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Seismic performance comparison of flaged-shaped hysteretic and traditional bridge piers

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Abctract

Recent earthquakes have demonstrated a need for a new design philosophy for retrofitting of bridge piers that avoids damage in order to ensure post-earthquake serviceability and reduce financial loss. Self-centering and rocking systems are such approaches that eliminate residual drift, maintain post-earthquake serviceability and reduce the possibility of being demolished after earthquakes. The aim of this study is to evaluate the seismic performance of self-centering bridge pier with seat angles as energy absorption device in comparison with traditional ones. In this regard, a series of nonlinear static and dynamic analysis on a typical normal bridge is performed and significant design criteria are investigated.

Keywords: *self-centering, bridge pier, seat angle*

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1. Introduction

The extensive damage to bridges during recent earthquakes with tremendous human and economic loss associated with the disruption of lifeline in urban areas has led to a world wide effort toward improving the seismic performance of the bridges. In addition to structural damage and potential loss of life resulting from an earthquake, severe economical impact is likely to follow when closure of bridges disrupts the transportation infrastructure. Two key damage indicators need to be minimized in new design or retrofit of bridges to remain in service; as follows: (1) damage to plastic hinges and (2) permanent drifts. Traditional bridge design approaches rely on the formation of flexural plastic hinges in bridge columns as a means to dissipate seismic energy [1]. However, controlled rocking and self-centering systems in bridge piers can provide a method of seismic resistance that results in significantly less damage following strong motions. The use of controlled rocking and self-centering systems at column ends provides a means to reduce the experienced damage and residual offset in bridge systems.

This paper evaluates self-centering bridge pier with seat angles as energy absorption device along with unbonded post tension cable in comparison with traditional ones.

1.1. Self-centering and controlled rocking systems in bridge piers

The goal of the self-centering system is to minimize the residual displacements of a bridge system following a seismic event. This goal is accomplished by including in the system an element that remains elastic throughout the event, which provides the system with a restoring force. High-strength, unbonded, post-tension cables have typically been used for this purpose. Being unbonded, the cables are free to move relative to the concrete, so that extension of the bar length are distributed over the whole unbonded length. This allows the system to reach a larger displacement without yielding the restoring element. In fact, post-tensioned tendons being unbonded rather than bonded means that strains in the Post-tension rods are not localized and smaller tendons could be used. Reference (Mander and Cheng1997) proposed rocking concrete bridge columns as a seismic resistant system consistent with a proposed design methodology called Damage Avoidance Design (DAD). In this concept, each bridge column was allowed to rock individually by making the rebar discontinuous at the column ends, thus allowing rocking at the column/cap beam and column/foundation beam interfaces. The columns were subsequently designed as pre-cast elements, post-tensioned vertically to increase and control the lateral strength. Reference (Palermo, Stefano and Calvi. 2005) has proposed the use of a hybrid system for concrete bridge piers that uses vertical post-tensioning of the concrete bridge columns and different forms of energy dissipation devices (hysteretic, friction, and visco-elastic). In their study, a displacement based approach is presented for the design of both bridge piers and/or bridge systems, then, experimental and analytical investigation on these hybrid connections for the seismic resistance of concrete bridges are carried out (Marriott, Palermo and Pampanin 2006)Quasi static and pseudo dynamic testing confirmed the desired performance of the connection that included no “physical” damage and the self-centering ability. In another research, reference (Mahin, Saka and Jeong 2006) proposed a new post-tensioning design to minimize residual displacements in columns, where a longitudinal post-tensioning tendon were utilized and replaced with some of usual longitudinal mild reinforcing bars.

1.2. Energy Dissipation Systems in bridge piers

To overcome the drawback of low hysteretic energy dissipation capacity, additional energy dissipaters were used to increase the hysteretic damping of the system. In most cases, hysteretic damping comes from the yielding of the steel element. Energy dissipaters can be divided into two main categories; namely, internal and external (fuses) energy dissipation systems. In accordance with studies that carried out by references (Chang et al. 2002, Ou et al. 2007), mild steel bars are used between pier segments as internal energy dissipaters. The bars proved their efficiency by significantly increasing the hysteretic energy dissipation. The major problem with this type of energy dissipater is that, after yielding, the bars are permanently deformed and the whole system suffers from permanent residual displacement after loading.

Reference (Chou and Chen 2006) provided one of their piers with a dog bone shaped external energy dissipater. They reported that their system increased the equivalent viscous damping of the system from 6.5% to 9%. Two different layouts of external energy dissipater systems for segmental piers are used by the authors (Marriott, Pampanin, and Palermo 2009). They used mild steel bars encased in steel confining tubes and injected with epoxy to have a fuse-like behavior and to be able to dissipate energy while subjected to tension and/or compression stresses. Reference (ElGawady, Booker and Dawood. 2010) investigated piers with external steel angles and rubber pads respectively as external energy dissipaters and isolation dissipation devices.

Overall, the external energy absorption devices had the advantage of being easily changed and, hence, not increasing the residual drift of the system.

2. Hysteretic Behavior of Seat Angles

In 1999 monotonic and cyclic loading tests on isolated angles were conducted (Jay and Astaneh-Asl 1999). From results of the test the bolted-angle connections demonstrate stable cyclic response and reliable energy dissipation capacity. The increase in strength was attributed to the combined effect of large deformations and material strain hardening. The cyclic ductility of the specimens was reported to range from 8 to 10. In other experimental works, studies were performed to evaluate the post tension connection with seat angles (Ricles, Sause, Peng, and Lu. 2002- Rojas, Ricles, and Richard 2005) Results clarified the effects of altering different parameters such as the thickness and gauge length of the angles and the post-tensioning steel.

2.1. Hysteretic behavior of seat angle model

In the literature, hysteretic model of seat angle was proposed to model seat angles of post-tensioned hybrid coupled wall (Quick, Kurama, and Weldon 2006). The rocking of beam on the wall in mentioned study is similar to rocking of proposed column on the foundation, so in this study, mentioned model is utilized to model the behavior of seat angles as external energy absorption device.

Reference (Quick, Kurama, and Weldon 2006). assumed that the failure of the angle occurs through the formation of two plastic hinges in the vertical leg; as shown in Figure (1).

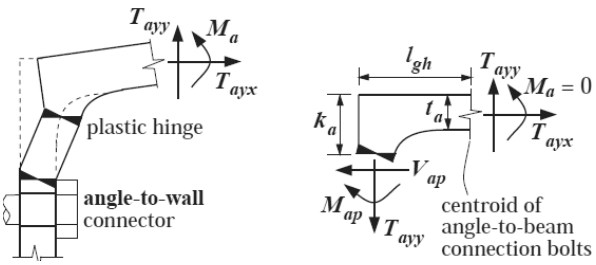

Figure (1) . Deformed shape and body diagram of the angle (Quick, Kurama, and Weldon 2006)..

In Figure 1, from the free body diagram of the angle between the plastic hinge adjacent to the fillet on the vertical leg and the centroid of the angle-to-beam connection bolts, it can be shown that:

$$T_{ayx} = V_{ap} \tag{1}$$

$$M_{ap} = \frac{V_{ap} l_{g2}}{2} \tag{2}$$

$$V_{ap} = \frac{f_{ay} l_a t_a}{2} \tag{3}$$

In which, *Tayx* = axial force in the angle horizontal, *Map* = plastic hinge moment, *Vap* = plastic shear force in the vertical leg including shear-flexure interaction, *fay* = yield strength of the angle steel, *la* = length of angle, *ta* = angle leg thickness and *lg2* = effective gage length for assumed plastic hinge mechanism. Under tensile loading, the yield strength is equal to *Vap*. Initial stiffness (*Kaixt*) were determined respectively by Equations 4 and 5 (Kishi, and Chen 1999, Lorenz, Kato, and Chen1993). In this model, the vertical leg is assumed to be fixed along the inner most edge of the line of angle-to-beam connectors and is pulled horizontally by the beam.

$$\frac{l_{g2}}{t_a}\left(\frac{V_{ap}}{V_{a0}}\right) + \left(\frac{V_{ap}}{V_{a0}}\right)^4 = 1 \tag{4}$$

$$K_{aixt} = \frac{3E_a I_a}{l_{g1}(l_{g1}^2 + 0.78 t_a^2)} \tag{5}$$

In the above equations, *EaIa* = a flexural stiffness of angle, *ta* = the thickness of angle and *lg1* = the length of vertical leg that is assumed to act as cantilever.

In this model, it is assumed that the maximum strength of the horizontal angle element in tension (*Tasx*) equal to 2 times the yield strength (*Tayx*) and is reached at an angle deformation (*δasx*) of 6 times the yield deformation. Under compression, the initial stiffness of an angle as it is pushed back horizontally toward the wall by the coupling beam is assumed to be equal to:

$$K_{aixc} = \frac{1}{40} \cdot \frac{E_a A_a}{l_{gh}} \tag{6}$$

In which, *Ea* = Young's modulus for the angle steel, *Aa* = gross cross-section area of the angle horizontal leg and *lgh* = gage length of the angle-to-beam connectors (measured from heel of the angle to the centroid of the angle-to-beam connection bolts). The unloading stiffness is assumed to be equal to the initial stiffness (*γunl*=1) for the modeling of steel unbonded post-tensioned coupling beams. Upon crossing the zero-force axis, the angle force-deformation behavior shoots towards the angle yield strength in compression (*Cayx*) which is assumed to be equal to 0.75 times the initial slip critical force, (*Casi*) of the angle-to-beam connection bolts. The 0.75 factor accounts for the losses that occur in the clamping forces of the angle-to-beam connection bolts and the resulting losses in the slip critical force as the structure undergoes large lateral displacements. In the compression angle, the beam corner comes into contact with the wall. Note that slip of the angle-to-beam connection bolts can also occur when the angle is pulled away from the wall this is not a desirable type of behavior. It is assumed that the slip critical capacity of the angle-to-beam connection bolts (*C as=0.75Casi*) is larger than the angle capacity in tension (1.25*Tayx*); and thus, slip does not occur in tension. The angle-to-beam connections should be designed to ensure this behavior.

Hysteretic behavior of seat angle that was proposed is presented in Figure 2.

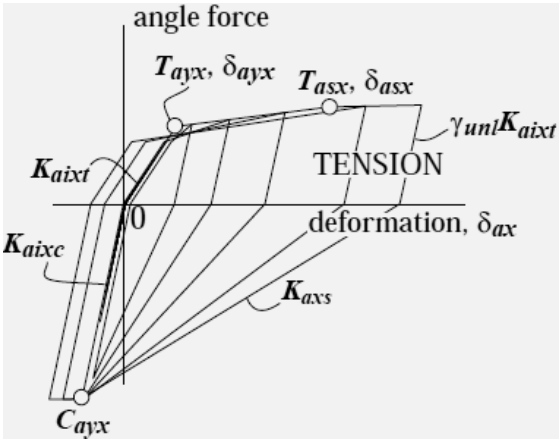

Figure (2) . Hysteretic behavior of seat angles (Quick, Kurama, and Weldon 2006).

3. Reference Bridge Pier

The one-quarter scaled conventional RC bridge pier with flexural-shear behavior and experimental static cyclic results is selected as a reference bridge (Institute of Earthquake Engineering and Engineering Seismology report 1984). The conventional bridge pier is modeled analytically to compare with proposed bridge pier.

The geometric as well as mechanical specification of the bridge pier are; as follows: diameter and length of pier respectively are 307 and 895 mm, cover to center of hoop bar is 36 mm, the axial force on the pier is 10% of the section capacity, longitudinal reinforcement ratio is 1.83%, diameter of spirals are 6 mm with spacing of 75 mm, compression strength of unconfined concrete is 34.4 Mpa and yield strength of longitudinal and transverse steel are 240 Mpa.

3.1. Modeling of reference bridge pier

The 2-D finite element model of the reference bridge pier as well as the test setup is developed and analyzed using the open-source finite element program (OpenSees) (Mazzoni et al. 2007). The plastic hinge approach with Clough material which was developed and implemented in the OpenSees material library is utilized for analytical modeling. Moment and rotations are obtained according to the bending capacity of column and flexural-shear interaction respectively.

4. Proposed Bridge Pier

Proposed bridge pier is allowed to rock by making the rebar discontinuous at the column ends. The pier is post-tensioned vertically with unbounded cable in order to control the lateral strength and decrease residual drift. For better comparison, geometrical dimensions and other mechanical properties of the proposed model material is considered to be the same as the conventional pier. Four external seat angles are connected to steel jacket to enhance energy absorption in the controlled rocking zone. Steel jacket is used at the end of pier for making better connection between seat angles and pier and also improve rocking behavior by confining rocking zone. In order to reach minimum residual drift and less damage, instead of seat angles, pier must remain elastic. Thus, dimension of seat angles have to be limited to the point where the yielding of the fuses takes place earlier than pier. The post-tensioning force is set to be less than 40% of the yield force of the post-tensioned cable. The yield strength of the cable and seat angles are assumed to be 1792 and 352 Mpa respectively and area of post tension cable is assumed to be 1 cm2. In this study, dimension of seat angles are chosen to be two sizes of L200*120*19 and L200*120*14 with length of 150 mm. Post-tensioned ratio, being cable force divided by section capacity, is chosen to be 0, 1, and 3 %. The scheme of proposed pier is illustrated in Figure (3).

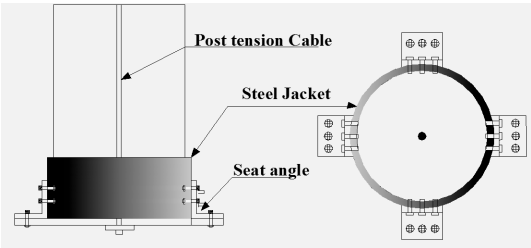

Figure (3). Scheme of proposed rocking pier.

4.1. Modeling of proposed bridge pier

Modeling of proposed bridge pier is carried out using OpenSees. The segment of pier without steel jacket is modeled using nonlinear beam column element which was separated into steel, confined and unconfined concrete fibers. The segment of pier with steel jacket is also modeled using nonlinear beam column element which was separated into steel, confined concrete fibers. In order to consider the effect of steel jacket confining in whole of section, rocking zone is modeled using zero length element section with concrete material. Hysteretic behavior of seat angle as shown in Figure 2 is modeled with hysteretic material in OpenSees. Horizontal shear is assumed to transfer between connections of column to foundation because seat angles are sufficient for this means. The post-tension cable is modeled with truss element using initial strain material that exists in OpenSees library.

5. RESULTS

Results clarifies that energy absorption in the proposed bridge pier is less than the conventional bridge pier when subjected to static cyclic loading. However, the other structural elements remain elastic in the new bridge pier. Although, increasing the thickness of seat angles results in more energy absorption, it leads to implying the nonlinear behavior of the pier. On the other hand, using L200*120*19 seat angles as energy absorption devices causes to increase the ductility capacity to9. In Figure 4, cyclic behavior of rocking piers using L200*120*19 seat angle without cable and conventional bridge pier are shown in which self-centering behavior of rocking piers are obviously observed.

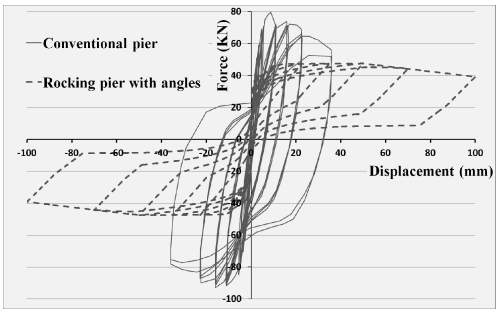

Figure (4). Cyclic behavior of rocking pier using L200*120*19 seat angles and conventional pier.

Cyclic behavior of rocking piers using L200*120*19 seat angles under axial load ratio of 5% with post-tensioning force ratio of 3% and without post-tension cable are illustrated in Figure 5. As shown in Figure 5 lateral strength of rocking pier is increased by presence of post-tension cable.

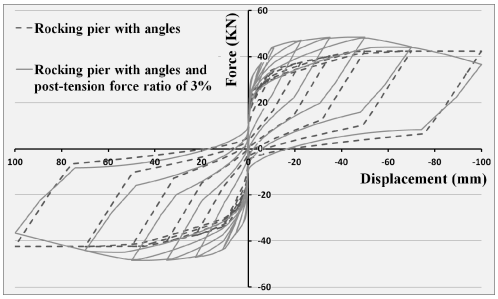

Figure (5). Cyclic behavior of rocking pier using L200*120*19 seat angles and rocking pier with angles and post-tension cable.

In order to verify the actual behavior of the proposed bridge pier, bridge pier using L200*120*19 seat angle with 3% post-tensioned load ratio is compared with the conventional bridge pier. To that end, a nonlinear dynamic analysis is conducted using seven far-field ground motion records

selected from the PEER NGA database. Records are selected to have magnitude, PGA, and PGV greater than 6.5, 0.2g, and 15 cm/s, respectively. Chosen records contain Duzce in Bolu station, Kobe in Shin Osaka, Kocaeli, Imperiall Valley in El Centro Array #11, Northridge in Beverly Hills-Mulhol, Landers in Yermo Fire and Chi-Chi in TCU045In order to one-quarter scaling factor of conventional and proposed pier, times of records are scaled to 0.5.

Time history results of conventional and rocking pier in Duzce and Kobe records are illustrated in Figure 6 a & Figure 6 b respectively.

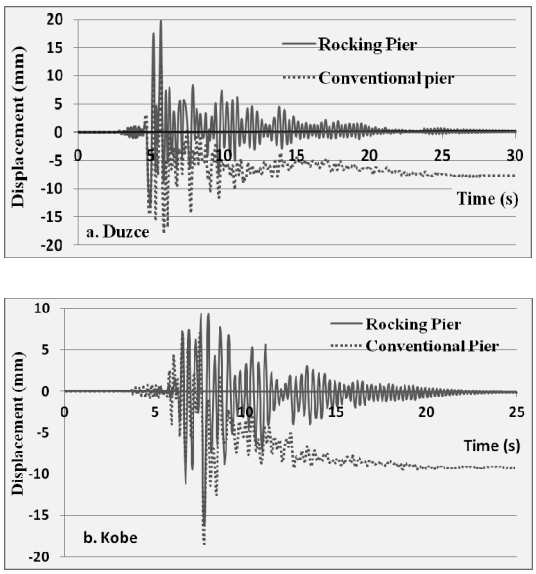

Figure (6) . Time history results of conventional and rocking pier in scaled, (a) Duzce, (b) Kobe records.

Time history results indicate that residual displacement in proposed bridge pier is much less than conventional one.

6. Discussion

Large lateral displacement capacity (9) of the proposed bridge pier is the major advantage in comparison with conventional one (5). As shown in Figure 4, cyclic behavior of rocking pier demonstrates stable cyclic response and reliable energy dissipation capacity in which strength degradation does not occur. Also, decreasing base shear force that leads to constructing weak foundation is the most crucial benefit. The lack of structural damage in other parts of pier associated with large displacements is very important. The external seat angles have the advantage of being easily changed and, hence, cost of repairing and retrofitting is minimized after an earthquake. The dimension of seat angles should be designed according to capacity of pier; hence, other parts could be elastic and no damage is occurred in other parts of pier.

In accordance with time history results no residual drift is observed in rocking pier and this advantage insures serviceability after an earthquake.

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مقایسه کارآیی پسماندهای پرچم شکل در برابر زلزله و اسکله‌های پل‌های سنتی

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چکیده

زلزله‌های اخیر نیاز دنیا به یک فلسفه طراحی جدید جهت مقاوم‌سازی شمع‌های پل‌ها را ضروری کرده تا بتوان با جلوگیری از آسیب‌های وارده قابلیت سرویس‌دهی پس از زلزله و کاهش خسارات مالی را تضمین نمود. سیستم‌های خودمرکزی و گهواره‌ای از جمله روش‌هایی هستند که جابجایی باقیمانده را حذف، سرویس‌پذیری پس از زلزله را حفظ و احتمال تخریب پس از زلزله را کاهش می‌دهند. هدف از این مطالعه ارزیابی عملکرد لرزه‌ای خود مرکزی شمع‌های پل‌ها می‌باشد که مواردی مثل جذب انرژی را با نمونه‌های قدیمی می‌کند. در این رابطه مجموعه‌ای از تحلیل‌های غیرخطی دینامیکی و استاتیکی روی یک پل معمولی انجام شده و معیارهای طراحی مهم بررسی می‌شود.

واژه‌های کلیدی: *خود مرکزی، اسکله پل، زاویه جایگاه*

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