Seismic analysis of an Integral Bridge with retrofitted RC pile foundation in different foundation soils using simplified SSI

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ABSTRACT: Retrofitting of Reinforced Concrete (RC) pile foundation below integral bridges is an important issue concerning functionality of such bridges in earthquake-prone regions. In the present study, the numerically investigated retrofitting method comprises addition of micropiles along the periphery of the pile caps with jet grouting around each new pile. Initially, the original bridge response is compared with the response of the bridge strengthened at abutment foundation only. Subsequently, retrofitting of pile foundation at specific pier locations is considered along with the abutment foundation strengthened earlier. Under a site-specific ground motion, bridge response for the two cases of strengthening are compared with the response of the original bridge model. Additional RC micropiles around the pile cap, embedded in jet grouting, tend to reduce the extent of nonlinear behavior of bridge piers and foundation piles. The lateral bearing resistance of the grouted soil medium also gets enhanced during possible earthquake shaking.

1 INTRODUCTION

In the seismic design of bridges with pile foundation, the foundation behaviour at the abutment and the pile locations is an important issue in view of possible nonlinear behaviour during strong earthquake shaking. Along with extensive nonlinear behaviour of the foundation, large displacements at deck level can lead to seismic failure of the overall bridge structure as observed in the 1995 Kobe earthquake. For pile foundation incurring seismic damages, retrofitting of pile foundation becomes an important issue. One of the possible retrofitting methods for pile foundation is the "encasement" method which has been derived from "incremental capacity method". The encasement method was first adopted by Japanese engineers in severe seismic zones (Level II earthquake shaking) with comparatively softer soil (Fukada et al. 2005; Wang, 2015). In that retrofitting technique, additional micropiles are installed around the existing pile cap up to a target design depth. Then, the soil underneath and around the pile cap is replaced by engineered grout through high-pressure jet grouting. Once the substrate gets solidified, the new piles and injected grout act as a monolithic unit. This method provides lateral stability to the pile group foundation by enhancing the passive soil pressure during earthquake shaking. Also, additional resistance to possible overturning moment is mobilized through soil-pile friction. The mentioned retrofitting technique is suggested to one of the cost-effective solutions for seismic retrofitting.

In the present study, the encasement method of retrofitting is implemented for pile foundation below an integral bridge with the bridge geometry considered from Humboldt Bay Middle Channel Bridge (Elgamal et al. 2008; Zhang et al. 2008) located in California USA. The soil-bridge interactions, namely the soil-pile interaction and abutment-backfill interaction, have been modelled using suitable springs and dashpots. The influence of the inertial effect of the bridge on abutment backfill interaction (Zhang & Makris, 2002) is not considered in the present study. Four different types of foundation soil are considered below the bridge. In the first stage of analysis, the new micropiles encased in jet grouting are modelled only below the abutments and the bridge response is compared with the response of the original bridge model without the new micropiles. Subsequently, micropiles encased in jet grouting are added at two specific intermediate pile groups below piers. For both the stages, the nonlinear response of individual pile gets reduced below the abutments and the piers. This further leads to the reduction of nonlinearity in piers as well as deformation at the deck level. To the best of the authors' knowledge, numerical investigation of the encased retrofitting method using the spring-dashpot model for soil-grout behaviour has not been carried out previously. Detailed parametric studies involving multiple ground motions and other bridge parameters can be carried out separately in future.

2 MODELLING OF STRUCTURE AND FOUNDATION

The original Humboldt Bay Middle Channel Bridge consists of nine spans with the total length of the bridge as 330 m (Elgamal et al. 2008; Zhang et al. 2008) as shown in Figure 1. The entire bridge modelling has been carried out using the structural analysis program SAP2000 *v*.19.2.1 (CSI, 2016). Piers and piles are modelled as nonlinear beam-column elements. Deck girders and cap beams are modelled as linear elastic beam-column elements. Abutment walls, pile caps and deck slab are modelled as linear thin shell elements. The deck slab is supported on four I-shaped concrete girders with all the girders rigidly connected to the piers by cap beams. The length, width and height of each abutment are 10 m, 1.2 m and 5 m, respectively. Each abutment is connected with foundation through 12m×3m sized and 0.7 m thick pile cap. Piers are supported on pile caps with plan area 7m×7m and a thickness of 1m. The abutment foundation consists of 18 nos. RC piles (of 0.6 m diameter) in double rows and each pier foundation consists of a pile group of five precast driven RC piles of 1.372 m diameter. Piers are of 2m×1.7m in cross section. The reinforcement details and fibre modelling of the components are discussed in previous studies (Dhar, 2018; Dhar & Dasgupta, 2018). The behaviour of cover and core concrete are modelled using Mander's model (1984). The lengths of the pier piles and abutment piles are varied in different types of soil depending on the properties (Prakash & Sharma, 1990). The soil resistance in pile foundation against various mechanisms is modelled using 2-noded spring elements. The Soil-Pile interaction (SPI) and the Abutment-Backfill Interaction (ABI) are considered as per API (2000) guidelines and BA 42/96 (2003) provisions respectively. At the restrained end of soil links Displacement Time Histories (DTHs), obtained from the site-specific ground motion at bedrock level, are applied. The dynamic analysis is carried out for the four types of foundation soil (Table 1), namely (a) medium stiff clay, (b) soft clay, (c) dense sand and (d) loose sand for the mentioned models.

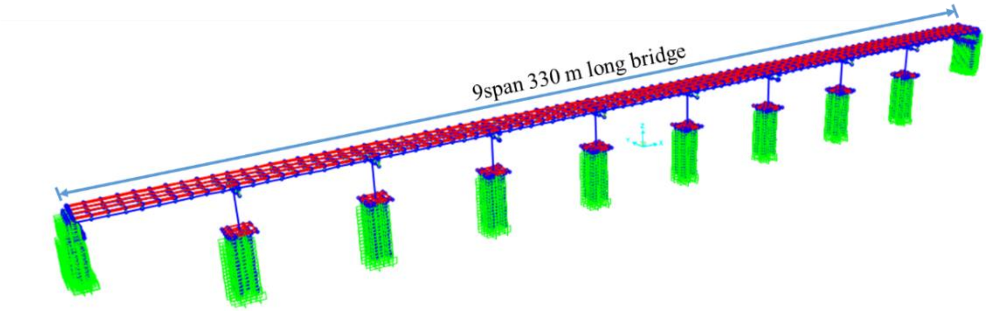

Figure 1. Integral bridge model with spring-dashpot modelling

Table 1. Soil properties used in the present study (Dhar, 2018)

Soil Type	Maximum shear modulus (MPa)	Poisson's ratio	Total unit weight (t/m^3)	Cohesion (MPa)	Internal friction angle
Medium stiff clay[1]	76	0.45	1.9	0.027	-
Loose sand[2]	55	0.45	1.6	-	29°
Soft clay	9	0.45	1.75	0.018	
Dense sand[2]	224	0.45	2.0		38°

[1] after Zhang et al. (2008)

[2] after Erhan & Dicleli (2017)

3 RETROFITTING OF RC PILE FOUNDATION

The present study focuses on numerical modelling of additional RC piles surrounding the pile cap and embedded in jet grouting which can enhance lateral resistance of the soil-grout medium during earthquake shaking. The physical properties of jet grouting are considered as per the study by He et al. (2016).

In the first stage of the present study, the encasement method is incorporated by encasing RC piles in jet grouting around the periphery of the pile cap below each abutment. From the NTHA of the original 9 span integral bridge model under GM#1, it is observed that the bending moment diagram in the abutment piles shows significant moment up to a depth of 4 m from the ground level. Below the depth of 4 m from the soil surface, mostly linear elastic behaviour of the piles is observed. So, the length of the new encased micropiles is considered as 4 m for the abutment pile foundation. The new encased piles are modelled with a minimal diameter of 0.4 m considered for cast-in-place driven RC piles. Henceforth, the original bridge model before retrofitting (Fig. 1) and the bridge model retrofitted only at abutment foundation, will be referred to as M1 and M2 models, respectively. The schematic diagrams of the retrofitted abutment foundation for M2 model are shown in Figures 2a–c. In the numerical model of the encased new piles, the spring-dashpot elements representing the lateral strength, lateral stiffness and the damping characteristics of the grout are modelled in a series configuration with the soil spring-dashpot elements. The effect of radiation damping due to far-field energy dissipation and participation of soil mass during dynamic interaction (Carbonari et al., 2017) have not been considered in the present study. Thus, the level of soil-pile response and the resulting nonlinearities are estimated conservatively. The lateral force - lateral displacement characteristics for jet grouting, are taken from the study by He et al. (2016).

For the M2 model, the maximum bending moments are observed in the piles below the 3rd and the 6th piers. Like the extent of the significant bending moment diagram observed in the original abutment piles, the pier piles are also subjected to significant bending moment till a depth of 4 m during NTHA under GM#1. So, the length of new encased piles is kept as 4 m. However, the diameter of the new encased piles is considered as 0.8 m. To integrate the new encased piles with the original pile cap, the original pile cap is extended (Wang, 2015) equally along both the directions from the size of 7m×7m to 8m×8m in plan. The schematic diagram of the retrofitted pier pile foundation is shown in Figure 3a. Henceforth, the bridge model with retrofitted abutment and pier pile foundations will be referred to as the M3 model.

4 ANALYSIS

To investigate the effect of encasement method on the bridge response, NTHA is carried out for the 9 span integral bridge with the models M1, M2 and M3 only under a site-specific ground motion (Dhar et al., 2016), which is denoted as GM#1 (Fig. 3b–c). In the first stage of analysis, the dynamic response of M1 and M2 models are compared and in the second stage of the analysis, the response of M2 and M3 models are compared. In Table 2, the natural periods of the M1 and M2 models in four different types of foundation soil are compared. The comparison reflects the higher stiffness of the M2 model as compared to the M1 model for all the types of soil.

5 RESULTS

5.1 *Comparison between M1 and M2 models*

In Figure 5, the moment-rotation response of the original abutment piles and the pier piles at a depth of 1.25 m from soil surface are compared for M1 and M2 models. The moment-rotation response gets reduced for M2 model in all the types of soils except dense sand for abutment piles (Figs. 4a, c). For M1 and M2 models, in medium stiff clay and dense sand, pier piles undergo extensive nonlinearity (Figs. 4b, d)). In soft clay, the response in the pier piles and the abutment piles get reduced in the M2 model as compared to the M1 model. Although in loose sand, the pier piles are observed to form nonlinear loops, the moment-rotation response is similar in magnitude for both M1 and M2 models. The reduced response in the retrofitted foundation imposes lesser seismic deformation demand on the bridge substructure and superstructure.

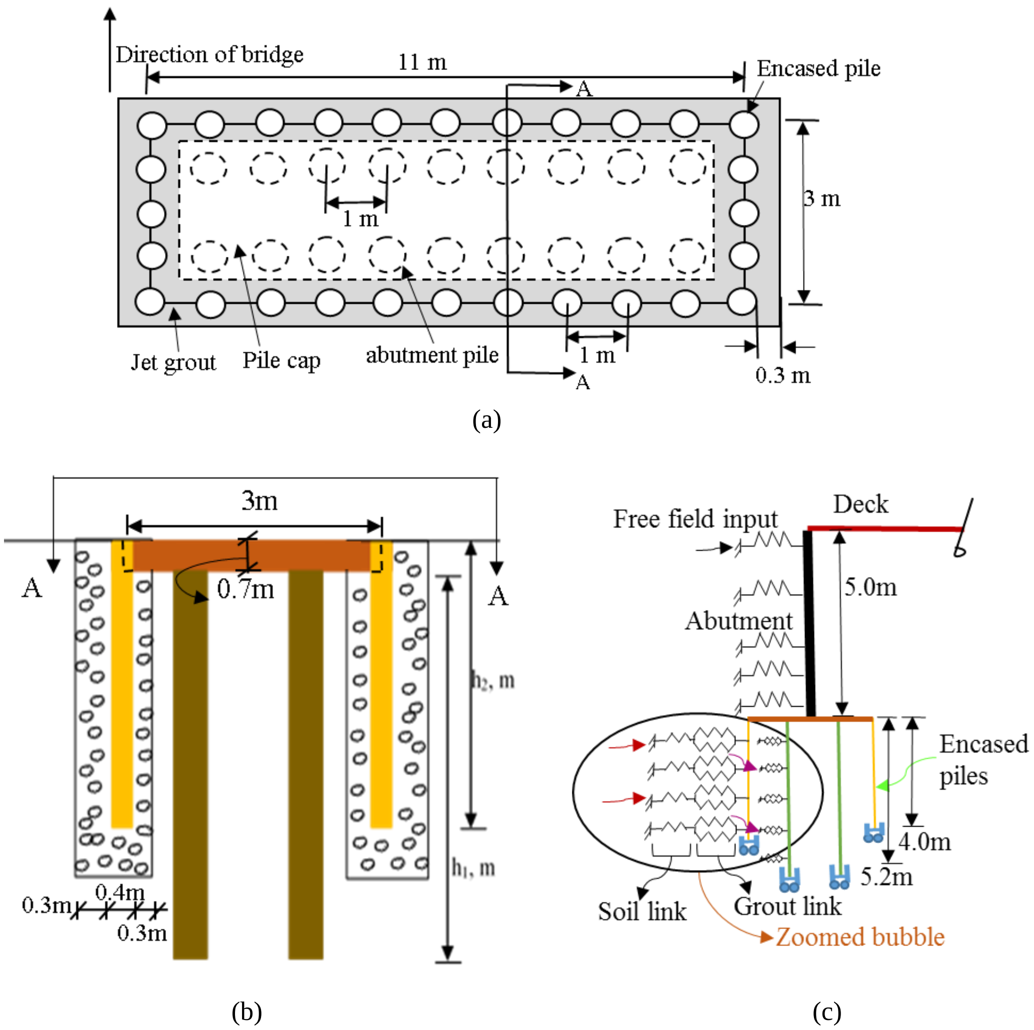

Figure 2. (a) Schematic diagram of retrofitted abutment foundation in plan, (b) vertical section A-A (h_1 = height of abutment pile; h_2 = height of encasement pile) and (c) spring-dashpot model of the abutment and its retrofitted foundation in medium stiff clay.

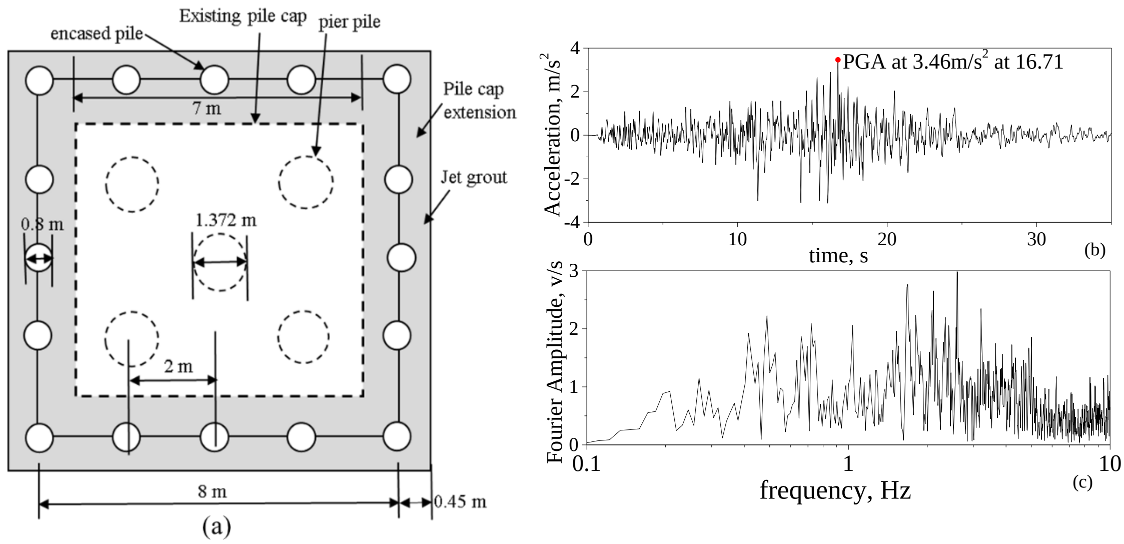

Figure 3 (a). Plan view of pier pile foundation with encased piles in jet grout, (b) Chosen GM#1 for dynamic analysis and (c) Fourier amplitude of GM.

Table 2. Natural periods (in secs.) of 9 span integral bridge model in different foundation soils

Model	Medium stiff clay	Loose sand	Soft clay	Dense sand
M2	2.49	2.60	2.65	2.38
M1	3.15	3.22	3.25	3.15

In Figures 4e-h, the moment-rotation response in the 3rd pier is shown for a cross-section located at a depth of 1.2 m from the top of the pier. In loose sand, dense sand and soft clay (Figs. 4e–g), insignificant nonlinear behaviour are observed in the cross-section for both M1 and M2 models, though the overall pier response gets reduced for the M2 model in those soils. In medium stiff clay (Fig. 4h), the pier in the M1 model shows significant nonlinear behaviour. However, it gets reduced for M2 model.

Under GM#1, an inelastic response is observed for the variation of axial force with axial displacement of the soil link elements at a depth of 2 m, in case of abutment piles of M1 model in loose sand (Fig. 5a). However, the same gets reduced to linear elastic behaviour in case of M2 model. For the same soil at a depth of 4 m, linear elastic behaviour is observed for the link elements but with the reduced response for M2 model (Fig. 5c). In dense sand, the response of soil link gets increased for the M2 model at the depth of 2 m (Fig. 5b), as compared to the response of M1 model at the same depth. At the depth of 4 m in dense sand, the soil link response remains almost the same for both the models (Fig. 5d). Thus, the response of soil links gets modified in presence of encased piles for loose sand at the depth of 2 m due to a higher lateral capacity of the overall pile group. The same effect is not observed in the case of dense sand. Similarly, for abutment piles in soft clay, the reduction in linear elastic response of the soil links is observed at the depths of 2 m and 4 m (Figs. 5e, g). In case of medium stiff clay, a marginal reduction in the linear axial force with axial displacement response is observed for the soil links at both the depths (Figs. 5f, h). Thus, the installation of new encased piles significantly affects the response of soil with lower lateral stiffness as compared to the response of soil with higher lateral stiffness.

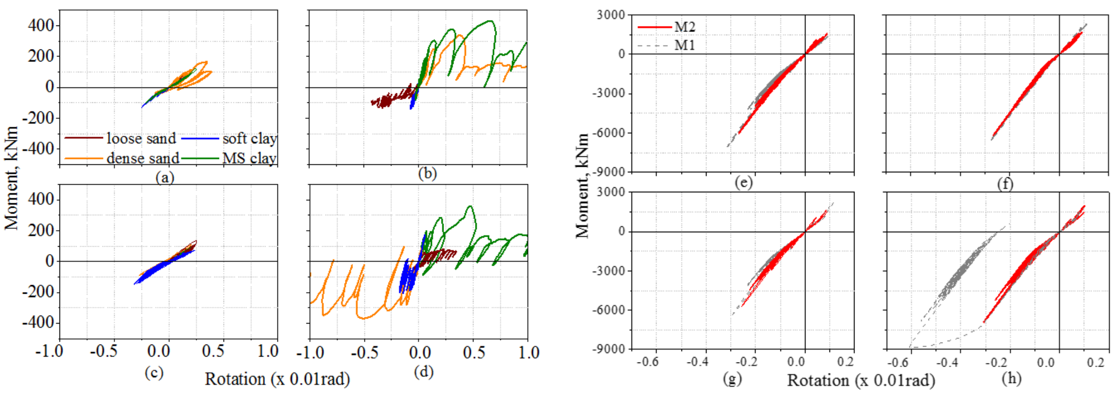

Figure 4 Moment-rotation curves of M2 model for (a) abutment and (b) pier piles, and of M1 model for (c) abutment and (d) pier piles at the depth of 1.25 m, (where, MS clay = medium stiff clay); response in 3rd pier of M1 and M2 models for the cross-section located at 1.2 m depth from the top of the pier, in (e) loose sand, (f) dense sand, (g) soft clay and (h) medium stiff clay, obtained under GM#1.

The variation of axial force with axial displacement of grout link elements is shown in Fig. 6a for different types of foundation soil at the depth of 1 m from the top of the encased pile. As observed, the grout link elements are subjected to very low axial strain levels, thus, low-level inelastic behaviour gets mobilized in the grout. The grout is expected to undergo cracking under the tensile strain, although the deformation of jet grouting is minimal in the present study. Among the soils, the maximum and the minimum response on the compression side are observed for encased piles in soft clay and loose sand respectively. As observed with link elements representing soil, grout links have also insignificant effect on pile behaviour.

The displacement time histories (DTHs) at the top of the 3rd pier (deck-pier junction) for M1 and M2 models are shown in Figs. 6b–c. The residual displacements of the M1 model in loose and dense sand are 0.10 m and 0.097 m, respectively (Fig. 6b). While in the M2 model, the overall deck response gets reduced along with the values of residual displacements as 0.03 m and 0.075 m in loose sand and dense sand, respectively. A similar reduction in bridge deck response is observed for the M2 model in soft clay and medium stiff clay as compared to the response of the M1 model in the same soils (Fig. 6c). The M1 model is observed to have peak displacement response of 0.24 m and 0.28 m respectively, and residual response of 0.069 m and 0.15 m respectively in soft clay and medium stiff clay. For the same soils, the residual displacement response at the deck level of M2 model gets reduced to 0.036 m and 0.10 m, respectively. The reduction in overall bridge deck response can be primarily attributed to the increased lateral stiffness of the abutment pile groups as well as that of the bridge through the addition of the new encased piles.

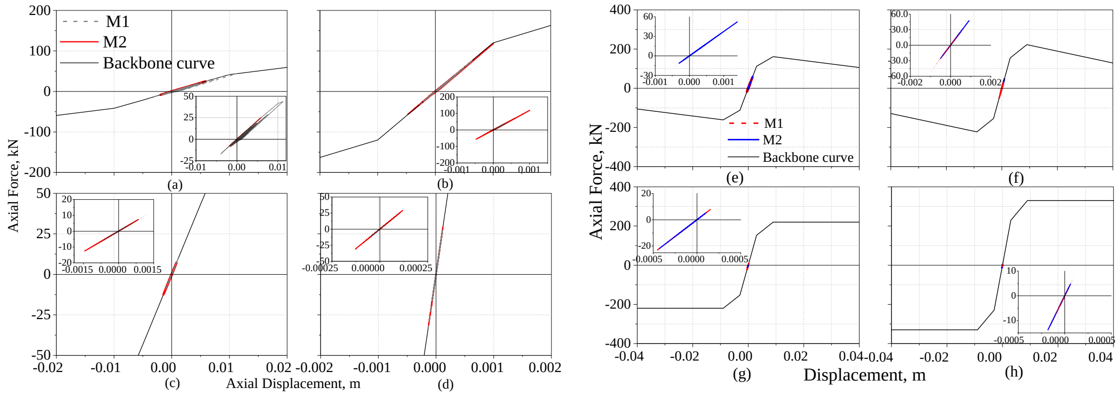

Figure 5. Response of soil link elements at the depth of 2 m in (a) loose sand, (b) dense sand, at the depth of 4 m in (c) loose sand and (d) dense sand, at the depth of 2 m in (e) soft clay, (f) medium stiff clay, and at the depth of 4 m in (g) soft clay and (h) medium stiff clay for abutment pile under GM#1.

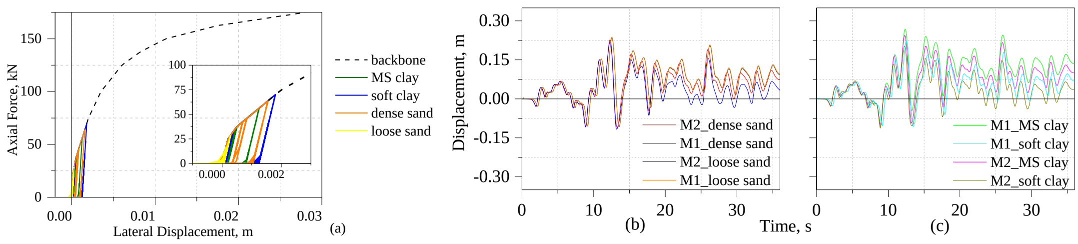

Figure 6. (a) Variation of axial force with axial displacement of grout link elements at the depth of 1 m of encased pile for M2 model (where, MS clay = medium stiff clay). Comparison of DTHs at the top of 3rd pier in (b) loose sand and dense sand, and (c) medium stiff clay and soft clay, for M1 and M2 models under GM#1 (where, MS clay = medium stiff clay).

5.2 *Comparison between M2 and M3 models*

Moment-rotation response in the 3rd pier of M2 and M3 models for the cross-section located at 1.2 m depth from the top of the pier in different types of foundation soil is shown in Fig. 7. In all the piers of M3 models, the moment-rotation response gets reduced significantly forming very narrow loops for the piers in all the types of soil. Thus, retrofitting at pier pile foundation is more effective than abutment pile foundation for this specific bridge as it reduces substructure response significantly.

The moment-rotation response of pier piles in M2 and M3 models are compared for cross-sections located at a depth of 1.25 m (Figs. 8a–d). Between the depths of 1–2 m of pile length, the monitored moment-curvature response is found to be significant. In loose sand (Fig. 8a) and soft clay (Fig. 8c), the nonlinearity in pier piles gets reduced in the M3 model as compared to the M2 model. The rotation in pier piles gets reduced with an increase in moments (Fig. 8c) for the M3 model. In dense sand and medium stiff clay, nonlinear behaviour of pier piles gets reduced in M3 model significantly and the piles tend to form narrow loops at the same depth of 1.25 m (Figs. 8b, d). For M3 models in different types of foundation soil, the response of the overall bridge is observed to be significantly reduced, particularly in medium stiff clay and dense sand.

For abutment piles in loose sand, negligible moment-rotation response is observed at the depth of 1.25 m from the ground level for M3 model as compared to M2 model (Fig. 8e). For all other soils, higher values of moment and rotation are observed for M3 models as compared to M2 models for abutment piles (Figs. 8f-h). Thus, measures need to be considered for achieving a linear elastic response at abutment piles for M3 models.

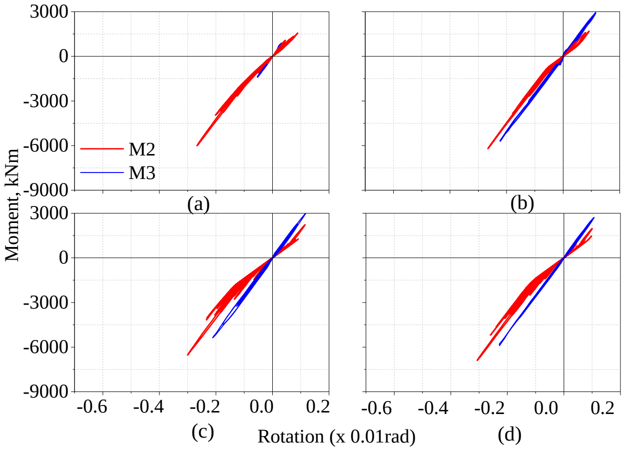

Figure 7. Moment-rotation response in 3rd pier of M2 and M3 models at 1.2 m depth from the top of the pier, in (a) loose sand, (b) dense sand, (c) soft clay and (d) medium stiff clay, obtained under GM#1

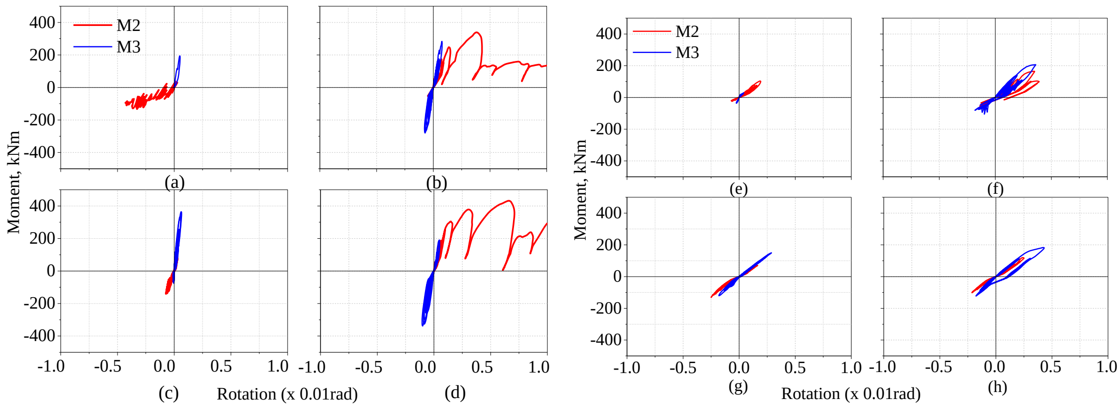

Figure 8. Comparison of moment-rotation response for pier piles in M2 and M3 models at a depth of 1.25 m with (a) loose sand, (b) dense sand, (c) soft clay and (d) medium stiff clay under GM#1. Comparison of moment-rotation response for abutment piles in M2 and M3 models at a depth of 1.25 m with (e) loose sand, (f) dense sand, (g) soft clay and (h) medium stiff clay under GM#1.

Comparison of the DTHs at bridge deck level for the two models shows the reduction of deck displacement for medium stiff clay and dense sand (Fig. 9). The displacement response of M3 model in loose sand and soft clay are quite similar to that of the M2 model. With the presence of new encased piles at abutments and pier locations, nonlinear behaviour of the piers gets reduced significantly (Fig. 7), thus, leading to a reduction in displacement response of the deck.

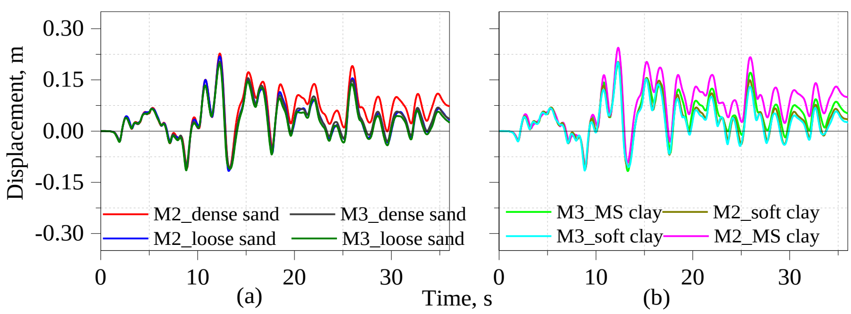

Figure 9. Comparison of DTHs at the top of 3rd pier of M2 and M3 models in (a) loose sand and dense sand, and (b) medium stiff clay and soft clay under GM#1 (where MS clay = medium stiff clay).

6 CONCLUSIONS

In the present paper, the effectiveness of retrofitting of the pile foundation is investigated through encasement of new piles in jet grouting, on a 9 span integral bridge model under a single site-specific ground motion. The response of abutment piles is observed to increase from M1 models to M3 models in all types of foundation soils except loose sand. However, the response of the pier piles is reduced significantly from M2 to M3 models on implementation of the retrofitting technique. The pier response is also reduced at all stages of retrofitting. The effectiveness of the first stage of retrofitting (response of the M2 model) is reflected through the reduction of residual displacement at deck level by 67.1% and 53.2% as compared to the response of M1 model in medium stiff clay and soft clay, respectively. When further encased piles are added at pier pile foundation, the response of M3 models decreases with reduction in deck-level displacement. These may also result in uninterrupted bridge serviceability. Also, a linear elastic response is observed in the bridge piers in M3 models for all types of foundation soils as compared to the nonlinear response of M1 models.

For the model upgrading, further research needs to be carried out for the mass contribution of foundation soil under dynamic shaking. A detailed parametric study is required by varying grout properties like water-cement ratio or cement replacement ratio to understand the soil-grout-pile effect under dynamic loading.

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