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Performance-based seismic design of unbonded precast post-tensioned concrete filled GFRP tube piers

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ABSTRACT

This paper presents a seismic design procedure for self-centering precast post-tensioned bridge piers. The piers presented in this manuscript consisted of concrete filled fiber reinforced polymer (FRP) tubes. A large set of 84 piers with different design parameters were analyzed using finite element models. The backbone curve of each pier was developed and bilinearized. The idealized backbone curves were used to develop a set of empirical equations that were able to reproduce the bilinearized backbone curve of a given pier. Different performance criteria have been proposed for the system according to the intensity and the frequency of occurrence of a seismic event. The developed empirical equations were arranged in a design procedure to achieve a given performance level at a specific seismic zone.

1. Segmental precast post-tensioned (SPPT) piers

The Kobe earthquake (Japan 1995) resulted in demolishing over 100 reinforced concrete bridge piers that reached residual drifts in excess of 1.5% [17]. This showed the need for a bridge pier's system that not only can withstand high seismic events without collapse, but also that is resilient.

Recently, a resilient segmental precast post-tensioned (SPPT) pier system consisting of precast segments stacked over each other and connected by unbonded post-tensioning tendons was developed [4,5,8,10,13,21]. Segments having hollow or solid cross sections were investigated. The segments were reinforced with longitudinal rebar and post-tensioning tendons or post-tensioning tendons only, and confined using rebar stirrups, fiber reinforced polymers (FRPs), or steel tubes. The SPPT piers showed high self-centering capabilities compared to conventional reinforced concrete (RC) piers.

Yamanobe et al. [27] investigated a different type of hybrid segmental construction. An ultra-high performance steel fiber (UFC) hollow-core segments were used in the potential plastic hinge region of bridge piers. A UFC segment had a wall thickness of approximately 20 mm. The UFC segments were filled with normal strength concrete. Two different details were used for the connection between the segments. The first detail did not use any vertical reinforcement within the segments and relied on the provided vertical reinforcement within the normal strength concrete-core for flexural resistance. The second detail used four unbonded 7.1 mm diameter rebar as vertical reinforcement inside the UFC segments. These rebar extended from the foundation to the top of the pier. Huang [14] tested two piers having hollow-core UFC segments filled with normal strength concrete in the potential plastic hinge region. Brass mesh was used to enforce opening of the interface joints between the segments.

ElGawady and Sha'lan [10] found that the average residual displacement of a SPPT bridge bent is approximately 10% of the maximum imposed lateral displacement on that bent. The residual drifts of the SPPT piers studied by Hewes and Priestley [13] were approximately 4–5% of the maximum imposed lateral displacements. It should be noted that such residual displacements were derived based on quasi static cyclic tests.

Equivalent viscous damping is an essential parameter that affects the behavior of a structural system under seismic excitations. Hewes and Priestley [13] reported, for SPPT having no flexural mild steel and subjected to quasi-static cyclic tests, an average equivalent viscous damping of approximately 5% up to a drift of 3%. Then, it increased due to spalling of the concrete cover of the reinforced concrete segments. Chou and Chen [5] reported that the equivalent viscous damping was 6.5% on average for SPPT piers with a minimum value of approximately 6%. ElGawady et al. [8] reported an average equivalent viscous damping of 5% for single segment pier similar to those investigated in this study.

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All the above mentioned experimental works showed the advantage of the SPPT system; however, there is no design procedure developed for SPPT system. This manuscript presents a design procedure for SPPT piers using empirical equations. To accomplish this target, a set of 84 piers having different design parameters were analyzed using a 3D finite element (FE) models. Nonlinear regression analyses were carried out on the results of this set of piers to develop a set of empirical design equations.

2. Summary of the 3D finite element models for SPPT

ABAQUS/Standard Version 6.8-2 [1], a general purpose finite element code, was selected as a basic platform for developing a 3D finite element (FE) model for this study. The model was presented in detail by Dawood et al. [7] and verified against three different experimental studies [7,9].

In this manuscript, the SPPT system consisted of one precast segment sandwiched between foundation and superstructure. All piers in this study consisted of concrete filled glass fiber reinforced polymer (GFRP) tubes without any rebar. The system is connected by unbonded post-tensioning tendons passing through ducts made in the segments during casting. The model was built up using 3D continuum elements for concrete and fiber reinforced polymer components and 3D beam elements for the post-tensioning tendons (Fig. 1).

The concrete damaged plasticity model [18,19] was used to model the concrete material behavior while the classic metal plasticity model was used for modelling the tendon's material. The fiber tube was modeled as an elastic orthotropic material. The ends of the tendon were embedded in the loading stub (that represents the superstructure) and the foundation to simulate the tendon's anchorage. The tendon was subjected to a stress type initial condition to simulate its post-tensioning. By neglecting the sliding of the foundation and by assuming a rigid soil underneath the foundation, the bottom surface of the foundation was constrained in the three motional directions.

Three loading steps were used for the analysis of the models. During the first step, a post-tensioning force was applied using a stress-type initial condition to the tendons. During the second step, the gravity load was applied as a traction force applied to the top surface of the model. The third loading step consisted of a monotonic push in the lateral direction simulated by a linearly increasing lateral displacement until the failure of the model occurs and the analysis was not able to proceed any further.

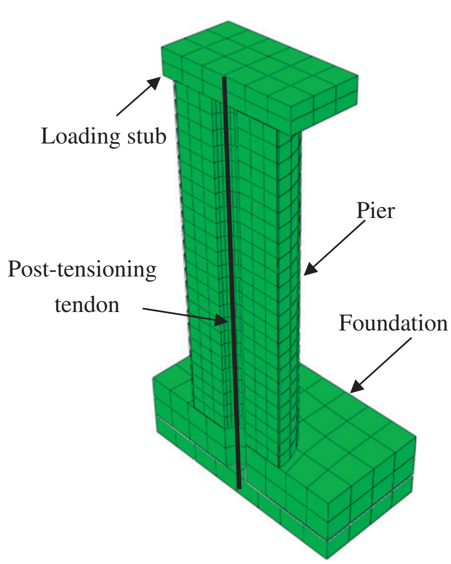

Fig. 1. FE model for SPPT pier used in this study.

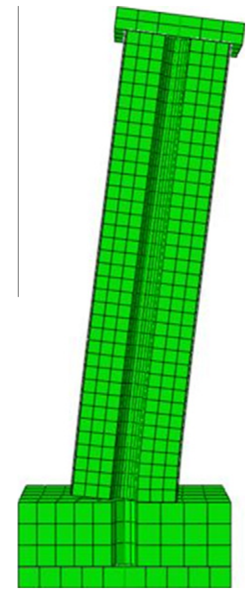

Fig. 2. Deformed shape of SPPT pier.

Fig. 2 shows the deformed shape of a pier due to the application of the lateral loads. As shown in the figure, the pier attained its lateral deformation through opening of the interface joint at the base. The FE model successfully captured the overall behavior of the system (i.e., the backbone behavior, stress concentrations at the pier's toe and failure modes). More details about the model description, implementation, and validation along with in-depth parametric studies are presented by Dawood et al. [7] and ElGawady and Dawood [9].

3. Study description

The previously described FE model was used to investigate the behavior of a large matrix of 84 piers having different design parameters, namely, height, diameter, effective post-tensioning load, and external gravity load. The values studied for each parameter are listed in Table 1. The piers were built up using plain concrete with a characteristic compressive strength (f'_c) of 41.4 MPa [6000 psi] cast directly in glass fiber reinforced polymer (GFRP) confining tubes 19 mm [0.75 in] thick (Table 2). The material characteristics of the tube were assumed based on commercially available GFRP materials. The wall thickness of the tubes was designed to avoid brittle shear failure under the anticipated ultimate lateral load of the piers.

The investigated piers have heights (H) ranging from 1830 mm [72 in] to 9144 mm [360 in] and cross sectional diameter (D) of either 1220 mm [48 in] or 610 mm [24 in]. This resulted in piers having aspect ratios of 3–15. The axial stress on the concrete induced by the gravity load normalized by its confined concrete ultimate strength f'_{cc} (DL) ranged from 5% to 10%. While the axial stress on the concrete induced by the post-tensioning force normalized by corresponding f'_{cc} (PT) ranged from 10% to 30%.

Table 1
Different investigated parameters for the SPPT piers.

Height	Diameter	PT (%)	DL (%)
1830 mm [72 in]	1220 mm [48 in]	10	5
3660 mm [144 in]	610 mm [24 in]	15	7
5490 mm [216 in]	–	20	10
9144 mm [360 in]	–	30	–