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COMPARISON BETWEEN TWO MODELING ASPECTS TO INVESTIGATE SEISMIC SOIL STRUCTURE INTERACTION IN A JOINTLESS BRIDGE

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Abstract. Seismic waves propagate through a series of rock and soil layers before they interact with the foundation and superstructure. Besides the original characteristics of the earthquake motion at the instant of fault rupture, it is also important how the soil site responds in terms of amplification or de-amplification of different frequency contents. Considering possible nonlinear response of soil and structure, a coupled soil-structure model is required to efficiently capture the dynamic behavior of the entire system. This paper focuses on comparison of two modelling strategies for Soil-Structure Interaction (SSI) aiming to define the behavior of a jointless bridge, namely (a) one with an explicit full-scale soil domain with bridge model and (b) another with Beam on Dynamic Winkler's Foundation (BDWF)/ nonlinear soil springs. Finally, the structural components that effect overall behavior of superstructure is compared between these two models and the variation of seismic response from performance-based study is discussed.

Keywords. Beam on Dynamic Winkler Foundation (BDWF), soil continuum, jointless bridge, abutment-backfill interaction.

1. Introduction

Creation of full scale soil domain as continuum requires significant attention and expertise and the analyses are numerically costly for SSI investigation. In particular, two models of a specific bridge have been investigated by using a structure that resembles the well-known Humboldt Bay Middle Channel (HBMC) Bridge present in California. It must be mentioned that modeling the target bridge through proper definition of all details (i.e. pier-deck connections, etc.) to describe accurately its seismic response was already achieved in the past studies [1-2] and is not the goal of present work.

In the first approach, a full-scale soil-foundation-bridge with or without abutment backfill interaction (i.e. *full SSI with/no BA* models) is modeled in *OpenSees* [3-4] and in the second approach, nonlinear springs are introduced to represent soil stiffness, replicating soil-pile interaction (SPI) and abutment backfill interactions (ABI) (i.e. *FB_SD no/with BA* models). Hence, in the first and the second modelling approaches, different structural parameters are

compared to investigate the overall response of the bridge structure. Modelling of *full SSI no/with BA* models is computationally intensive. Thus, modelling of continuous soil domain to consider SSI is not a very common practice in design firms currently. So, to incorporate simplistic SSI in seismic analysis of bridge, continuous soil domain has been replaced with nonlinear spring dashpots to take care of SSI in *FB_SD no/with BA* models. Further, comparisons are made with and without ABI between these two modelling approaches.

2. Selection of Ground Motions

A uniform hazard response spectrum (UHRS) for bedrock-level ground motions is used as the target spectrum to select and scale the input ground motions for the analyses as discussed in Dhar *et al.* [5]. The UHRS is developed from the 2008 United States Geological Survey [6] national seismic hazard maps for the Humboldt Bay area for rock outcrop assuming $V_{s, 30m} = 800\text{m/s}$ (according to NEHRP [7], site class B). The corresponding 5% damped elastic displacement response spectrum has been given as target to REXEL-Disp [8] to select the ground motions for dynamic analysis from strong ground motion database SIMBAD [9]. The input parameters in REXEL-Disp to find the ground motions are: magnitude = 5.5-7.5; fault to site distance = 0-30km; spectrum matching tolerance = $\pm 20\%$; spectrum matching period = 0.2s-5s; site specification = EC8 site class A; probability of exceedance = 10% in 50 years, representing the return period of 475 years. Seven real record ground motions are chosen for horizontal direction by scaling in the response spectrum within the period of interest, such that the mean spectral response lies between the tolerances. Different parameters of selected motions are given in Table 1. Corresponding 5% damped elastic displacement spectra with the average of the ground motions are shown in Fig. 1.

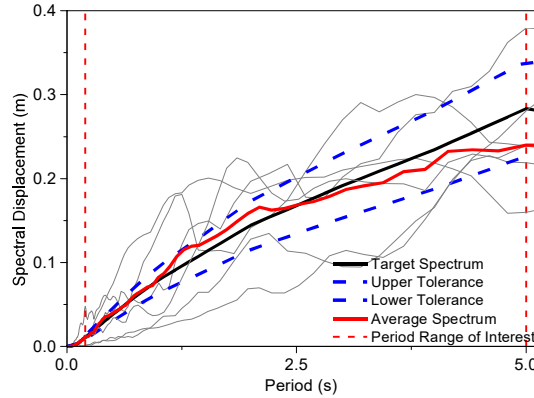


Figure. 1. Displacement spectra of all the ground motions used in present study

Table 1 shows the selected and scaled rock outcrop Acceleration Time History (ATH) set [5]. The motions highlighted with gray are selected to discuss in detail the soil and structural response; the acceleration plots in time and frequency domains are shown in Fig. 2. The two selected ground motions (GM#1 and GM#2) correspond to the same station but with different Peak Ground Acceleration (PGA) amplitudes. Thus, the focus of this paper is to identify similarities and differences for the two different modelling techniques in light of the observed response.

Table. 1. Different parameters of selected ground motions [5]

Station ID	Earthquake Name	Date	M_w	Fault Mechanism	Epicentral Distance, km	PGA, m/s^2	Scale Factor	Scaled PGA, m/s^2
ALT	Irpinia	23 Nov 1980	6.9	Normal	23.77	0.54	6.31	3.46
ST_106	South Iceland	17 Jun 2000	6.5	strike-slip	5.25	3.39	0.90	3.06
ST_112	Olfus	29 May 2008	6.3	strike-slip	8.25	3.28	1.67	5.47
ST_101	Olfus	29 May 2008	6.3	strike-slip	7.97	5.00	1.41	7.06
BSC	Irpinia	23 Nov 1980	6.9	Normal	28.29	0.95	0.72	0.68
ST_106	South Iceland	21 Jun 2000	6.4	strike-slip	21.96	0.51	1.42	0.73
LPCC	Christchurch	21 Feb 2011	6.2	Reverse	1.48	9.16	1.38	12.64
mean values:			6.5		13.85	3.26	1.98	4.73

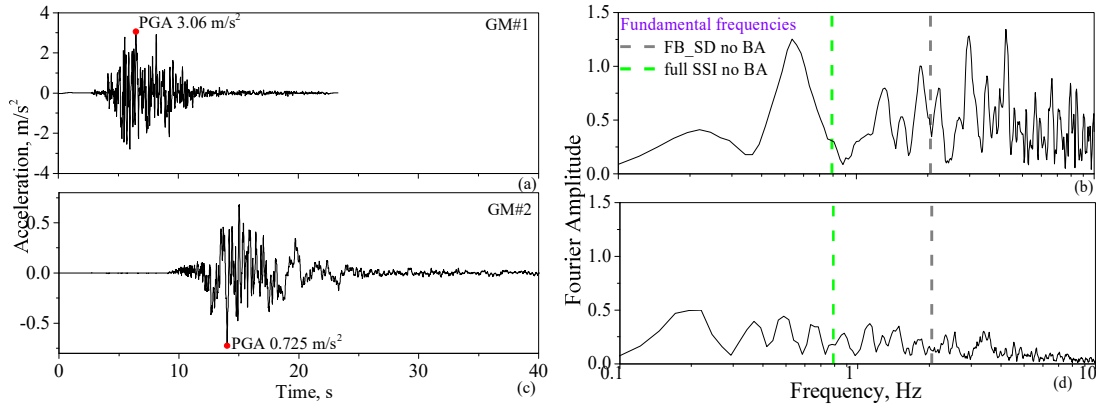


Figure. 2. Rock outcrop earthquake (scaled) records used in this study: (a) strike-slip fault of the 17 June 2000 South Iceland Earthquake, $M_w=6.5$, recorded at the 5.25km from epicenter, station ID ST_106; Ground Motion (GM) #1, (b) Fourier amplitude spectrum of GM#1, (c) strike-slip fault of the 21 June 2000 South Iceland Earthquake, $M_w=6.4$, recorded at the 21.96km from epicenter, station ID ST_106; Ground Motion (GM) #2, and (d) Fourier amplitude spectrum of GM#2 with fundamental frequency (marked) for different considered models.

3. Modelling

3.1. Structural modelling

As illustrated in detail in Zhang *et al.* [2], the Humboldt Bay Middle Channel Bridge, located near Eureka in California (USA), is 330m long, 10m wide, and 12m high (average height over mean water level). The bridge superstructure consists of nine spans with four precast prestressed concrete I-girders and cast-in-place concrete slabs. The bridge deck is supported by two seat-type abutments and eight bents consisting of a single column and hammer head cap beam. The third and sixth bents have nonlinear shear keys for energy dissipation. Pile caps of 1m thickness are supported by deep foundations consisting of driven precast

prestressed concrete pile groups. For the sake of simplicity, only the longitudinal dynamic response is analyzed in this study. Two finite element (FE) models of the bridge are considered: a linear one with elastic beam-column elements for both the superstructure and the piers and a nonlinear one where piers are modeled by forced-based fiber elements. In both the cases, the shear keys are not included and continuous joints are considered. The elastic properties of structural elements for linear analyses are adopted from Zhang *et al.* [2] as A (area, m^2) = 12, 4.56, 3.4 and I (moment of inertia, m^4) = 1.44, 3.212, 0.8188 for abutment, deck and pier sections, respectively. All the concrete elements have the same elastic modulus of 28GPa.

In the nonlinear simulations, force-based fiber elements [10] with five and ten integration points are used in piers. Pier cross-section is discretized as shown in Fig. 3(a). Kent-Park-Scott [11] concrete model is used to model nonlinear concrete material with degraded linear unloading/reloading stiffness and no tensile strength is considered. Giuffre-Menegotto-Pinto [12] steel material is specified with 0.8% isotropic strain hardening for reinforcement bars with 200 GPa elastic modulus and 276 MPa yield strength. The properties of confined and unconfined concrete used in the study are the same as adopted by Zhang *et al.* [2]. Compressive strengths of confined and unconfined concrete are 34.5 MPa and 27.6 MPa, respectively. The simulations have been performed using the finite element program *OpenSees* [3]. Rayleigh damping scheme is introduced as viscous material damping to calibrate the Rayleigh damping parameters. The damping ratio is prescribed as 5% at 0.5Hz and 5.0Hz for *full SSI no/with BA* models in *OpenSees* shown in Fig. 3(b).

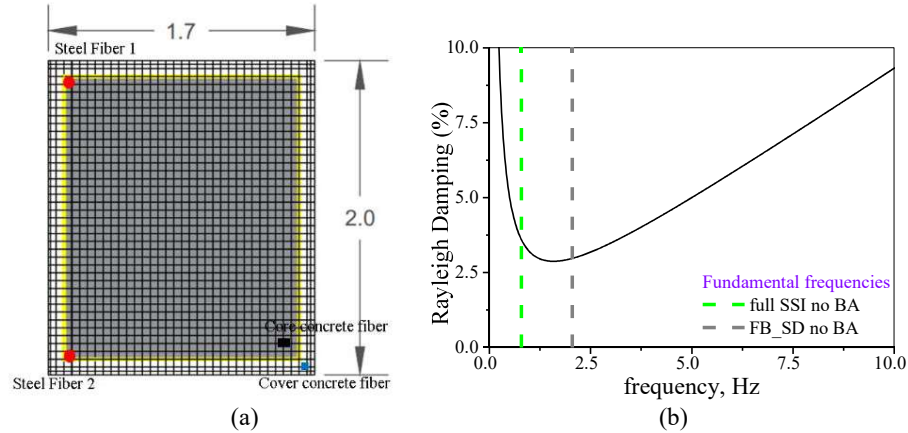


Figure 3. (a) Fiber-based discretization of the pier cross-section, nonlinear axial response for the ones shown with corresponding markers will be discussed in detail and (b) Rayleigh damping considered in the SSI analyses

3.2. Geotechnical modelling

Two-dimensional soil modeling has been carried out in *OpenSees* [4]. Soil domain is 1500 m wide (evaluated through iterations to reach free field motion at boundary) and 220 m in depth. The entire soil domain consists of 4 different layers (Fig. 4a) having the static and dynamic properties (Table 2), in which the geotechnical constitutive parameters are adopted from Zhang *et al.* [2]. Pressure independent multi-yield material has been used to describe the soil behavior through a formulation based on the multi-surface plasticity concept [13] with associative flow rule, inbuilt in *OpenSees*. The yield surfaces are of the Von Mises type. Since total stress analyses is carried out, any direct consequence of significant excess pore water pressure generation has been naturally neglected in the present study. To represent the

nonlinear nature of the soil domain, variation of shear modulus degradation and damping ratio with shear strain are adopted as per the proposition of Darendeli [14], shown in Fig. 4(b) and (c), respectively. The soil domain lateral boundary conditions are implemented by tied degrees of freedom (TDOF) [1, 15] at the lateral two ends of soil domain. This implies that the soil domain follows the pattern of a 2D shear beam constraints, in which generally the horizontal response dominates over the vertical response. At the base level, classical Lysmer and Kuhlemeyer [16] type absorbing boundary conditions are applied in the horizontal direction by properly calibrating the dashpot coefficients together with the classical vertical displacement restraints. The dynamic base input motion is given in horizontal direction to study the horizontal response of soil-structure system. In the case of *full SSI no/with BA* models, the precast prestressed concrete pile groups, considered linear elastic in all the analysis, are analyzed per [17-19] with an equivalent pile groups of stiffness 1.1 GPa. The precast driven piles are of 5.2 m in length and floating type. In the case of *full SSI with BA* model (Fig. 5), abutment backfill dynamic properties are kept similar as the soft clayey soil (top most layer) of soil domain. To model the abutment-backfill interaction (ABI), gap or interface element is not considered at the abutment-backfill interface.

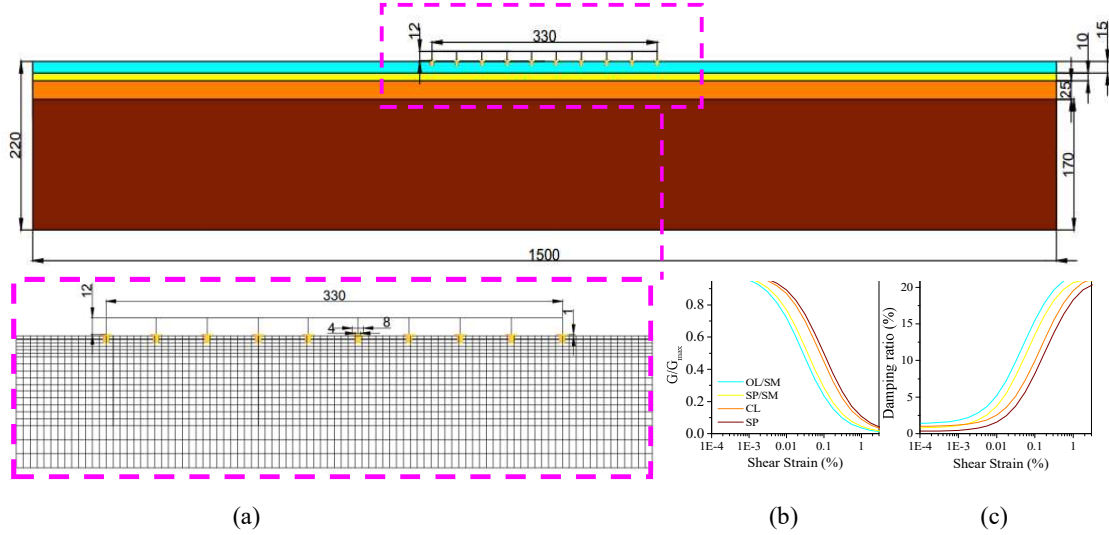


Figure 4. (a) Full SSI noBA model in OpenSees (all dimensions in m), (b) Normalized shear modulus degradation vs. shear strain and (c) Damping ratio vs. shear strain curves for different soil layers [14].

Table 2. Properties of different soil layers used in the present study (modified after [2])

Soil layer	Elastic properties			Non-linear properties		
	Maximum shear modulus (MPa)	Poisson's ratio	Total unit weight (t/m ³)	Undrained shear strength, (kPa)	Shear modulus ratio (Figs. 4(b), (c))	Plasticity Index
OL/SM	76	0.45	1.9	30	Cyan Line	10
SP/SM	171	0.45	1.9	11.9	Yellow Line	0
CL	288	0.45	1.8	100	Orange Line	30
SP	525	0.45	2.1	52.5	Brown Line	0

In the simplified models, on the other hand, *FB_SD no BA* model represents the soil-pile interaction (SPI) and *FB_SD with BA* represents SPI and ABI, together. In the aforementioned models, SPI and ABI are represented through classical nonlinear two-noded

zero length links and dashpots to represent idealized SSI at far and near fields. In the near field spring-dashpot system, hysteretic damping is considered through nonlinearity of multilinear plastic uniaxial material, inbuilt in *OpenSees*. Lateral and vertical springs are modeled as per API-rp2a [20] in parallel to represent lateral load bearing capacity and skin friction of pile shaft or pile tip end bearing, respectively. Far field soil stiffness and radiation damping are modeled using the coefficients provided by Gazetas and Dobry [21]. Far field spring-dashpots are linear elastic and modelled in series with near field hysteretic springs. Far field spring and dashpots are modelled in parallel configuration. At the fixed end of far field spring-dashpots, free field motions are applied based on the exact depth of soil column at corresponding depth of springs. Detailing of springs and dashpots with schematic diagram of SPI (modelled explicitly with near and far field) and ABI are discussed in Figs. 5(a) and 5(b), respectively: where K_s is the linear stiffness and C_s is the radiation dashpot coefficient of far field spring. It has been proved from past researches [22-24] that force-displacement relationships from API overestimate the soil stiffness to address SPI. Therefore, implementation of API curves in *FB_SB no/with BA* models gives conservative response, moreover, far-field spring-dashpots are linear elastic. Thus, to reduce this error, 10% Rayleigh damping is introduced in *SB_FB no/with BA* models.

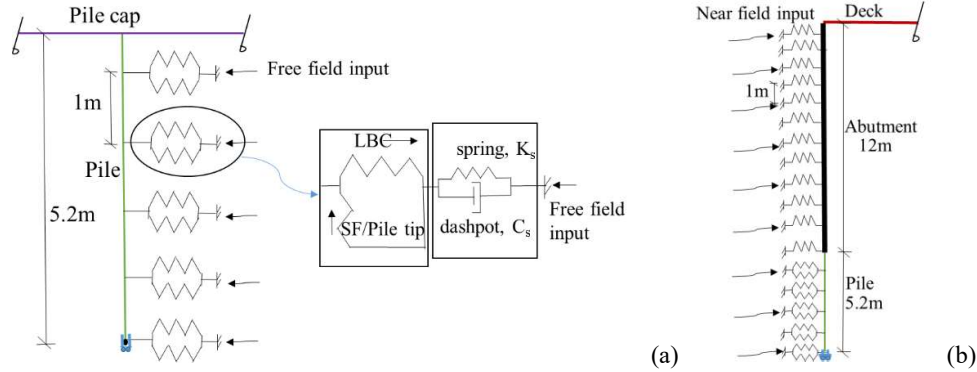


Figure 5. Schematic diagram of (a) soil-pile interaction and (b) abutment-backfill interaction

In *FB_SD with BA* model, nonlinear springs are added to model abutment backfill soil. The abutment is considered as frame type abutment and nonlinear springs are positioned at 1m distance along the height of abutment (Fig. 5(b)). The force-deformation curves for the springs are calculated as per BA 92/46 [25].

4. Analysis

During the first phase of analysis, two models are compared, namely (a) *full SSI no BA model* and (b) *FB_SD no BA model*. Initially, a single step static analysis is carried out. Then, input motion is applied as force time history at the base of soil domain in full SSI with BA model. For *FB_SD with BA* model, after static gravity analysis, free field input motions are applied at different corresponding depths as Displacement Time Histories (DTHs) extracted from 2D ground response analysis of similar soil column performed in *OpenSees*. For *full SSI with BA* and *FB_SD with BA* models, salient parameters are compared in the following sections and differences in structural behavior from nonlinear THA are discussed.

5. Results

5.1. Comparison of response of *full SSI no BA* and *SB_SD no BA* models

From Fig. 6(a) and 6(b) under GM#1, ninth pier top and bottom ATHs are compared and in (c) and (d) their respective Fourier Transforms (FT) are shown. The observed ATHs at the bottom of the pier have similar values (Figs. 6(a) and 6(c)), but at the deck level (Figs. 6(b) and 6(d)) *FB_SD no BA* model shows more amplified response as compared to the *full SSI no BA* model. However, the peaks of Fourier amplitude occur at 1.78Hz and 2.29Hz for *FB_SD no BA* and *full SSI no BA* models, respectively. Foundation of *FB_SD no BA* model is found to be stiffer and nonlinearity still not developed in the underlying soil because the API curves overestimate soil stiffness at different depths of piles. Moreover, a resonance effect is also noted near 2 Hz, which is not present in the fully-coupled SSI counterpart.

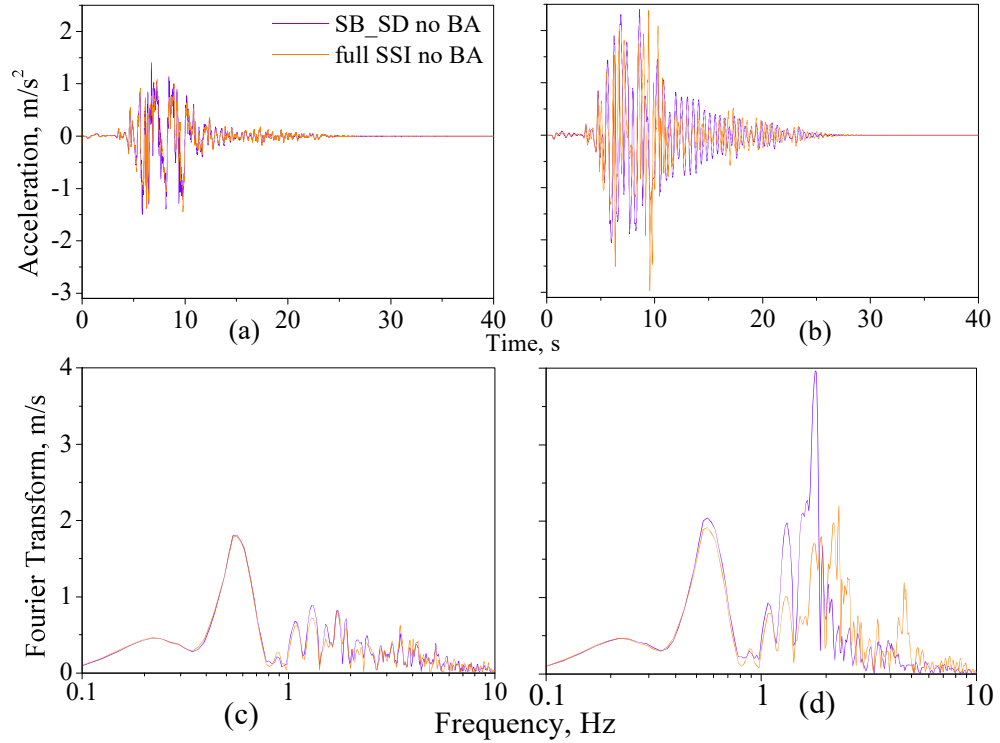


Figure 6. Acceleration time history (ATH) (a) at the bottom, (b) ATH at top of 9th pier, (c) FT of the ATH shown at (a) and (d) FT of the ATH in (b) under GM#1.

Similar response is observed for Shear Force Time History (SFTH) plots in Fig. 7(a). Due to seismic waves amplification at deck level, SFTH is higher in *FB_SD no BA* model. From the SFTH, it can be stated that *FB_SD no BA* model shows significantly less nonlinearity in the bridge substructure component as compared to the *full SSI no BA* model. From normalized FT of SFTHs in Fig. 7(b), it is observed that the peak of fourier amplitude of *FB_SD no BA* model is at 1.78Hz, thus magnitude of Fourier amplitude is significantly higher. This implies that once the seismic waves propagate from the foundation to the deck level, *FB_SD no BA* model shows more amplified response as compared to the *full SSI no BA* model. This is also evident from the moment-curvature response at the top and bottom sections of the ninth pier (Figs. 7(c) and 7(d)). Moment curvature response at the top of ninth pier is found to be underdamped in *FB_SD no BA* model and mainly forms the negative moment-curvature loops in the third quadrant. At the base of the ninth pier, moment curvature response (Fig. 7(d)) shows similar pattern on the opposite quadrants. After

investigating several parameters at the top and base of the pier, it can be stated that *FB_SD no BA* model is getting more amplified from the foundation to the deck level as compared to the *full SSI no BA* model, thus, mismatches in response arise for different response parameters at the deck level.

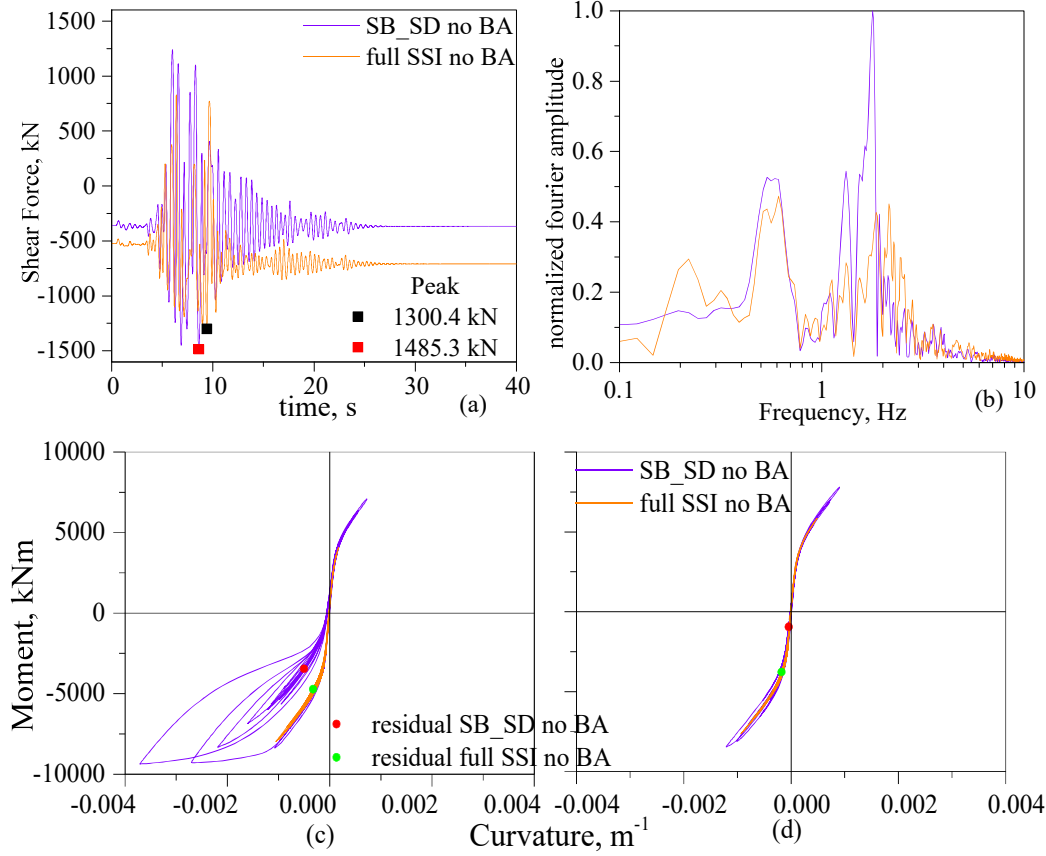


Figure. 7. (a) Shear Force time history at the top of 9th pier and (b) Fourier transform of the SFTH. Moment curvature response of 9th pier at (c) top and (d) bottom under GM#1

Under GM#2, SFTHs at the top of ninth pier (deck level) are shown for the two models in Fig. 8(a). The difference in energy content of the response is observed through the normalized FT of SFTHs in Fig. 8(b).

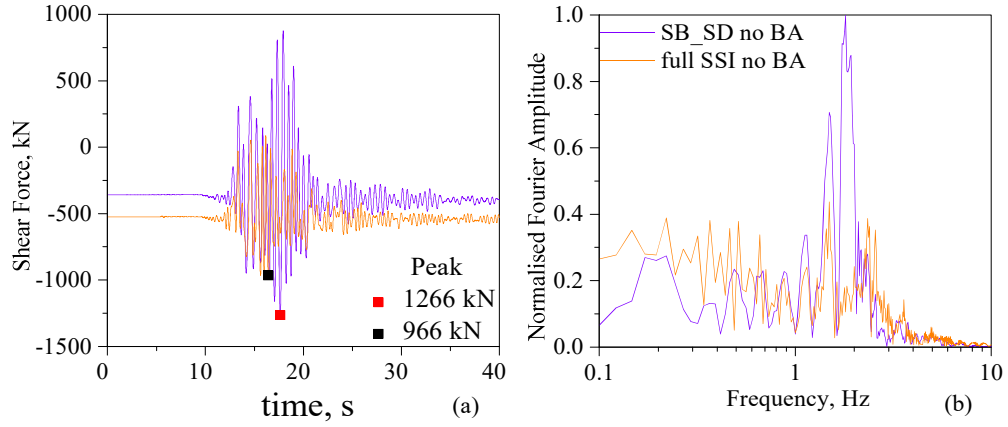


Figure. 8. (a) SFTH at the top of 9th pier and (b) FT of the SFTHs in (a)

5.2. Comparison of response of *full SSI with BA* and *FB_SD with BA* models

For comparison, the ATHs are shown at the top and the bottom of the ninth pier in Figs. 9(a) and 9(b), and the corresponding Fourier amplitudes of ATHs are shown in Fig. 9(c) and (d) under GM#1. ATH at the base of the pier in *FB_SD with BA* model is marginally higher than *full SSI with BA* model. Thus, in Fig. 9(c), the peaks of the amplitudes are higher from 0.6Hz onwards. However, at the top of ninth pier (Fig. 9(b)) ATHs are quite comparable for both the models. In Fig. 9(d), peak Fourier amplitudes for *FB_SD with BA* and *full SSI with BA* models are at 2.83Hz and 2.12Hz, respectively, which indicate that *FB_SD with BA* model is marginally stiffer as compared to the *full SSI with BA* model.

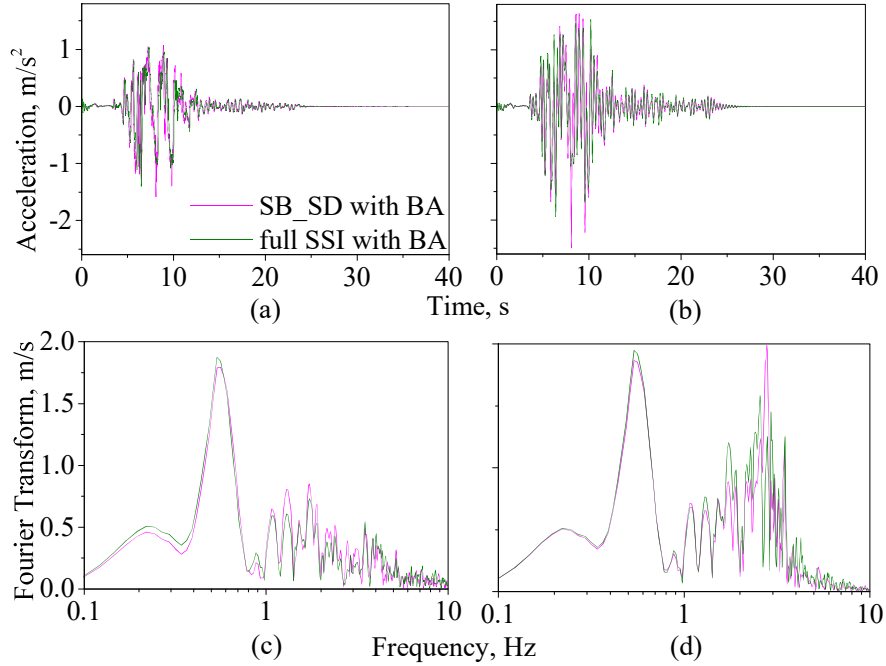


Figure. 9. ATH (a) at the top and (b) at the bottom of 9th pier, (c) Fourier transform of the ATH shown in (a) and (d) Fourier transform of the ATH in (b) under GM#1.

Under GM#2, similar responses are observed, at the top and bottom of first pier ATHs in Figs. 10(a) and 10(b) and their corresponding FTs in Figs. 10(c) and 10(d), respectively. Due to low intensity input motion, the ATHs at the base of the pier are quite similar for both the models along with their FTs. At the top of the pier, ATHs have two different peaks at different time instants. In Fig. 10(d), peak amplitudes occur at the frequencies of 1.48Hz and 3.07Hz for *FB_SD with BA* and *full SSI with BA* model, respectively. At low intensity of PGA input, foundation soil is expected to behave in the linear elastic range. As the nonlinearity of the foundation soil does not influence the behavior of the superstructure, the bridge is observed to be more flexible for *FB_SD with BA* model as compared to the *full SSI with BA* model. Thus, the peak of FT at deck level in *FB_SD with BA* model is at lower frequency instant than *full SSI with BA* model. Moreover, amplitudes are quite same both spectrally and temporally.

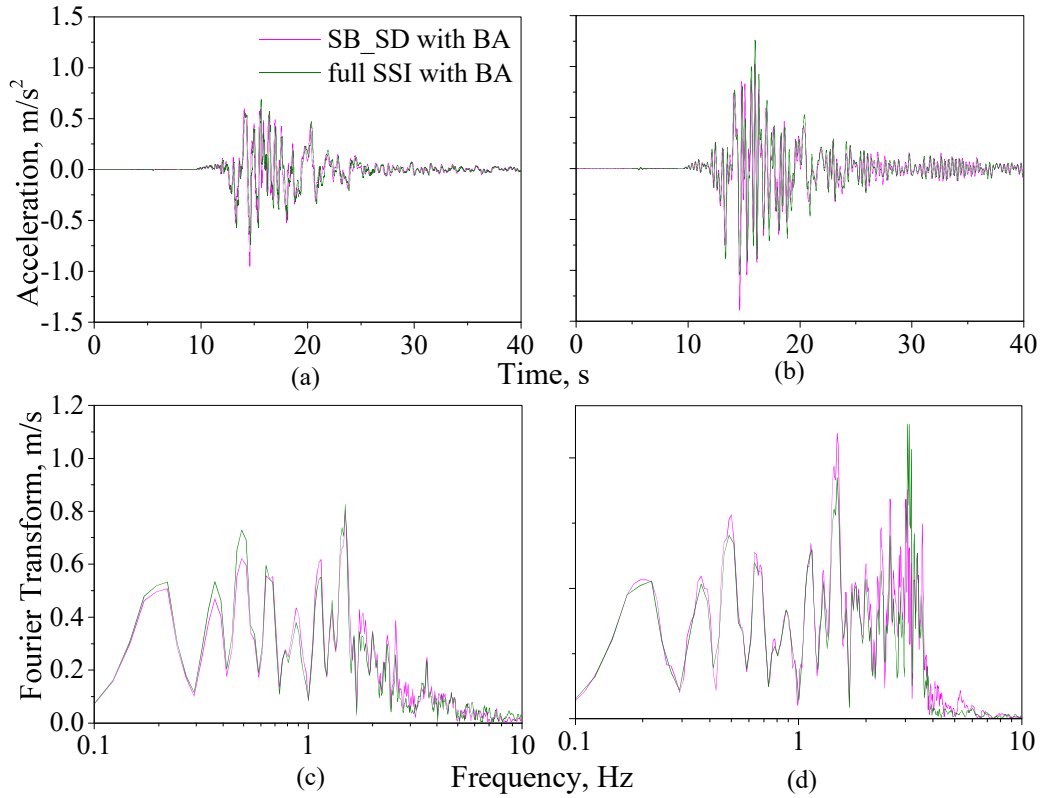


Figure. 10. ATH (a) at the top, (b) ATH at bottom of 1st pier, (c) fourier transform of the ATH shown at (a) and (d) fourier transform of the ATH in (b) under GM#2

At the top of the ninth pier of *FB_SD with BA* model in Fig. 11(a) under GM#1, both the peak and the residual shear forces are higher as compared to the *full SSI with BA* model; this shows that more nonlinearity develops in the *FB_SD with BA* model under GM#1. The Fourier amplitude plot of SFTHs in Fig. 11(b) shows that in *FB_SD with BA* model, the seismic forces are higher as compared to the *full SSI with BA* model with a frequency of 0.8Hz, beyond which the SSI model shows higher seismic force. The peaks of Fourier amplitudes occur at 0.42Hz and 1.06Hz for *FB_SD with BA* and *full SSI with BA* models, respectively. Thus, it signifies that in the former model, seismic response is being more amplified at deck level and more nonlinearity develops at pier sections than the *full SSI with BA* model. Under GM#2, the variation of SFTHs at the top of ninth pier shows that the peak

shear force in *FB_SD with BA* model is higher than the *full SSI with BA* model (Fig. 11(c)); also the *FB_SD with BA* model shows higher residual response. Thus, in *FB_SD with BA* model piers have developed significant nonlinearity as compared to the *full SSI with BA* model. From the comparison of Fourier amplitudes of SFTHs in Fig. 11(d), the seismic force content is observed to be higher in *FB_SD with BA* model upto a frequency of 1Hz, beyond which the *full SSI with BA* model carried higher forces in the high frequency range. This further proves that the *FB_SD with BA* model is weaker and more flexible than *full SSI with BA* model.

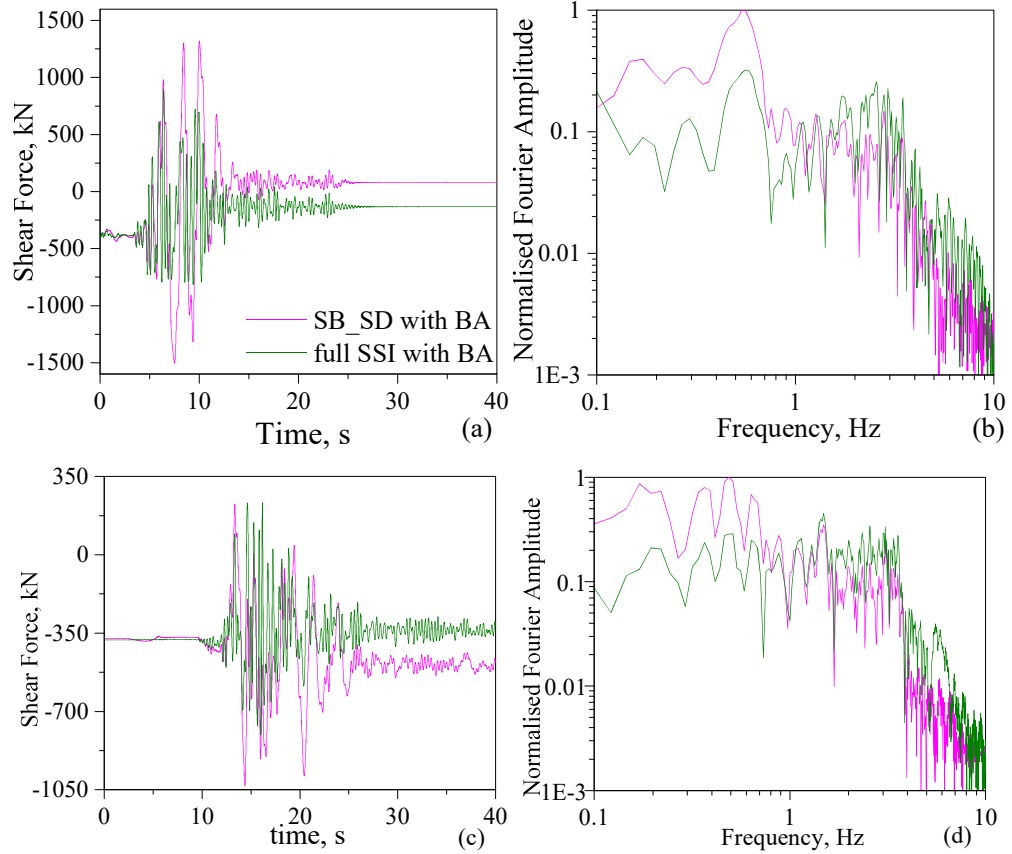


Figure. 11. (a) SFTHs at the top of 9th pier and (b) Fourier transform of the SFTHs in (a) under GM#1, (c) SFTHs at the top of 9th pier and (d) Fourier transform of the SFTHs in (c) under GM#2.

6. Conclusion

Based on the nonlinear time history analysis of bridge-soil system under the two selected ground motions, the two mentioned modelling approaches yield reasonably close response. The salient conclusions are stated as follows:

- As compared to the *full SSI no/with BA* models, shear forces and bending moments at piers are higher for *FB_SD with/no BA* models because foundation soil stiffness is higher in API force-displacement curves for clayey soil.

- The *Full SSI with BA* model is observed to show the lowest nonlinear dynamic response for the bridge structure. As the backfill soil provides longitudinal restraints to the bridge, the seismic force from the superstructure dissipates into the backfill soil through passive resistance.
- *FB_SD no BA* model shows higher response as compared to the *full SSI no BA* model as the prescribed API guidelines overestimate the soil stiffness for the foundation soil. Structural response is amplified significantly at deck level for *FB_SD no BA* model and results in higher forces and moments at pier-deck junctions.
- For *FB_SD with BA* and *full SSI with BA* models, the former model is stiffer at superstructure level and deforms in higher curvature after dynamic analysis, thus, the mobilized nonlinearity and the residual response are observed to be higher for this modeling approach. Due to insufficient soil nonlinearity at the foundation and backfill components, the *FB_SD with/no BA* models exhibit overall higher stiffnesses.
- In *full SSI with/no BA* models, due to soil significant nonlinearity, bridge response does not amplify at deck level, thus in this type of modelling bridge response is lower than the simplified SSI modelling approach.

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