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Novel Rocking Pier on Pile Foundation

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ABSTRACT

Considering the current scenario in which we see major earthquakes where the structures are damaged severely beyond repairs, we need such a foundation which shall suffer only minimal damage. This paper proposes novel rocking pier with elastomeric pad installed at top of pile cap along with external restrainers made up of shape memory alloy (SMA) bars. The effect of pile soil interaction along with ground response analysis is also incorporated in the individual pier model adopted for the study. The earthquake waves amplify at the ground surface and the response of soil profile was analyzed using a 1D equivalent site response program. By performing non-linear dynamic time history analysis, a comparison with a fixed based pier at pile cap top and the novel rocking pier is been studied. It has been found that the proposed pier has good recentering capability and better performance during seismic than the conventional pier adopted in current design practice.The novel rocking pier experienced lesser forces and negligible permanent drifts due to its controlled rocking behaviour by using smart material in the form of SMA bars.

KEYWORDS: Resilient Pier; Shape Memory Alloy; Elastomeric pad; Bridge pier

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I. I.INTRODUCTION

The concept of ductility is used in the conventional design of bridge pier wherein the pier reinforcement is detailed to develop flexural plastic hinges at the base and top of pier. Although bridges designed in this manner may be safe from collapse but not from damages due to severe earthquake excitations as seen in Fig1. Rocking isolation in the form of structural rocking or geotechnical rocking of the bridge pier experience far less damage when subjected to high intensity earthquake ground motion with added bonus of pier that recenter due to the increased period of vibration owing to the flexibility of the resilient pier. Antonellis and Panagiotou (2014) investigated the three-dimensional seismic response of conventional fixed based pier and piers rocking on pile foundations that are designed to remain elastic. The rocking of pile foundation was achieved by wrapping the protruding part of piles into the pile cap by neoprene sheet and rubber pad. Marriott et al (2011) experimentally investigated biaxial response of post-tensioned precast bridge pier with external replaceable mild steel dissipaters which maintains the structural integrity during severe biaxial loading and thus reduces the repairs cost by simple replacement of the external dissipaters. Quinn et al (2016) modeled soil-structure interaction by using nonlinear discrete Winkle springs and adopted force-deformation (p-y) curves to model soil springs as per American Petroleum Institute Standard (API 1993). Boulanger et al (1999) validated nonlinear Winkler foundation analysis method for analyzing seismic soil-pile interaction by means of results of a series of dynamic centrifuge model tests. For Free-field site response analysis a 1D equivalent linear site program SHAKE 91 was used.

This paper compares a fixed base pier at pile cap top having a soil profile varying from loose sand to dense sand as depicted in Brahmaputra River near Guwahati with a novel rocking resilient pier having elastomeric pad above the pile cap and external restrainer (SMA) attached to footing over the pads. Considering amplification of waves from the bed rock to the ground surface and by performing nonlinear dynamic time history analysis it has been found the novel rocking resilient pier is a better alternative for areas subjected to high seismic. By isolating the pier from the pile cap the forces in pile and pier are considerably reduced and by using special smart material like shape memory alloy bars the proposed pier is subjected to negligible residual drifts and thus proves to be an economical and robust solution for bridges located in server seismic zones.

Fig 1 Recent failures of Bridges

II. ROCKING RESILIENT BRIDGE PIER ON PILE FOUNDATION

2.1 Description

The word *"Resilient"* means the one which can recover itself from difficult conditions. The resilient bridge foundation proposed in this paper uses low damage materials which is shape memory alloy (SMA) bars and elastomeric pads. The elastomeric pad is placed at the top of pile cap and by means of horizontal stiff arms the SMA bars are connected to the base of pile cap as shown in Fig 2& Fig 3. The footing simply rests on these pads. The gap between stopper and footing over pile cap is kept minimum so that horizontal sliding of the resilient foundation is avoided and thus only vertical uplift is allowed. In practice, a suitable cushion in the form of rubber pad maybe inserted in this gap so the pounding of two concrete surfaces is avoided.

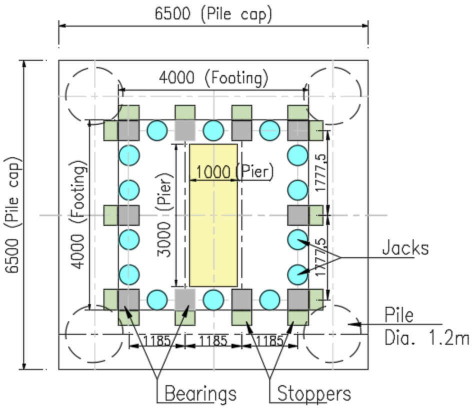
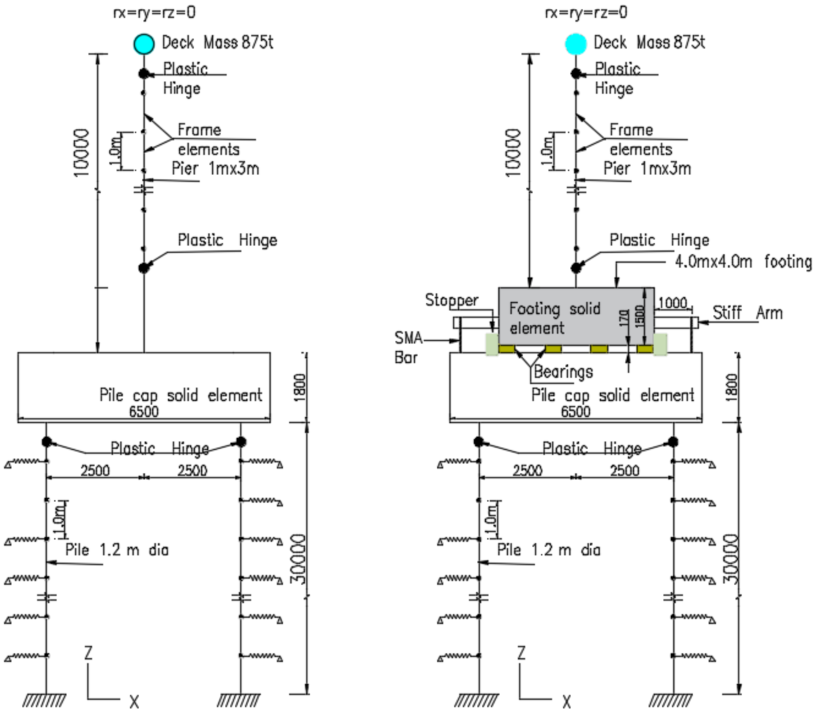


Fig 2 Plan of pile cap for novel rocking resilient pier

2.2 Numerical Modelling The novel rocking pier bridge foundation is modelled in *CSi Bridge 2017*(Computers and Structures, Inc. 2017) and the results are compared with the conventional monolithic pier considered in current design codes as per European practice. A nonlinear stage construction case was defined wherein SMA bars were installed after the deformation of pads due to self-weight of pier, footing and dead load of the superstructure. The nonlinear time history analysis was performed using the stiffness at end of this nonlinear stage construction case. The dead load mass of the superstructure having area of deck as 9.85m² and superimposed dead load of 4.55 t/m for a span of 30 m was assigned as concentrated load of 875 t on the pier top which was activated during the construction stage analysis performed in the computer programme. For Fixed and the proposed novel rocking pier, the pier top was considered to be monolithically connected to the deck and to achieve the pier fixity at top, the rotations about transverse axis is restrained for all cases while the pier top is free to move along all axes.

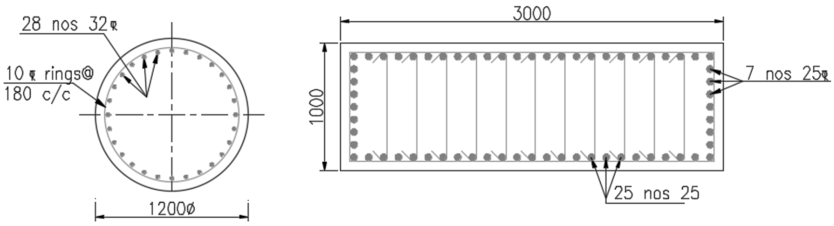


(a) (b)

Fig 3 Finite element model a) Conventional Pier b) Novel Rocking Resilient Pier

2.2.1 Material and Geometry

A conventional monolithic wall type pier of size 1m x3m having height of 10m with pile cap of 6.5mx6.5m and depth of 1.8m with spacing of piles being 5m in both directions is considered in this study. The novel rocking resilient pier has the same dimensions as the fixed one along with footing of size 4mx4m considering the space required for jack to replace the pads placed along the periphery of the footing. The grade of concrete for fixed base pier, pile cap, novel rocking resilient pier and footing is M-35 while it is M-45 for piles of 1.2m diameter. The reinforcing steel for pile is 2 percent of the cross-section area of pile and while for pier it is 1% percent of the cross-section area of pier as shown in Fig.3 (a) and Fig.3 (b)respectively. The idealized moment curvature graph is obtained by balancing the areas between the actual and the idealized M-Ø curves beyond the first reinforcing bar yield point as per Eurocode 8.



(a) (b)

Fig .3(a) Pile cross-section (b) Pier cross-section

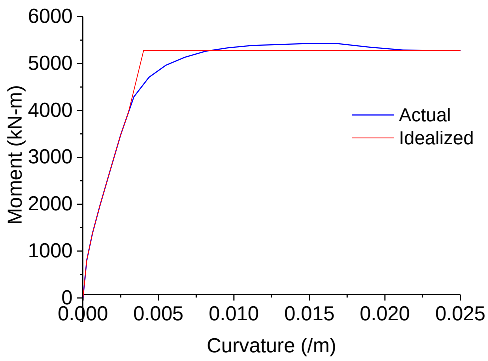


Fig 4 Moment-curvature relationship for pile with 2 percent steel

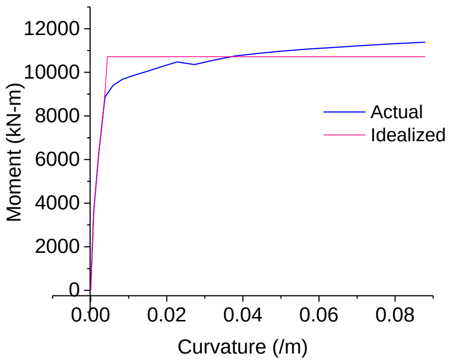


Fig 5 Moment-curvature relationship for pier with 1 percent steel

2.2.2 Nonlinear modelling

To stimulate post-yield behaviour of piers and piles, a concentrated plastic hinge is assigned to the frame element of the pier and piles. To capture the coupled axial and bending behaviour, P-M3 hinge is assigned to the piers at relevant locations with the input being the moment-curvature graph (Fig.4&5) and the hinge length of 1.15 m for pier and 1.11m for pile is calculated as per equation given in Priestley (1996).The confinement of concrete has been taken into account using Mander confined model (Mander et al 1988) to represent the stress-strain behaviour of the concrete core. The model conventional pier and therocking pier are shown in Fig.3 in which the pier is modelled with 10 frame elements and pile with 30 frame elements. The pile cap is modelled with 240 solid elements and footing resting on pile cap for resilient pier is modelled with 100 solid elements.

2.2.3 Elastomeric pad

A total of ten pads with distributed along the footing periphery are modelled using compression only links available in the *CSi Bridge*. The size of bearing is 425mm x425mm with height of 170mm which consists of five rubber layers of 30 mm thickness and four steel shims of 5mm thickness was selected after the design of pad was made as per Eurocode EN 1337-Part 3. This particular size of bearing was chosen to avoid any large uplift of pads and also to control very large vertical initial compression of pad. These pads shall be specially made to suit the rocking behaviour of the bridge foundation.

The vertical (k_v) and horizontal stiffness (k_h) of bearing is evaluated using following equations (Naeim and Kelly 1999):

$$k_v = \frac{E_c.A}{\sum t_i}; \quad (1)$$

$$k_h = \frac{G.A}{\sum t_i}; \quad (2)$$

$$E_c = \frac{Ec'.B}{Ec'+B}; \quad (3)$$

$E_c' = 6.73\ GS^2$ (4)

Where, G is the shear modulus of the bearing having value of 0.7 MPa, B is the bulk modulus as 2000 MPa and S is the shape factor which for square pad is a/4t where "a" is size of rubber pad and t is the thickness of each pad. The initial vertical deformation of pad with superstructure load is found to be 14.4 mm and the stress in pad is found to be 4.57 MPa against permissible stress of 10 MPa as per Eurocode EN 1337-Part 3

2.2.4 Shape memory alloy bars

SMAs are unique materials with a paramount potential for various application in bridges. The novelty of this material lies in its ability to undergo large deformations and return to its undeformed shape through stress removal (super elasticity) or heating (shape memory effect). The parameters to model shape memory alloy bars are shown in Table 1. These properties are adopted from DesRoches and Delemont (2002). By using these stress-strain values, an analytical model in *CSi Bridge* is created to have an idealized force-deformation curve of 50 mm diameter SMA bar as shown in Fig 6 which shows the primary features of the superelastic effects of shape memory alloy bars. The SMA bars are modelled using a double link element. The first element is multi linear plastic element (PLE) using Pivot model (Dowel et al 1998) to define the hysteresis loop and a multi linearelastic (MLE) link is used to shift the hysteresis loop away from the origin. Three shape memory alloy bars of diameter 50 mm composed nickel and titanium (Ni-Ti) alloy was chosen on each side of the footing and was connected to footing by stiff arms. The length of SMA is measured from the centre of footing depth to the concrete base and is calculated considering the initial deformation of the pad. The SMA bar is to be installed after initial deformation of pad due to the dead load deck and self-weight of pier.

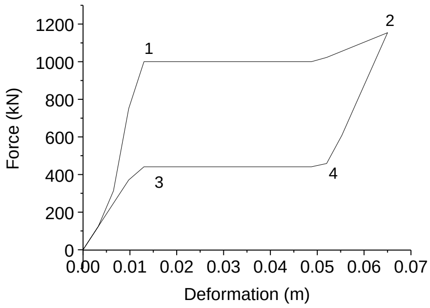


Fig. 6 Analytical model of SMA restrainer of 50mm diameter and unit length used in CSi Bridge

Table-1 Constitutive material properties for NiTi based SMA bar

Parameters	Value
Austenite to martensite starting stress (1)	523 MPa
Austenite to martensite finishing stress (2)	588 MPa
Martensite to austenite starting stress (4)	241 MPa
Martensite to austenite finishing stress (3)	225MPa
Yield strain	1.1%
Recoverable pesudo elastic strain	6.2%

III. PILE SOIL INTERACTION

A typical soil profile of Guwahati city is adopted from Dammala et al (2017) as shown in Table 2 in order to see the effect of pile soil interaction for the proposed resilient pier pile foundation. Only lateral soil resistance is considered in the present analysis while the axial pile soil interaction which is on conservative side is ignored in the present study. To simulate lateral soil resistance along piles nonlinear springs were attached at 1 m interval for a 30m long pile. The force-deformation (p-y) curves (Fig 7) for these soil springs were defined using the hyperbolic tangent method as described in the American Petroleum Institute Standard (API 2000) as

$$F = A\ X\ p_u X\ tanh\left(\frac{k.z.y}{A.p_u}\right) X\ L \quad (5)$$

Where A= empirical correction factor =0.9

k= Soil Modulus; Z= soil depth from top of soil layer to specified node; y= horizontal deflection and L= length of pile section.

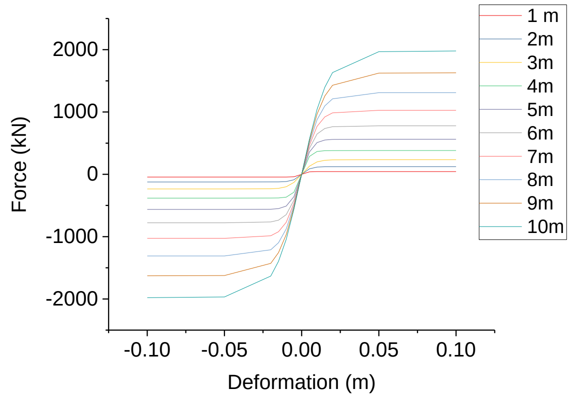


Fig.7 Force deformation curve for pile soil spring at first 10 m depth

Table- 2 Soil profile in Guwahati City

Soil type	Depth,m	SPT, N	V_S,m/s	ϒtotal,kN/m³
Loose sand	1-3	4	149	15.1
	4-11	8	178	15.1
Medium sand	12-13	12	211	16.2
	14-18	15	226	16.5
	19-25	24	262	17.8
Dense sand	26-30	31	284	19.4

IV. SELECTION OF EARTHQUAKE EXCITATION

The fixed pile foundation and resilient pier pile foundation were analysed for two real accelerograms compatible to ground Type 1- C-dependent Eurocode 8 elastic spectra. The Peak Ground Accelerations (PGA) selected were 0.2g(Chi-Chi) and 0.35g (Loma Gilroy) which act as input motion at the outcrop in the computer program DEEPSOIL V6.1 to study the amplification of PGA for a 30m deep soil. The response spectra of the analysed accelerograms are shown in Fig.8.

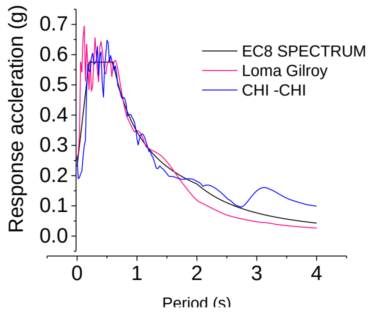


Fig. 8 Response spectra of accelerograms compatible to ground Type C-dependent Euro code 8-1 elastic spectra (PGA = 0.20g).

4.1.1 Free field Ground Response Analysis

For amplification of seismic waves, Ground Response Analysis (GRA) has been performed first using one-dimensional (1D) equivalent linear response program DEEPSOIL V6.1 (Hashash et al 2015). The effect is then considered in the pile as multiple-support excitation problem in which displacement is given at each pile soil spring node in the form of displacement time history obtained from the acclerograms analyzed in DEEPSOIL. Only excitation in longitudinal direction of the bridge is considered in this study. The Fig 9 and Fig10 shows the PGA and relative displacement amplification for two different earthquakes namely Chi-Chi (0.2g) and Loma Gilroy (0.35g)

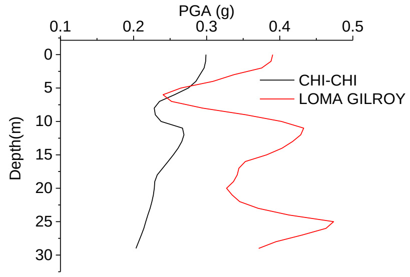


Fig. 9 Amplification of PGA for Chi-Chi and Loma Gilroy

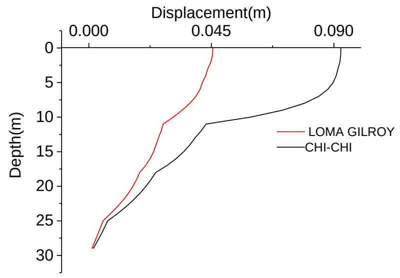


Fig.10 Amplification of relative displacement for Chi- Chi and Loma Gilroy

V. RESULTS AND DISCUSSION

It can be seen from Tables 3&4 where the mean values of responses are calculated based on the acceleration time histories for PGA of 0.2g and 0.35g for medium and high seismic excitations that novel rocking resilient pier and its pile experienced smaller bending moments than the fixed pier and its pile. The resilient pier had good reentering characteristics as the residual drift for both the PGAs is found to be negligible. It has been found that there is substantial reduction (about 47% for 0.2g and 41 % for 0.35g) in B.M in pier when resilient pier pile foundation is adopted. The footing for the resilient foundation did not experience any uplift due to presence of external restrainers. An interesting fact that is found is the reduction of horizontal displacement at pier top when fixed base pier is compared to resilient pier due to the fact of using higher diameter of external restrainers which controls these displacements.

The time history graphs (Fig 11 to 14) is provided only for one of the PGA of 0.2 g as the relative displacement for this is more than the other PGA of 0.35g as per the Ground Response Analysis.

The Fig. 15 shows the superlastic behaviour of SMA bars in which the process of yielding and successive increase in stiffness allow SMA bars to dissipate large amount of energy and reduce the pier displacements when compared to the fixed pile model. Also, it can be seen that restrainers do not have symmetrical response for the where the right restrainers (SMA bars) resist more force than the left ones because the earthquake pulses are not symmetrical. Note that the bars only act in tension since they have been designed for the same as no response in compression zone is observed.

Table-3 The maximum values of the seismic loading for the two pier models for PGA 0.2g

PGA 0.2 g		
	Fixed Pier	Resilient Pier
Ux Pier(mm)	112	46
Residual Drift (%) for pier	0.196	0.018
Footing Uplift (mm)	0	0
B.M Pier bottom(kN-m)	13910	6553
B.M Pile top (kN-m)	4295	3333

Table-4 The maximum values of the seismic loading for the two pier models for PGA 0.35g

PGA 0.35 g		
	Fixed Pier	Resilient Pier
Ux Pier(mm)	72.5	42
Residual Drift (%) for pier	0.0640	0.007
Footing Uplift (mm)	0	0
B.M Pier bottom(kN-m)	12810	5306
B.M Pile top (kN-m)	3699	3650

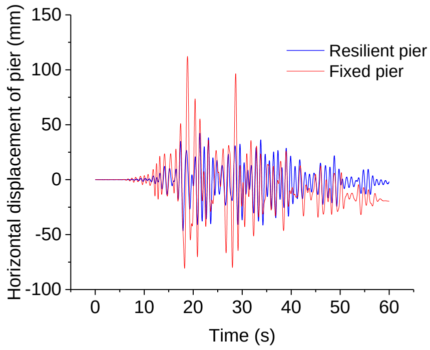


Fig 11Time history of horizontal displacement at pier top for real acceleration of Chi-Chi (PGA 0.2g)

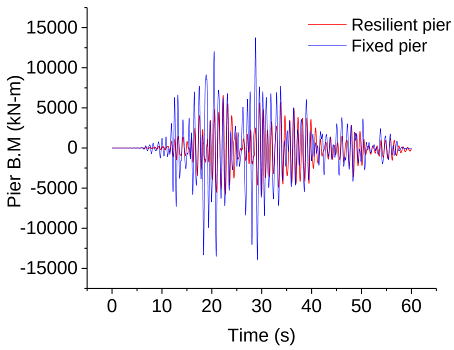


Fig 12Time history of B.M in Piers for real acceleration of Chi-Chi (PGA 0.2g)

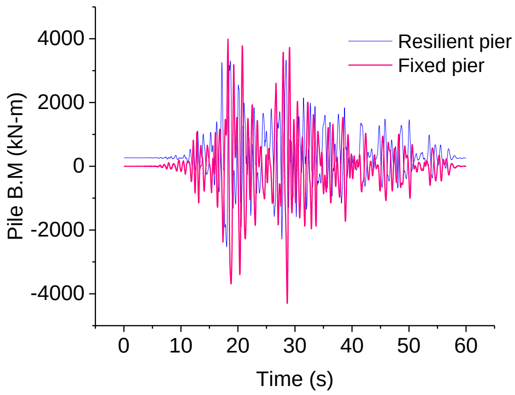


Fig 13Time history of B.M in Piles for real acceleration of Chi-Chi (PGA 0.2g)

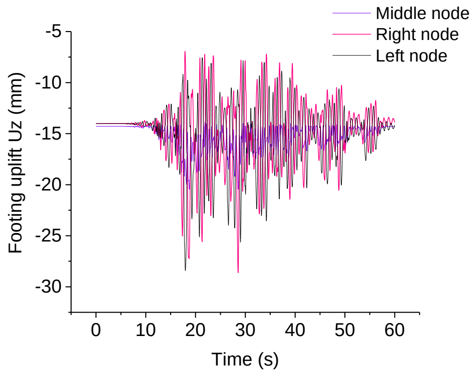


Fig 14Time history of vertical displacement of footing for resilient pier for real acceleration of Chi-Chi (PGA 0.2g)

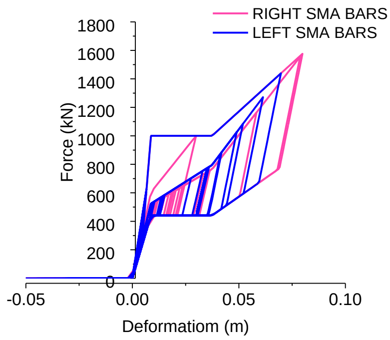


Fig 15Force-displacement curve for SMA bars in resilient pier for real acceleration of Chi-Chi (PGA 0.2g)

VI. CONCLUSIONS

This paper proposes a novel rocking resilient bridge pier resting on pile foundation which using elastomeric pads at the top of pile cap and shape memory alloy bars as restrainers to promote controlled rocking of the system. The rocking mode in pier is also promoted by restricting the horizontal movement by means of stoppers around the footing. The rocking resilient pier has been compared with the fixed pier model on basis of displacements and forces in piers/piles which are subjected to horizontal seismic motion in the longitudinal direction of bridge which is more critical than the transverse direction for wall type piers taking into account the pile soil interaction and ground response analysis. Based on the analysis performed the following conclusion were drawn when pile is in sandy strata:

1. The novel rocking resilient pier pile foundation has better seismic performance than the fixed pile models since the elastomeric pad provides flexibility while the SMA bars showed superelastic hysteresis and controlled the pier displacements.
2. Rocking pier is cost-effective solution since the pier and pile forces are reduced as compared to the fixed model. Alternatively, sizes of pier can be reduced when concept of rocking is adopted especially in high seismic zones which can been seen from moment curvature graphs where the yield value of B.M is less than the value obtained from the analysis as shown in tables 3&4.
3. The rocking pier had almost full recentering capacity with negligible residual drifts for 0.35g and 0.2g seismic excitations.
4. Overall, it can be concluded that the proposed novel rocking bridge pier on pile foundation is promising and advantageous especially for areas prone to medium and high seismic zones. Furthermore, the seismic demand observed in the rocking pier is far less than the conventional bridge pier on pile foundation adopted in the current practice of various bridge design codes.

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