>
<tr>
<td>2</td>
<td>Loma Prieta, 1989</td>
<td>Piedmont Jr. High School</td>
<td>788</td>
<td>6.93</td>
<td>73.00</td>
</tr>
<tr>
<td>3</td>
<td>Loma Prieta, 1989</td>
<td>San Francisco Sierra Pt.</td>
<td>804</td>
<td>6.93</td>
<td>63.15</td>
</tr>
<tr>
<td>4</td>
<td>Iwate, 2008</td>
<td>AOMH13</td>
<td>5530</td>
<td>6.90</td>
<td>159.62</td>
</tr>
<tr>
<td>5</td>
<td>Chi-Chi, 1999</td>
<td>TCU129</td>
<td>3507</td>
<td>6.30</td>
<td>36.81</td>
</tr>
<tr>
<td>6</td>
<td>Coyote Lake, 1979</td>
<td>Gilroy Array #6</td>
<td>150</td>
<td>7.57</td>
<td>9.12</td>
</tr>
<tr>
<td>7</td>
<td>Chi-Chi, 1999</td>
<td>TCU045</td>
<td>1485</td>
<td>7.62</td>
<td>77.91</td>
</tr>
<tr>
<td>8</td>
<td>Loma Prieta, 1989</td>
<td>UCSC</td>
<td>809</td>
<td>6.93</td>
<td>24.05</td>
</tr>
<tr>
<td>9</td>
<td>Loma Prieta, 1989</td>
<td>Gilroy Array #1</td>
<td>765</td>
<td>6.93</td>
<td>33.55</td>
</tr>
<tr>
<td>10</td>
<td>Helena, Montana, 1935</td>
<td>Carroll College</td>
<td>1</td>
<td>6.00</td>
<td>8.71</td>
</tr>
<tr>
<td>11</td>
<td>Northbridge, 1994</td>
<td>Vasquez Rocks Park</td>
<td>1091</td>
<td>6.69</td>
<td>41.90</td>
</tr>
<tr>
<td>12</td>
<td>Victoria, Mexico, 1980</td>
<td>Cerro Prieto</td>
<td>265</td>
<td>6.33</td>
<td>35.48</td>
</tr>
</tbody>
</table>
2.2.4. Validation of the two-node load-pattern

To validate the two-node load-pattern, a single pylon FE model is extracted from the full bridge FE model and then subjected to the proposed two-node load pattern. Note that the cable-induced initial axial loads in the pylon are considered in the single pylon model. Fig. 8 compares envelope responses of axial force, shear force, bending moment and defl ection along the pylon that are obtained from the dynamic analysis of the full bridge FE model under 1.0 g Chi-chi TCU054 wave and the corresponding two-node load-pattern pushover analysis of the single pylon FE model with the displacement ratio of 5, together with illustrations of plastic regions and their occurrence orders.

From Fig. 8, it is seen that plastic regions appear first at the bottom of upper column, then at the bottom of lower column and finally at the top of lower column. These sections are deemed to be critical sections. In general, peak internal forces and lengths of plastic regions near these critical sections agree reasonably well between the dynamic and two-node pushover analyses. Considering the inherent simplicity of the two-node load-pattern itself, it is reasonably deemed to be capable of capturing the characteristics of inverted Y-shape RC pylons under later loads. In addition, it is worth noting that the proposed two-node load-pattern for pushover analysis can be used for efficient estimates of seismic responses of as-built bridges that were not designed for occasionally large earthquakes, which can assist in seismic retrofits and maintenance. On the other hand, for bridges under design at early stages of projects that dimensions are often changed, the two-node pushover analysis can efficiently estimate responses and consequently assist in optimal seismic designs.

2.3. Single pylon model with equivalent dead load

2.3.1. Similarity coefficients and test model simplification

Due to the limited space of Earthquake Engineering Hall of Tongji University at Jiading campus where this test took place, the Buckingham π theorem of dimensional analysis [64] is followed to scale the single pylon. Table 5 lists the adopted similarity coefficients, where the scale factor for length is set as 1/35, whereas scale factors for material properties of stress, strain and elastic modulus are all set as 1.0. It is worth noting that in this manner, real steel and concrete
Table 4
Studied frequency- and duration-related ground motion parameters.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Name (Unit)</th>
<th>Definition</th>
<th>Ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td>V/A</td>
<td>Peak velocity and acceleration ratio (s)</td>
<td>Peak ground velocity (PGV) divided by PGA (i.e., PGV/PGA).</td>
<td>/</td>
</tr>
<tr>
<td>Tp</td>
<td>Predominant period (s)</td>
<td>Period where the maximum spectral acceleration occurs.</td>
<td>/</td>
</tr>
<tr>
<td>Tm</td>
<td>Mean period (s)</td>
<td>( T_m = \sum (C_i f_i / \sum C_i) ), where ( C_i ) are Fourier amplitudes; ( f_i ) are discrete Fourier transform frequencies between 0.25 and 20 Hz.</td>
<td>[60]</td>
</tr>
<tr>
<td>Nc</td>
<td>Number of effective cycles (( / ))</td>
<td>Sum of ratios of cycle amplitudes in accelerogram divided by maximum cycle amplitude.</td>
<td>[61]</td>
</tr>
<tr>
<td>Dc</td>
<td>Uniform duration (s)</td>
<td>Total time during which the acceleration is larger than the limit value of 5% of PGA.</td>
<td>[62]</td>
</tr>
<tr>
<td>Db</td>
<td>Bracketed duration (s)</td>
<td>Total time elapsed between the first and the last excursions of the 5% of PGA.</td>
<td>[62]</td>
</tr>
<tr>
<td>Ds</td>
<td>Significant duration (s)</td>
<td>Interval of time over which the 5% and 95% of the total Arias Intensity [63] is accumulated.</td>
<td>[57]</td>
</tr>
<tr>
<td>Dt</td>
<td>Total duration (s)</td>
<td>The total time of the input ground motion.</td>
<td>/</td>
</tr>
</tbody>
</table>

Fig. 6. Coefficient of determination (\( R^2 \)) and Pearson’s correlation coefficient (\( \rho \)) between the displacement ratios and the studied ground motion parameters.

Fig. 7. Regression models between displacement ratios for the two-node load-pattern and bracketed duration: (a) linear; (b) logarithmic; and (c) logarithmically linear.

2.3.2. Section and reinforcement modifications for construction feasibility

Since a relatively small scale factor of 1/35 is used, wall thicknesses of ideally scaled hollow sections are quite small (i.e., 3.43 – 4.29 cm), which are very difficult for construction using real rebar and concrete materials. Therefore, the ideally scaled dimensions are not perfectly followed in the test. Instead, this study re-designed the RC hollow sections to meet prototype-model equivalence on critical limit states of the sections, including the first-yield and ultimate limit states. Note that the first-yield limit state is defined by the first rebar reaching its yielding stress while the ultimate limit state is represented by the concrete core crushing or rebar snapping, whichever occurs first. Fig. 9 shows the re-designed sections and their reinforcements. For column sections, 8 mm rebars and 6 mm bars are adopted as the longitudinal and transverse reinforcements, respectively. It should be noted that the ideally scaled stirrup-spacing of 6 cm is empirically deemed to induce buckling issue with respect to the 6 mm stirrups [65]. For this matter, the stirrup-spacing is designed as 3 cm for the test. As for the crossbeam that is normally allocated with plenty of prestressed tendons in prototype to ensure an elastic behavior, a solid section with sufficient reinforcements is adopted in this test for construction feasibility as well as for ensuring a practically elastic behavior during the test.

Section moment-curvature curves are then obtained using pure bending analyses in OpenSees. Fig. 10 displays moment-curvature relationships for the ideally scaled prototype and test model under initial axial loads, together with their first-yield and ultimate limit states. From Fig. 10, they coincide generally well for individual sections. In addition, Table 6 lists the first-yield and ultimate curvatures for these sections. It is seen that errors are within 8% for the first-yield limit state...
and 15% for the ultimate limit state. Such results indicate that the re-designed RC hollow sections can generally reflect the bending behavior of the prototype sections. It should be noted that the re-designed sections are capacity-protection for shear failure as commonly adopted in structural seismic design.

2.3.3 Construction of the pylon model

Fig. 12 displays the construction process of the test model, including pasting of strain gauges, colligations of reinforcement cages, formations, concrete casting and curing. Meanwhile, concrete cubes and prisms were constructed and rebar/bars were collected and tested to determine the material properties. Skins of the test model were blanched before test for convenience of crack observations.

<table>
<thead>
<tr>
<th>Parameter (Notation)</th>
<th>Similarity relationship</th>
<th>Similarity coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (l)</td>
<td>$\delta_l$</td>
<td>0.0286 (i.e., 1/35)</td>
</tr>
<tr>
<td>Displacement ($\delta$)</td>
<td>$\delta_S = \delta_l$</td>
<td>0.0286</td>
</tr>
<tr>
<td>Rotation ($\theta$)</td>
<td>$\delta_\theta$</td>
<td>1</td>
</tr>
<tr>
<td>Strain ($\varepsilon$)</td>
<td>$\delta_\varepsilon$</td>
<td>1</td>
</tr>
<tr>
<td>Stress ($\sigma$)</td>
<td>$\delta_\sigma$</td>
<td>1</td>
</tr>
<tr>
<td>Elastic modulus ($E$)</td>
<td>$\delta_E$</td>
<td>1</td>
</tr>
<tr>
<td>Acceleration ($A$)</td>
<td>$\delta_A$</td>
<td>1</td>
</tr>
<tr>
<td>Mass (m)</td>
<td>$\delta_m = \delta_\sigma^2 / \delta_A$</td>
<td>0.000817</td>
</tr>
<tr>
<td>Force (F)</td>
<td>$\delta_F = \delta_\sigma^2 / \delta_A$</td>
<td>0.000817</td>
</tr>
<tr>
<td>Curvature ($\phi$)</td>
<td>$\delta_\phi = \frac{1}{\delta_l}$</td>
<td>35</td>
</tr>
</tbody>
</table>

Table 5

Similarity coefficients for the test model.

Fig. 8. Comparison of responses between the full bridge model dynamic analysis subject to 1.0 g Chi-chi TCU054 record and the corresponding two-node load-pattern pushover analysis: Plastic regions and occurrence orders; and envelope distributions of (a) axial force; (b) shear force; (c) bending moment; and (d) deflection along the pylon.

Fig. 9. Schematic of the quasi-static test: overview, reinforcement and instrumentation.
2.4. Instrumentation and lateral loading protocol

Again seen in Fig. 9, lateral forces and displacements at two loading points were obtained through two horizontal actuators. In addition, 38 pairs of strain gauges and 24 pairs of displacement gauges (i.e., linear variable differential transformers (LVDTs), strokes of 10 cm and precisions of 0.05 mm) were adopted to trace developments of rebar strains and/or section curvatures. Details of the technique for converting recorded raw data to curvature results can be found in Wang et al. [66,67]. It is worth noting that in the elastic range where a relatively low amplitude of curvature involves, a pair of strain gauges in a section can accurately capture strains in rebars, which are then used to derive the section curvature. For the pair of LVDTs at the same section, by contrast, the limited accuracy hinders their application in measuring small deformations of the section, but they are capable of estimating large deformations of the section such as those in high levels of inelasticity where the strain gauges often malfunction.

As mentioned before, a displacement ratio of 5 for the two-node lateral load pattern is adopted in the quasi-static test. To this end, a displacement-controlled monotonic lateral loading protocol with a ratio of 5 at two actuators (i.e., H1 and H2, as depicted in Fig. 9) is applied with a speed up to 2 mm/s. Both actuators were servo-controlled and produced by the MTS Systems Corporation (Eden Prairie, MN). The Actuator H1 has a force-capacity of 1000 kN and a stroke of ± 50 cm, while the Actuator H2 has a force-capacity of 500 kN and a stroke of ± 50 cm. Fig. 13 shows the loading protocol for the quasi-static test.

During the test, an unexpected outage occurred in the data acquisition system when the Actuator H1 pushed to a displacement level of 70 mm. All the recorded data were consequently missed. In this regard, the test model was unloaded and pulled back to the original position. It should be noted that some horizontal minor cracks were observed near bottoms of the upper and lower trailing columns during this unsuccessful load/unload process, but they practically closed up after pulled back to the original position. After nearly a half hour of reboot and evaluation of the data acquisition system, the test model was reloaded and continued until an unexpected rebar snapping (explained later) occurred when Actuators H1 and H2 pushed up to 320 mm and 64 mm, respectively. For convenience, displacement values mentioned later in this paper refer to the displacements at Actuator H1 (i.e., the bifurcation-node).

3. Physical damage observation

Table 7 summarizes the observed critical damage to the test model. Fig. 14 shows global and local damage as depicted in Table 7. It is found that horizontal cracks are the most frequent damage in the test, indicating a flexural damage mode for the pylon model. Plastic regions were observed at bottoms of the upper and lower trailing columns and lower leading column as well as the top of lower trailing column, which are generally consistent to the critical sections derived from the dynamic analysis mentioned above (see Fig. 8).

4. Test and numerical analyses for failure mechanism assessment

In order to reveal the seismic failure mechanism of Y-shaped RC pylon for long span cable-stayed bridges as well as due to the limited test records, numerical modelling is adopted as an assistance. A refined FE model for the test model is built in this section. Results from the test and numerical model, including force-displacement relationships, curvature and strain distributions, are compared to validate the refined FE model, which is then used to capture variations of axial forces during the test. The influence of such variations on the yield- and ultimate-
states of sections are considered to reveal the seismic failure mechanism of the test model.

4.1. Refined FE modelling of the quasi-static test

Fig. 15 schematically shows the refined FE modelling of the quasi-static test using OpenSees [43]. Compared to the full FE model of Sutong Bridge (see Fig. 2), the refinements of this FE model are reflected in the following aspects.

(1) Column-crossbeam joints are modeled using rigid elastic beam-column elements, together with equal degree-of-freedom (EqualDOF) technique, as illustrated in Fig. 15. The reason to use the EqualDOF technique is due to the fact that anchor rods were connected to Actuator H2 and extended into the crossbeam (see Fig. 9). This configuration leads to nearly synchronous movement and rotation for the joint and end-crossbeam nodes.

(2) The crossbeam is modeled using displacement-based beam-column elements to check and confirm the practically elastic behavior of crossbeam during the test.

(3) The bond-slip effect at bottoms of the pylon is modeled using zero-length-section elements with the Bond SP01 material [68] in OpenSees, in which the key parameters are as below: the yield slip, \( S_y = 0.36 \text{ mm} \), determined using Eq. (1) [68]:

\[
S_y = 2.54 \left( \frac{d_b}{8437} \frac{f_y}{\sqrt{f_c}} (2k + 1) \right)^{1/3} + 0.34
\]

where \( d_b = 10 \text{ mm} \) is the rebar diameter; \( f_y = 450 \text{ MPa} \) is the yielding stress of rebar obtained from the material test; \( f_c = 41 \text{ MPa} \) is the concrete compressive strength; and \( k = 0.4 \) is the parameter used for the local bond-slip relation [69]. Accordingly, the ultimate slip, \( S_u = 38S_y = 13.8 \text{ mm} \) [68]. It is worth noting that in the test model, the anchorage length of rebars is about 40 cm (see Fig. 9), which satisfies the minimum requirement of anchorage length (approximate 11 cm) for the adopted bond-slip model [68]. In addition, to assess the influence of this bond-slip effect, another FE model with conventional base-fixed constraints is built and analyzed for comparison.

Table 7 lists constitutive parameters of this refined FE model, which are obtained through a series of material tests as well as the related concrete and steel models in literature [44,45]. Meshes of the fiber sections are shown in Fig. 15 as well. The displacement-controlled loading technique in OpenSees [43] is used to simulate the two-node load-pattern pushover in the test. To solve the nonlinear equilibrium equations, algorithms such as Newton, Newton-with-line-search and Krylov-Newton [70], are tried in turn to reach convergence results as soon as possible. The convergence tolerance on the norm of the displacement residuals is set to \( 10^{-4} \).

Table 7

<table>
<thead>
<tr>
<th>Disp.</th>
<th>Description of observed damage</th>
<th>Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td>25 mm</td>
<td>Horizontal crack at the outer transverse-normal surface of the bottom of upper trailing column</td>
<td>14(a)</td>
</tr>
<tr>
<td>35 mm</td>
<td>Horizontal crack at the outer transverse-normal surface of the bottom of lower trailing column</td>
<td>14(b)</td>
</tr>
<tr>
<td>45 mm</td>
<td>Horizontal crack at the inner transverse-normal surface of the top of lower trailing column</td>
<td>14(c)</td>
</tr>
<tr>
<td>50 mm</td>
<td>Horizontal crack at the outer transverse-normal surface of the top of upper trailing column (just below the bifurcation-node)</td>
<td>14(d)</td>
</tr>
<tr>
<td>140 mm</td>
<td>Diagonal crack at the transverse-parallel surface of trailing column-crossbeam intersection</td>
<td>14(e)</td>
</tr>
<tr>
<td>320 mm</td>
<td>Concrete cover began to spall at the bottom of lower leading column; Rebar snapped at the bottom of upper trailing column; Concrete cover spalling extended along the elevation; Few minor cracks along the top of upper trailing column</td>
<td>14(f)</td>
</tr>
<tr>
<td>320 mm</td>
<td>Test ended.</td>
<td>14(f)</td>
</tr>
</tbody>
</table>
4.2. Validation of the numerical model

4.2.1. Global force-displacement relationships

Fig. 16 shows recorded and predicted results of global force-displacement relationships at two loading points. The predicted result is based on the full loading protocol shown in Fig. 13 to account for the degradation of stiffness due to the initial unsuccessful load/unload process, while the recorded counterpart represents the latter successful reload process.

From Fig. 16(a), it is seen that the predicted results at the bifurcation-node coincide quite well with the recorded counterpart, regardless of whether the bond-slip effect is considered or not. As for the results at the crossbeam (Fig. 16(b)), some dispersions are observed between the recorded and predicted results for displacement levels beyond 30 mm. Nevertheless, the FE model with the bond-slip effect produces a relatively smaller lateral stiffness and consequently a better prediction as compared to the model without the bond-slip effect.

Table 8

<table>
<thead>
<tr>
<th>Material</th>
<th>Parameter (Unit)</th>
<th>Upper column</th>
<th>Lower column</th>
<th>Crossbeam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete cover</td>
<td>$f_{c,cover}$ (MPa)</td>
<td>41.00</td>
<td>41.00</td>
<td>41.00</td>
</tr>
<tr>
<td></td>
<td>$f_{c,cover}$ 0.002</td>
<td>0.002</td>
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<tr>
<td></td>
<td>$f_{c,cover}$ 14.88</td>
<td>14.88</td>
<td>14.88</td>
<td>14.88</td>
</tr>
<tr>
<td></td>
<td>$f_{c,cover}$ 0.006</td>
<td>0.006</td>
<td>0.006</td>
<td>0.006</td>
</tr>
<tr>
<td>Concrete core</td>
<td>$f_{c,core}$ (MPa)</td>
<td>53.66</td>
<td>54.82</td>
<td>50.91</td>
</tr>
<tr>
<td></td>
<td>$f_{c,core}$ 0.0026</td>
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<td>0.0025</td>
<td></td>
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<tr>
<td></td>
<td>$f_{c,core}$ 40.08</td>
<td>41.24</td>
<td>41.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$f_{c,core}$ 0.042</td>
<td>0.045</td>
<td>0.034</td>
<td></td>
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<tr>
<td>Rebar</td>
<td>$f_y$ (MPa)</td>
<td>450</td>
<td>450</td>
<td>450</td>
</tr>
<tr>
<td></td>
<td>$E_s$ (GPa)</td>
<td>169</td>
<td>169</td>
<td>169</td>
</tr>
<tr>
<td></td>
<td>$b_s$</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
</tbody>
</table>

Fig. 14. Development of the global deformation and corresponding local damage to the test model for increasing displacement levels at the bifurcation-node: (a) 25 mm; (b) 35 mm; (c) 45 mm; (d) 50 mm; (e) 140 mm; (f) 320 mm (ending displacement).

Fig. 15. Refined FE modelling of the quasi-static test: schematic and section mesh.
4.2.2. Curvature developments

Fig. 17 shows recorded and predicted curvature developments at the trailing and leading columns with increasing indicative displacement levels from 10 to 300 mm. It is worth noting that in this figure, the recorded results at relatively small displacement levels of 10 and 50 mm were derived from the strain gauges on rebars, whereas the rest of recorded results were derived from the displacement gauges. Note that some gauges were impaired during the concrete casting process or malfunctioned during the test. The impact of bond-slip effect is assessed as well.

From Fig. 17(a) and (b), in general terms, good agreements are achieved between the recorded and predicted results, thereby indicating the FE model with the bond-slip consideration can generally capture the curvature distributions across the considered displacement levels. In addition, comparative inspections of Fig. 17(c) and (d) to (a) and (b), respectively, imply that the bond-slip effect has a minor impact on curvature distributions of the trailing column, while it significantly affects those of the leading column, especially for those at the pylon bottom. More specifically, the FE model without the bond-slip effect may produce curvature responses three times as much as the model considering the bond-slip effect. Therefore, it can be concluded that the refined FE model with bond-slip effect is generally capable of estimating curvature responses along pylons.

4.2.3. Rebar-strain developments

Fig. 18 shows recorded and predicted developments of rebar-strain envelopes along the trailing column across relatively small displacement levels from 5 to 40 mm. Note that data at elevations of 1.2 and 1.6 m represent strain responses of the rebar located in the inner corner of the section of trailing column (refer to Fig. 9), while the rest of data represent responses of the rebar near the outer surface. The predicted results are based on the refined FE model with the bond-slip effect. From Fig. 18, generally good agreements are observed between the predicted and recorded results, which further validates the refined FE model.

It is worth noting that someone may argue that the bond-slip effect could also be involved in the column-crossbeam joints. For this issue, the beamColumnJoint element proposed by Lowes et al. [71] in the
framework of OpenSees may be a practical solution. However, this element falls into a limited application range where the test model in the present study is not completely included. More specifically, (1) each spring in the beamColumnJoint element is characterized by deformation behavior including effects of both bond-slip and cracking; this task often requires adequate FE modelling experience on parameter determinations as well as reliable test results related to such effects, which are absolutely beyond the obtained information in this test; and (2) springs in the beamColumnJoint element are placed symmetrically with respect to the centroid of the joint and thus may not be amenable to treating unsymmetrical situations such as the sloping column-crossbeam joints in this test. Therefore, the beamColumnJoint element as currently available in OpenSees is not well suitable for the present study. Nevertheless, considering the above validation results for the refined FE model that accounts for the bond-slip effect at bottoms as well as its relatively easy implementation, it is reasonable to conclude that the refined FE model is capable of capturing the primary characteristics and responses of the RC pylon.

4.3. Failure mechanism assessment considering variations of axial loads

4.3.1. Indicative process to determine the yielding and ultimate limit states

It is well known that for framed structures such as the studied pylon, section axial loads shall vary under lateral loads, which inevitably affects yield- and ultimate-states of the section [66,72]. In other words, the variation in axial loads affects the “capacity” of the section. In this regard, the validated FE model is used to trace the axial loads during the test, which are then used in moment-curvature analyses of sections to determine the curvatures at the yield- and ultimate-states (i.e., “capacity”). On the other hand, developments of curvature responses (i.e., “demand”) are predicted using the FE model as well as recorded through strain/displacement gauges. The “demand” results across various displacement levels are plotted together with the “capacity” counterparts to identify intersection points that present the yield and ultimate limit states of the section.

Fig. 19 shows the indicative process to determine the yield-state of bottom section of lower trailing column and the ultimate-state of bottom section of lower leading column. Variations of the axial loads are plotted together in this figure. From Fig. 19(a), it is seen that the bottom of lower trailing column yields at displacement levels of 38 mm for the predicted result and 43 mm for the recorded result. They are reasonably close. It is worth noting that the section yield-state can also be determined directly through the development of strain distribution shown in Fig. 18. In this case, the “capacity” represents the yielding strain of rebar obtained from the material test (i.e., 0.00267 = 450 MPa (yielding stress)/169 GPa (elastic modulus), refer to Table 8). In such a way, the indicative section is found to yield at a displacement level around 40 mm, which is quite close to the above curvature-based method. As for Fig. 19(b), the predicted result indicates the fail (concrete core crushing) of bottom section of lower leading column at a displacement level of 553 mm, which is extremely larger than the ending displacement of 320 mm in the test.

In addition, it is worth recalling the variation of initial axial compressive force (60 kN) at the top section of the lower column between the ideally scaled prototype and the test model (Fig. 10). Based on the process to determine the yield-state of the bottom section of lower trailing column as shown in Fig. 19(a), it is reasonable to infer that the relatively small variation (60 kN) should trigger quite small changes to curvature demand and capacity that are used to determine the yield-state of this section. Thus, the effect of this variation is deemed to be generally neglectable from the view of practice.

4.3.2. Failure mechanism summary

Following the above indicative process, Fig. 20 summarizes the predicted and recorded failure mechanism of the pylon model (i.e., the occurrence order of yield- and ultimate-states for the critical sections). Note that in the FE model, the first section reaching its ultimate-state is defined as the ultimate limit state of the pylon. From Fig. 20, the recorded and predicted results indicate the same yielding order for the critical sections. More specifically, for yield-states, plastic regions are detected first at the bottom of upper trailing column (① of triangles in Fig. 20), then successively at the bottom and top of lower trailing column (② and ③ of triangles); and finally at the bottom of lower leading column (④ of triangles). Fig. 21 displays the recorded and predicted displacement levels for the yield-states of critical sections. Obviously, they show quite good agreements in amplitudes with dispersions less than 15%. In addition, it is worth noting that the recorded and predicted occurrence orders of plastic regions are consistent to that obtained from the dynamic analysis mentioned above (Fig. 9). This result further validates the refined FE model that can be used for parametric analyses.

As for the ultimate-state, the predicted result shows that the bottom of lower leading column fails first (④ of squares in Fig. 20), whereas the recorded result indicates the failure of pylon at a much smaller displacement level of 320 mm where an unexpected rebar snapping occurred. This is a noted limitation of the quasi-static test. To explain this unexpected issue, a plane-hypothesis-based preliminary analysis using the recorded curvature and the dimension of this section implies that the estimated rebar strain at this level (i.e., 0.0284 = approximately 0.2 m −1 (recorded curvature) × 0.142 m (distance of the rebar to the section neutral axis)) is much less than its ultimate strain from the material test (i.e., 0.15). The reason for this unexpected failure may be attributed to the impact of the anchor rods that connected Actuator H2 with the pylon model, which inevitably inter-acted with the snapped rebar at this section. In other words, the configurations for loading points in the test may be somewhat unreasonable. Although sophisticated FE modelling using continuum media may be a possible solution to the mechanism of such a complex interaction, the limited test records related to this issue probably hinder this solution. Moreover, the unreasonable configurations are not realistic for engineering practices. Therefore, for the purpose of conciseness, such a tough investigation is not performed in this study.

It should be noted that for inverted Y-shape RC pylons under earthquakes, the two columns act as leading and trailing ones by turns. Therefore, in a view of generalization, the terms “leading” and “trailing” can be removed when summarizing the failure mechanism of the pylon.

In addition, it is worth noting that the observed plastic hinges at the columns for the crossbeam-column joints follow with current practices that often design the crossbeams sufficiently strong to assure reliable load transferring paths for pylons and/or for bearings installed on the...
crossbeams to support decks. Nevertheless, designing in a way that plastic hinges are formed at the crossbeams, which scarifies the crossbeams to protect the columns, is an alternative solution for inverted Y-shape pylons where no bearings are installed on the crossbeam, e.g., cable-stayed bridges with completely floating systems.

4.4. Ductility factor definition and result

According to the obtained failure mechanism, multi-level ductility factors are introduced as preliminary documents for performance-based ductility design of bridge RC pylons in future.

A multi-level displacement ductility-limited factor (or called displacement ductility factor for limited ductile design), $\mu_i$, that accounts for the number of plastic hinges ($i$) is defined using Eq. (2).

$$\mu_i = \frac{\Delta_0}{\Delta_{0; i}}$$  \hspace{1cm} (2)

where $\Delta_0$ is the displacement level for the first plastic hinge; and $\Delta_{0; i}$ ($i = 2, 3, 4, \ldots$) represents displacement levels for second, third and fourth etc. plastic hinges, which are marked in Fig. 20. Moreover, an ultimate displacement ductility factor, $\mu_u$, that represents the full ductile capacity of the test model is expressed in Eq. (3).

$$\mu_u = \frac{\Delta_{ult}}{\Delta_{y; 1}}$$  \hspace{1cm} (3)

where $\Delta_{ult}$ represents the displacement level for the ultimate limit state of the test model.

Table 9 lists recorded and predicted ductility factors for the pylon model. Among them, the displacement ductility-limited factors ($\mu_i$) fall into similar ranges of 1.15 $\sim$ 3.67 and 1.23 $\sim$ 3.00 for the predicted and recorded results, respectively (i.e., the variations of 6.5$\%$ $-$ 22.3$\%$ are generally small from the view of practice). As for the ultimate displacement ductility factor ($\mu_u$), relatively large values are obtained for both recorded and predicted results. The apparent dispersion between them is due to the noted distinction of their ultimate limit states as mentioned before. In summary, the test pylon model exhibits a considerable ductile capacity.

5. Parametric study of loading displacement ratio

This section performs a parametric study to further understand the impact of displacement ratio for the two-node load-pattern on the failure mechanism and associated ductility factors of the pylon. Various loading displacement ratios ranging from 1 to 7 are assessed. These values are selected based on the results of IDA previously shown in Fig. 5. Note that the predicted result of the test model (loading displacement ratio of 5) are presented hereinafter for comparisons. To ensure the rest of conditions as identical as possible, the full loading protocol shown in Fig. 13 is adopted in the parametric studies, but merely the result of the reloading part is presented for conciseness.

Fig. 22 compares the failure mechanism of the RC pylon model under various loading displacement ratios and Table 10 lists the corresponding ductility factors as defined above.

It can be seen from Fig. 22 that shapes of the global force-displacement curves are dependent on the loading displacement ratios. In general, a larger loading displacement ratio leads to a larger peak strength. When measuring the secant stiffness between this peak strength and the original point, it is found that a smaller loading displacement ratio results in a smaller lateral stiffness of the pylon. This is

<table>
<thead>
<tr>
<th>Table 9</th>
<th>Recorded and predicted ductility factors for the test model.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\mu_{y; 2; i}$</td>
</tr>
<tr>
<td>(1) Recorded</td>
<td>1.23</td>
</tr>
<tr>
<td>(2) Predicted</td>
<td>1.15</td>
</tr>
<tr>
<td>[(2)-(1)]/(1) (%)</td>
<td>6.50</td>
</tr>
</tbody>
</table>
μ
μ
μ
7.89 is obtained because at a given displacement of the bifurcation-node, a smaller displacement ratio can change the failure mechanism (i.e., occurrence order of plastic hinges) of the RC pylon model. More specifically, cases with larger loading displacement ratios such as 5 and 7 trigger the first plastic hinge at the bottom of upper column, whereas those with relatively smaller loading displacement ratios (e.g. 1 and 3) yield first at the bottom of lower column. This is also due to the relatively larger deflection for the lower column under a smaller loading displacement ratio as compared to a larger one. In addition, it is interesting to note that for cases with small loading displacement ratios, the bottom of upper column (∅ in Fig. 22) yields even later than the top of lower column (○). Recalling the generally good correlation between loading displacement ratios and bracketed durations of ground motions (Fig. 7), it is reasonable to speculate that ground motions with relatively short durations (e.g. those with bracketed durations less than 30 s, as referred to Fig. 7) may transfer the location of first plastic hinge from the bottom of the upper column to that of the lower column, as compared to long duration ground motions. Moreover, it is worth noting that for the studied largest loading displacement ratio of 7, the fifth plastic hinge occurs at the top of lower leading column (i.e., ∅ in Fig. 22) before the failure of any section.

Table 10 lists results of ductility factors, together with their statistical properties including mean, standard deviation and coefficient of variance (COV). In general, the loading displacement ratio shows a generally slight impact on the displacement ductility-limited factors for the second and third plastic hinges (i.e., COV less than 10%), while it significantly affects those for the fourth plastic hinge and the ultimate ductility factor. In particular, a larger value of \( \mu_{4,5}^u = 7.89 \) is obtained for the case of loading displacement ratio of 1. This is because this case leads to a quite small displacement level for the first plastic hinge (∅ on the curve) while a relatively larger displacement for the fourth plastic hinge at the bottom of upper column (i.e., ○ on the curve), as described above. Besides, a larger loading displacement ratio tends to trigger a larger ultimate ductility factor. In other words, inverted Y-shape RC pylons subjected to long duration earthquakes tend to have larger ductility.

6. Conclusions

This paper assesses the transverse seismic failure mechanism and ductility of an inverted Y-shape RC pylon for long span cable-stayed bridges using quasi-static tests combined with numerical analyses. A two-node lateral load-pattern, one at the bifurcation-node and the other at the crossbeam, is proposed based on IDA using recorded ground motions to facilitate the limited loading equipment and technique in laboratory. After test, a refined FE model is built and validated using the test results, which is then used to perform a parametric study for further understanding the impact of loading displacement ratios on the failure mechanism and ductile properties. Considering the inherent limitations of the test, main findings are as follows.

(1) The proposed two-node lateral load-patterns can be used for efficient estimates of seismic responses of inverted Y-shape RC pylons. Loading displacement ratios for the two-node load-pattern are found to be correlated reasonably well with bracketed durations of ground motions. Inverted RC pylons subjected to ground motions with longer durations tend to trigger larger loading displacement ratios for the proposed two-node load patterns for pushover analyses. The range of the displacement ratios is found to fall into a range of 1 to 7 in general.

(2) Test results indicate a flexural damage mode with considerable ductility for inverted Y-shape RC pylon model in the test. Plastic hinges were detected first at bottom of upper column, then at bottom and top of lower column, successively, for the pylon model subjected to the two-node loading with a displacement ratio of 5. These sections are deemed critical portions for seismic design of inverted Y-shape RC pylons.

(3) Results from the refined numerical model of the inverted Y-shape pylon with the bond-slip consideration at the base coincide reasonably well with the test results in terms of global force-displacement curves, curvature and strain distributions along column heights, and displacements at multiple yield-states.

(4) A multi-level displacement ductility-limited factor related to the number of plastic hinges is proposed in this study. The tested RC pylon model under a displacement ratio of 5 shows ductility factors of 1.23, 1.43 and 3.00 for the second, third and fourth plastic hinges formed in the test. Together with the further parametric study involving displacement ratios of 1, 3 and 7, mean ductility factors of 1.22, 1.47 and 4.65 are detected for the second, third and fourth plastic hinges, respectively. Small dispersions are detected for ductility factors for the second and third plastic hinges, while a relatively larger dispersion is detected for that for the fourth plastic hinge.

(5) The parametric study indicate that the loading displacement ratio significantly affects the seismic failure mechanism and ductility of the studied RC pylon model. A relatively smaller displacement ratio generally transfers the location of first plastic hinge from the bottom of the upper column to that of the lower column.

This study is limited to a scaled model of typical inverted Y-shape RC pylons for long-span cable-stayed bridges. Considering the length of this paper, future studies will explore comprehensive parameter spaces to assess the seismic failure mechanism and ductility factors of real inverted Y-shape RC pylons for various spans of cable-stayed bridges. In
addition, the research approach applied in this study can be used for further investigations on other types of bridge pylons or multi-story viaducts.

Acknowledgments

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Appendix A. Supplementary material

Supplementary data to this article can be found online at https://doi.org/10.1016/j.engstruct.2019.03.045.

References

2004.


