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Shaking table experimental study on pounding between adjacent structures founded on different soil types

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ABSTRACT

The aim of this study is to extensively investigate the effect of the soil type on the response of colliding structures based on shaking table experimental tests. Two single-storey models of steel buildings with different dynamic parameters were considered in this study. Three pounding scenarios were taken into account by applying different seismic gaps (0.5 cm, 1 cm and 1.5 cm as well as the no pounding case). First, the effect of pounding on the response of these two structures was analysed. Then, the effect of the seismic gap on the response of colliding structures was studied. Finally, the effect of soil type on the response of structures exposed to interactions was investigated. Five soil types were considered in the study, which are the five soil types defined in the ASCE 7-10 code (hard rock, rock, very dense soil and soft rock, stiff soil and soft clay soil). The results of the investigation show that pounding significantly increases the level of accelerations of structures during the whole time of vibrations as well as the peak acceleration itself. Pounding is more significant for the flexible structure than for the stiff one. Also, the seismic gap has a significant effect on the acceleration response of colliding structures. Not necessarily larger gap leads to lower responses unless it is large enough to eliminate collisions at all. Moreover, the results of the study show that the soil type has a significant effect on the response of colliding structures. The soil type effect is more significant when pounding takes place. The maximum and minimum peak acceleration differs for various soil types, pounding scenarios, seismic gaps and earthquakes.

1. Introduction

Earthquake-induced pounding is considered as one of the major effects of earthquakes as it has a significant effect on the response of colliding buildings [1–4]. It has been experienced in many earthquakes. For instance, in the Mexico earthquake (1985), 40 % of buildings experienced pounding and in 15 % of the buildings with severe damage or collapse, pounding was found [5] where in 20–30 % of them pounding was the major reason of damage [6]. Pounding was also found in 200 out of 500 surveyed buildings in the Loma Prieta earthquake [7]. Indeed, pounding was also found in recent earthquakes, such as the Christchurch (New Zealand, 2011) [8,9] and the Gorkha (Nepal, 2015) [10].

Research on earthquake-induced pounding has been recently extensively studied (see [11–15], for example). Pounding occurs due to the narrow gap provided between adjacent buildings as well as the difference between the dynamic properties of adjacent structures. Pounding becomes more significant when adjacent structures have a significant difference in the mass, natural period and other dynamic properties [1,16]. Indeed, pounding has been found to lead to a substantial increase in the peak interstorey drift (IDR), residual IDR, floor peak accelerations, shear forces and impact forces while the displacement response can be increased or decreased based on the dynamic properties of the colliding buildings [17–23]. This increase is experienced in the direction of pounding while the response in the other directions is unaffected [24]. The degree of amplification is influenced by the dynamic properties of colliding buildings [25,26]. The frequency ratio has the largest influence on the maximum impact force and ductility demands while the frequency and mass ratios have the largest influence on the impact impulse (mass ratio is predominant for low frequency range) [27,28].

Most of the studies focusing on earthquake-induced pounding ignored the soil-structure interaction (SSI) and considered only fixed-base buildings. In fact, the SSI induces flexibility of the structure due to the flexible soil/base [29]. Moreover, considering fixed base buildings for braced frames was found to be conservative and considering SSI is not necessary (but more economic) while considering SSI for unbraced frames resting on soft soils is necessary as it has a significant effect on the

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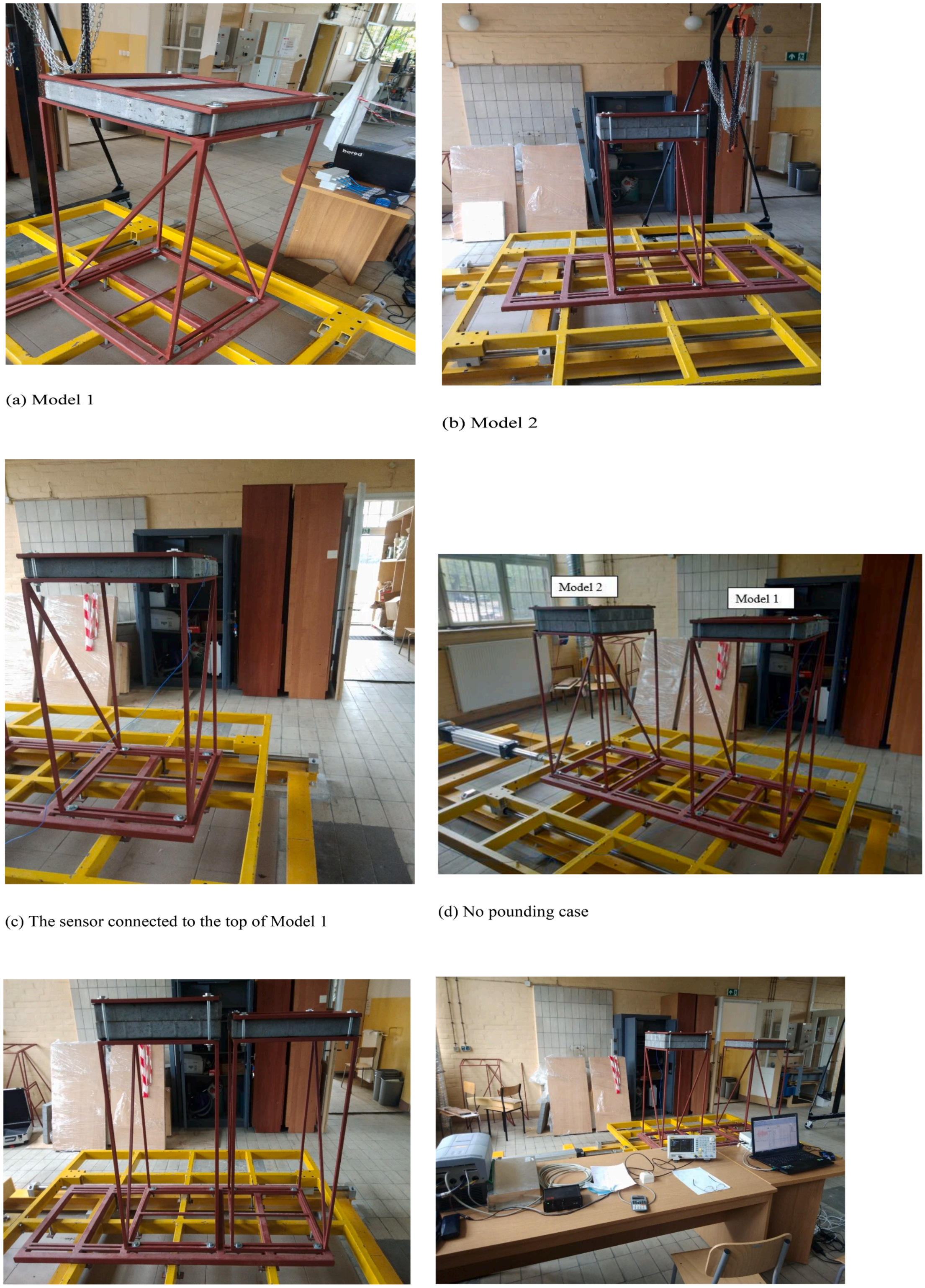


(a) Model 1

(b) Model 2

(c) The sensor connected to the top of Model 1

(d) No pounding case

(e) Pounding case

(f) Measuring system of the experiments

Fig. 1. Structural models considered in the study.

lateral deflections and interstorey drifts resulting in severe damage or collapse [30]. Far [31] compared the responses of unbraced midrise buildings resting on soft soils in the case of fixed base buildings and in the case of SSI. It was found that considering SSI leads to the decrease of the base shear as compared to the fixed base conditions while the interstorey drifts substantially increase in the case of SSI as compared to the fixed base conditions. This means that the SSI have a significant influence on the response of midrise buildings resting on soft soils. Also, Tabatabaiefar et al. [32–34] found that as the shear wave velocity and shear modulus of the soil decreases, the interstorey drifts and the necessity of considering SSI effects in the seismic design of buildings increases. Indeed, several methods have been developed to determine the

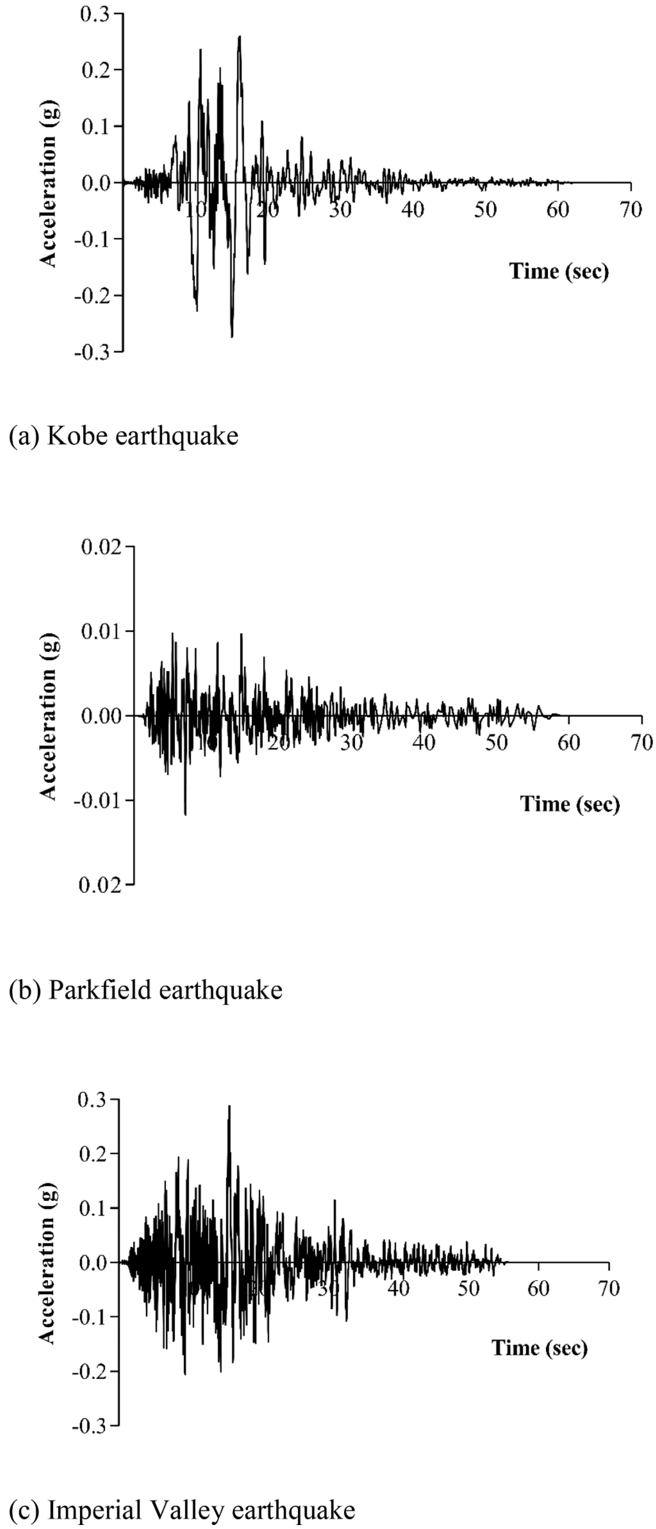


Fig. 2. Acceleration time histories of the considered earthquakes.

Table 1
Earthquake records used in the study.

Earthquake	Magnitude	PGA (g)	Station	Year
Kobe	6.9	0.27577	Kobe University	1995
Parkfield	6.19	0.01175	San Luis Obispo	1966
Imperial Valley	6.53	0.28726	Agrarias	1979

PGA - Peak Ground Acceleration.

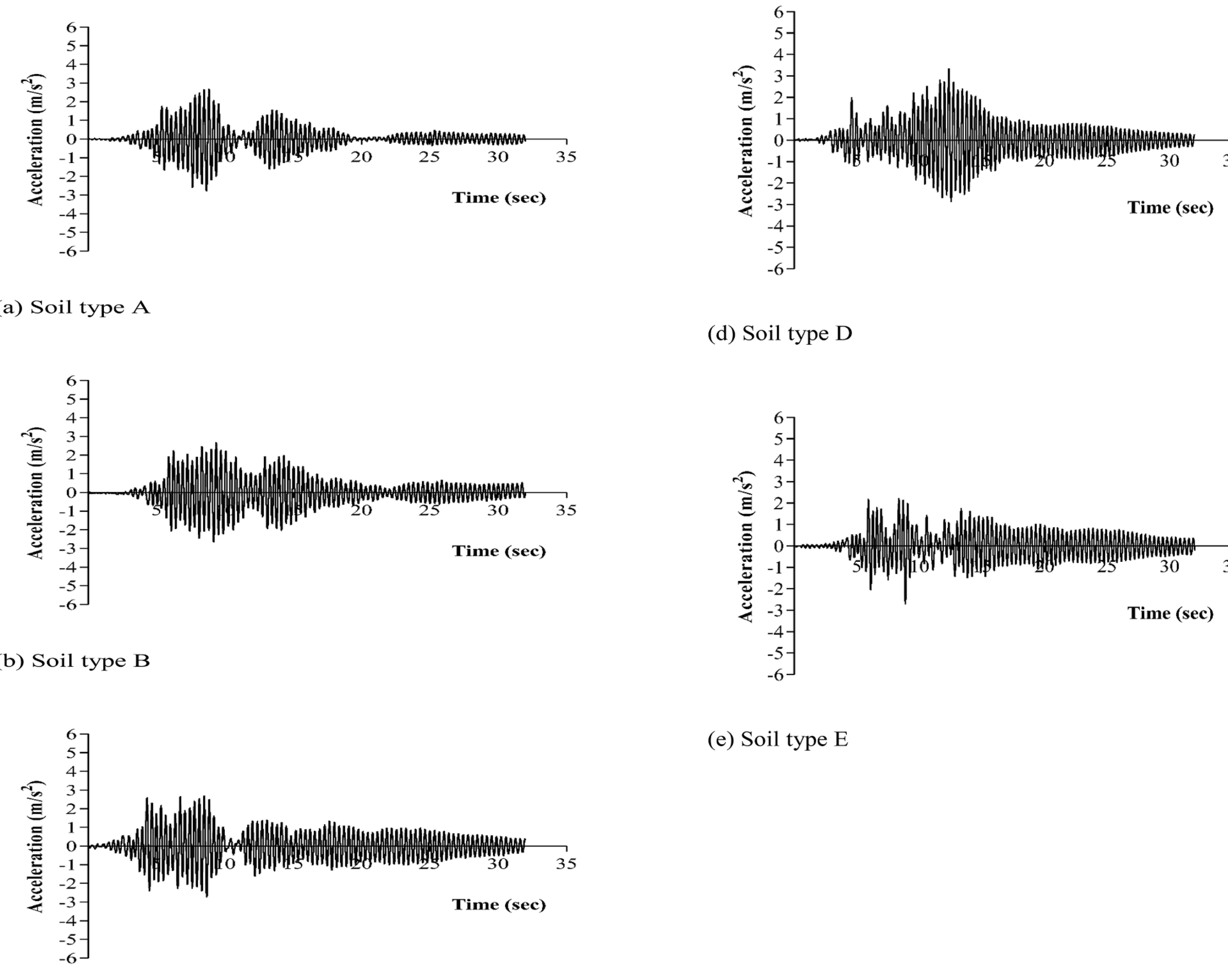
Table 2
Definition of the site classes.

Site class description	Site class definition
A	Hard rock
B	Rock
C	Very dense soil and soft rock
D	Stiff soil
E	Soft clay soil

seismic response of vibrating buildings under the influence of SSI (see [34–36], for example). Also, the nonlinearity should be considered in the soil-structure problems since considering linearity significantly underestimates the response and considering nonlinearity provides results with acceptable accuracy [37]. Furthermore, pounding was found to have substantial effects in the case of SSI [38] as well as in the case of soil-pile structure interaction [39]. Previous studies focusing on pounding showed contradictory results since pounding with SSI was found to increase the displacements, shear forces and impact forces in some studies [38,40–45] while it was found to decrease the displacements, shear forces and impact forces in other investigations (see [46–49] for example). Also, pounding can be more destructive to flexible buildings, as compared to stiffer structures considering SSI [40,41,46,50]. These contradictory results are referred to several factors including the soil types and foundation types used in different analyses. Moreover, the effect of the soil type on the response of colliding buildings was investigated only numerically and it was found that the soil type has a significant influence on the structural response [51–55]. Therefore, the aim of this study is to investigate experimentally the effect of the soil type on the response of colliding structures, based on shaking table tests. Two single-storey models of steel buildings with different dynamic parameters were considered in this study. Three pounding scenarios were taken into account by applying different seismic gaps (0.5 cm, 1 cm and 1.5 cm as well as the no pounding case). First, the effect of pounding on the response of these two structures was analysed by comparing several pounding scenarios with the no pounding case. Then, the effect of the seismic gap on the response of colliding structures was studied. Finally, the effect of soil type on the response of structures exposed to interactions was investigated. Five soil types were considered in the study, which are the five soil types defined in the ASCE 7–10 code (hard rock, rock, very dense soil and soft rock, stiff soil and soft clay soil) [56].

2. Methodology of the experiments

Two models were considered in this study (see Fig. 1). These two models are single-storey steel structures composed of welded rectangular hollow section elements (RHS 15 × 15 × 1.5 mm). The spacing between the columns is 0.465 m in the longitudinal direction and 0.556 m in the transverse one. The models have a height of 1.2 m. Diagonal bracings were used in the sidewall planes so as to counteract the transverse and torsional vibrations. Indeed, concrete plates (50 × 50 × 7 cm) of 47.56 kg each were used to simulate the weight of the slabs. Model 1 was loaded with one concrete plate (see Fig. 1a) while two concrete plates were used in the case of Model 2 (see Fig. 1b). Two accelerometers were mounted at the top of each structure so as to measure their accelerations during vibrations (see Fig. 1c). The third accelerometer was also installed at the shaking table platform and it was used for the verification purposes. Several values of the seismic gap were considered between the two models, which are: 0.5 cm (case 1), 1.0 cm (case 2), 1.5 cm (case 3) and 50 cm (which represents the no pounding case analysed for the comparison purposes) (see Fig. 1d and 1e). Three earthquakes, i.e. Kobe, Parkfield and Imperial Valley (see Fig. 2 and Table 1) downloaded from the PEER website [57], were used during the tests. The experiments were performed using the shaking table located at Gdańsk University of Technology, Poland. Some researchers have



(a) Soil type A

(b) Soil type B

(c) Soil type C

(d) Soil type D

(e) Soil type E

Fig. 3. Acceleration time history of Model 1 under different soil types exposed to the Kobe earthquake for the no pounding case.

discussed in details the practical procedure of creating the physical model (see [58], for example) as well as the soil mixture (see [59], for example) necessary to perform the shaking table tests. Several methods have been performed in the shaking table tests in the studies focussing on the effects of SSI such as the laminar soil containers (see [60,61] for example). In these methods and studies, the main concern was to study the effect of the SSI on the response of vibrating buildings. Hence, in the shaking table tests, the buildings were considered in the fixed-base conditions and in the SSI conditions and then the responses were compared. No pounding conditions were taken into account. In this study, the shaking table tests have been performed by adjusting the amplitude of the ground motion introduced to the generator to the real amplitude of the ground motion by trial and error. After adjusting the amplitude of the ground motions, the intensity of the generator for the target ground motion will be specified. The intensity of these earthquakes has been reduced to 25 %. In this study, the main concern is to study the effect of the soil type on the response of buildings experiencing collisions. To simulate the effect of the soil, these three earthquakes were scaled to the five soil types A, B, C, D and E defined in the ASCE 7–10 code (see Table 2) [56]. The site parameters were selected as follows: 0.5 for S_1 (mapped risk-targeted MCE_R spectral response acceleration parameter at 1-s period), 1.25 for S_s (mapped risk-targeted MCE_R spectral response acceleration parameter at short period), and 8 s for T_L (the long-period transition period) [51,52].

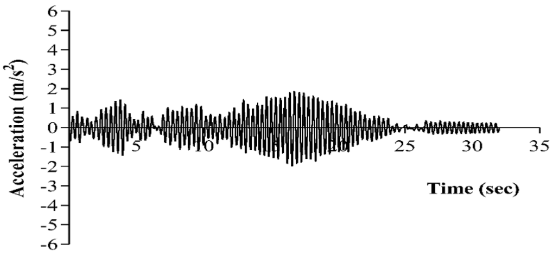
3. Results and discussion

3.1. Results

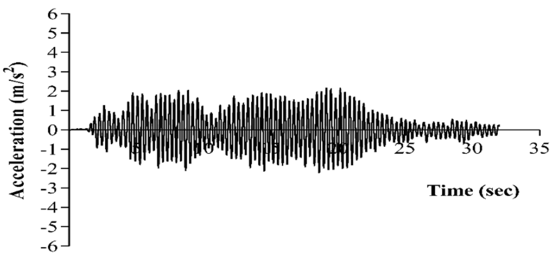
Figs. 3, 4 and 5 present the acceleration time history of Model 1 founded on different soil types for the no pounding case exposed to the Kobe, Parkfield and Imperial Valley earthquakes, respectively. It can be seen from the figures that the soil type has significantly affected the acceleration response of Model 1 under the three ground motions for the no pounding case.

Figs. 6, 7 and 8 present the acceleration time history of Model 2 founded on different soil types for the no pounding case exposed to the Kobe, Parkfield and Imperial Valley earthquakes, respectively. The results shown in the figures indicate that the soil type has also significantly affected the acceleration response of Model 2 under the three ground motions for the no pounding case.

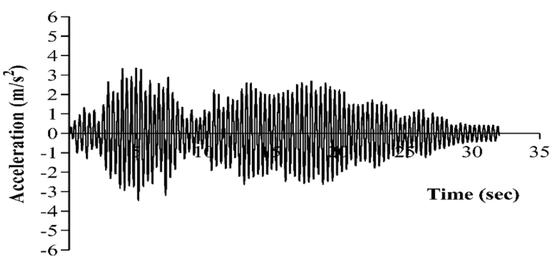
Figs. 9, 10 and 11 present the acceleration time history of Model 1



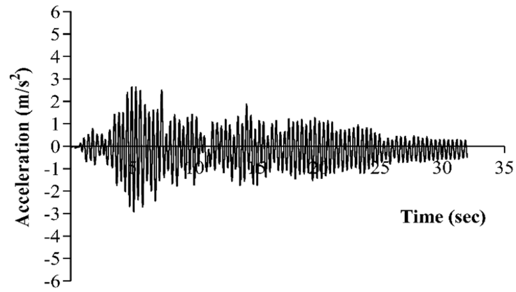
(a) Soil type A



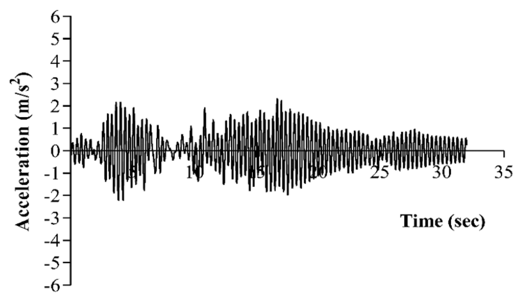
(b) Soil type B



(c) Soil type C



(d) Soil type D



(e) Soil type E

Fig. 4. Acceleration time history of Model 1 under different soil types exposed to the Parkfield earthquake for the no pounding case.

founded on different soil types for case 1 exposed to the Kobe, Parkfield and Imperial Valley earthquakes, respectively. The results presented in these figures show that the soil type has significantly affected the acceleration response of Model 1 under the three ground motions for this pounding case. Moreover, by comparing the results presented in Figs. 9-11 with the results shown in Figs. 3-5, it can be concluded also that the effect of the soil type is more significant for Model 1 in the case when pounding takes place.

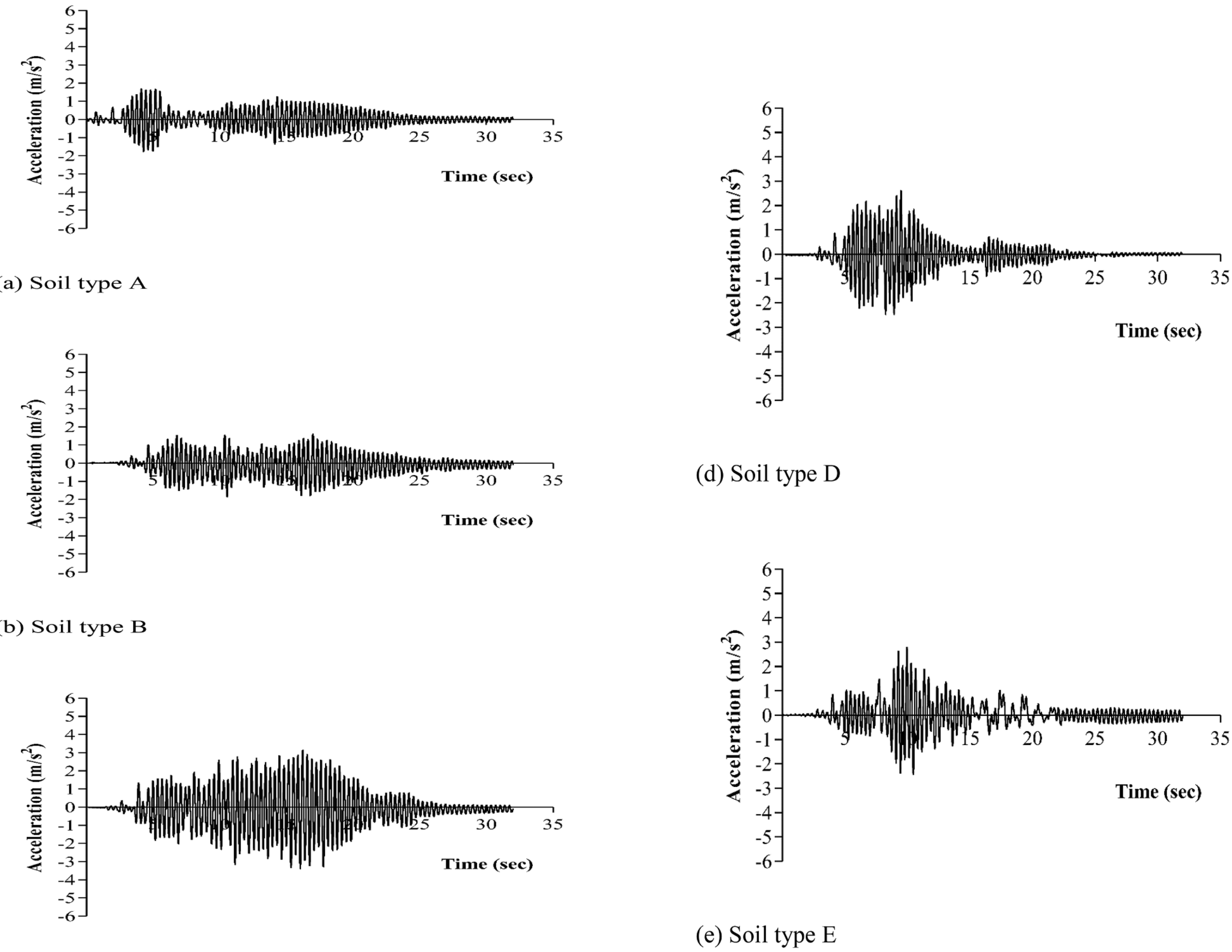
Figs. 12, 13 and 14 present the acceleration time history of Model 2 founded on different soil types for case 1 exposed to the Kobe, Parkfield and Imperial Valley earthquakes, respectively. The results presented in these figures indicate that the soil type has also significantly affected the acceleration response of Model 2 under the three ground motions for this pounding case. Furthermore, by comparing the results presented in Figs. 12-14 with the results shown in Figs. 6-8, it can also be concluded that the effect of the soil type is more significant for Model 2 in the case when pounding takes place.

Figs. 15, 16 and 17 present the acceleration time history of Model 1 founded on different soil types for case 2 exposed to the Kobe, Parkfield and Imperial Valley earthquakes, respectively. The results presented in these figures confirm previous findings that the soil type has a significant effect on the acceleration response of Model 1 and this significance is more substantial when pounding takes place. Also, by comparing the results presented in Figs. 15-17 with the results shown in Figs. 9-11, it can be seen that the value of the seismic gap has a significant effect on the acceleration response of Model 1.

Figs. 18, 19 and 20 present the acceleration time history of Model 2 founded on different soil types for case 2 exposed to the Kobe, Parkfield and Imperial Valley earthquakes, respectively. Also, the results presented in these figures illustrate that the soil type has a significant effect on the acceleration response of Model 2 and this significance is more substantial when pounding takes place. By comparing the results of Figs. 18-20 with the results presented in Figs. 12-14, it can be seen that the value of the seismic gap has a significant effect on the acceleration response of Model 2.

Figs. 21, 22 and 23 present the acceleration time history of Model 1 founded on different soil types for case 3 exposed to the Kobe, Parkfield and Imperial Valley earthquakes, respectively. The results presented in these figures confirm previous findings that the soil type has a significant effect on the acceleration response of Model 1 (and this significance is more substantial when pounding takes place) as well as that the seismic gap has a significant effect on the acceleration response of Model 1.

Finally, Figs. 24, 25 and 26 present the acceleration time history of Model 2 founded on different soil types for case 3 exposed to the Kobe, Parkfield and Imperial Valley earthquakes, respectively. Also, the results presented in these figures illustrate that the soil type has a significant effect on the acceleration response of Model 2 (and this significance is more substantial when pounding takes place) as well as that the seismic gap has a significant effect on the acceleration response of Model 2.



(a) Soil type A

(b) Soil type B

(c) Soil type C

(d) Soil type D

(e) Soil type E

Fig. 5. Acceleration time history of Model 1 under different soil types exposed to the Imperial valley earthquake for the no pounding case.

3.2. *Effect of pounding and the seismic gap on the response of colliding structures*

Tables 3 and 4 present the peak acceleration values of Models 1 and 2, respectively, for different poundings scenarios (cases 1, 2, and 3 as well as for the no pounding case). Additionally, the tables also show the ratios between the peak acceleration for each pounding scenario and the peak acceleration for the no pounding case. The results show that these ratios range between 1 and 12 for Model 1 and between 1 and 6 for Model 2. This means that pounding significantly increases the peak acceleration of the colliding structures in all cases. Furthermore, by comparing the peak accelerations of Model 1 and Model 2 for cases 1, 2 and 3 under the three ground motions (Tables 3 and 4), it can be seen that Model 1 has experienced higher peak accelerations than Model 2 for all three pounding cases. Indeed, by comparing the ratio (ratio of the peak acceleration of the pounding case to that of the no pounding case) for Models 1 and 2, it can be seen that the ratios of Model 1 are higher than those of Model 2. This means that pounding is more significant for Model 1 (flexible structure) than for Model 2 (stiff structure).

Moreover, it can be seen from Tables 3 and 4 that the seismic gap has a significant effect on the peak acceleration of colliding structures. For instance, Model 1 founded on soil type C and exposed to the Kobe earthquake has experienced a peak acceleration of 20.26 m/s^2 for case 1, 19.26 m/s^2 for case 2 and 31.79 m/s^2 for case 3. Indeed, in certain cases, increasing the gap leads to the decrease in the peak acceleration. For example, Model 1 founded on soil type E and exposed to the Parkfield earthquake has experienced a peak acceleration of 26.21 m/s^2 for case 1, 20.73 m/s^2 for case 2 and 19.81 m/s^2 for case 3. However, in other cases, increasing the gap leads to the increase of the peak acceleration. For instance, Model 1 founded on soil type D and exposed to the Kobe earthquake has experienced a peak acceleration of 17.76 m/s^2 for case 1, 21.08 m/s^2 for case 2 and 24.54 m/s^2 for case 3. It means that increasing the gap will not necessarily lead to lower responses unless it is large enough to eliminate collisions at all.

3.3. *Effect of the soil type on the response of colliding structures*

Tables 5 and 6 present the peak acceleration values of Model 1 and Model 2, respectively, for different poundings scenarios (cases 1, 2, and 3 as well as for the no pounding case) founded on different soil types. It can be clearly seen from the tables that the soil type has significantly affected the peak acceleration of the colliding structures. For instance, Model 2 under the Parkfield earthquake has experienced in case 3 a peak acceleration of 13.11 m/s^2 when founded on soil type A, 10.32 m/s^2 when founded on soil type B, 16.66 m/s^2 when founded on soil type C, 26.90 m/s^2 when founded on soil type D, and 19.81 m/s^2 when founded

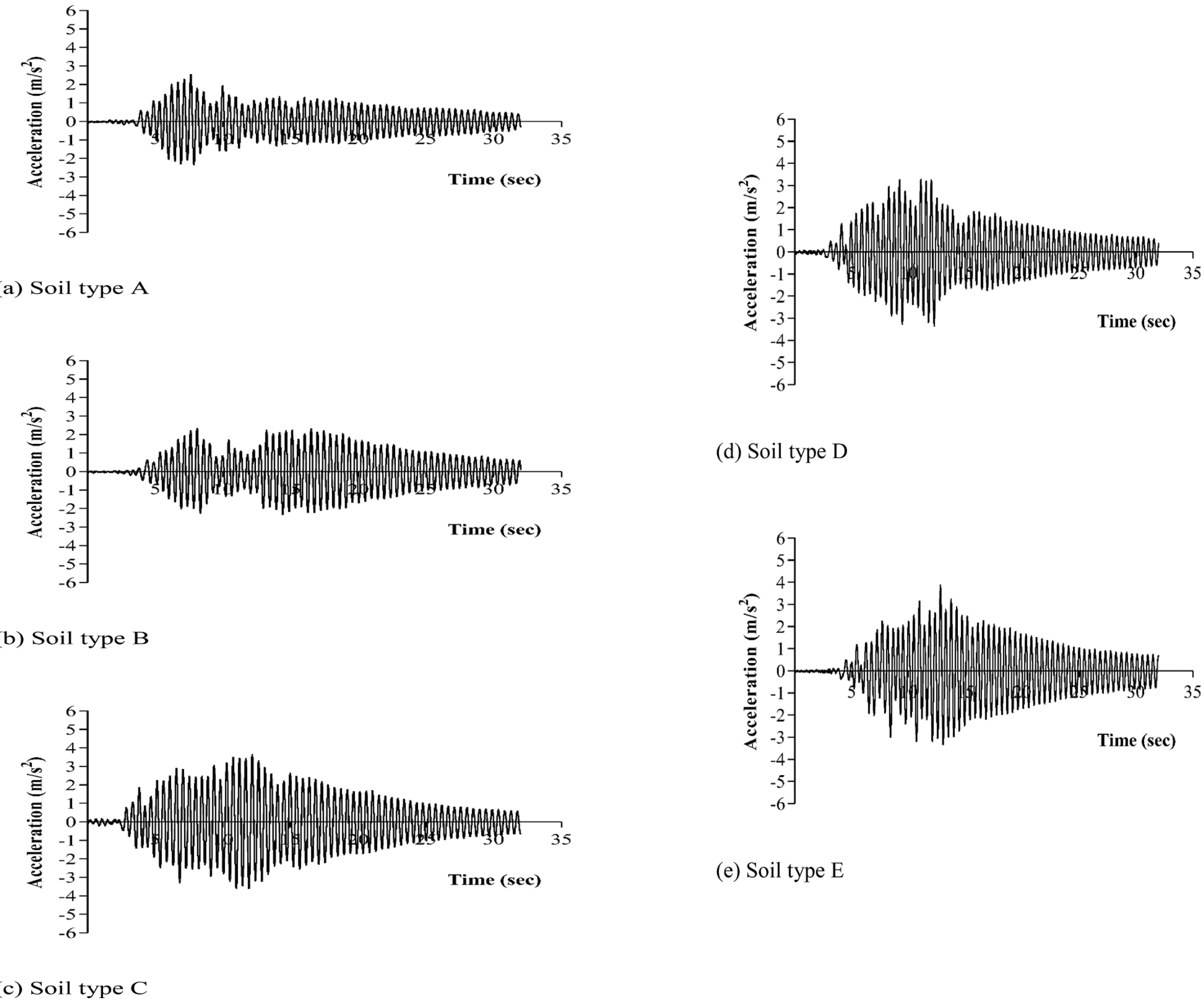


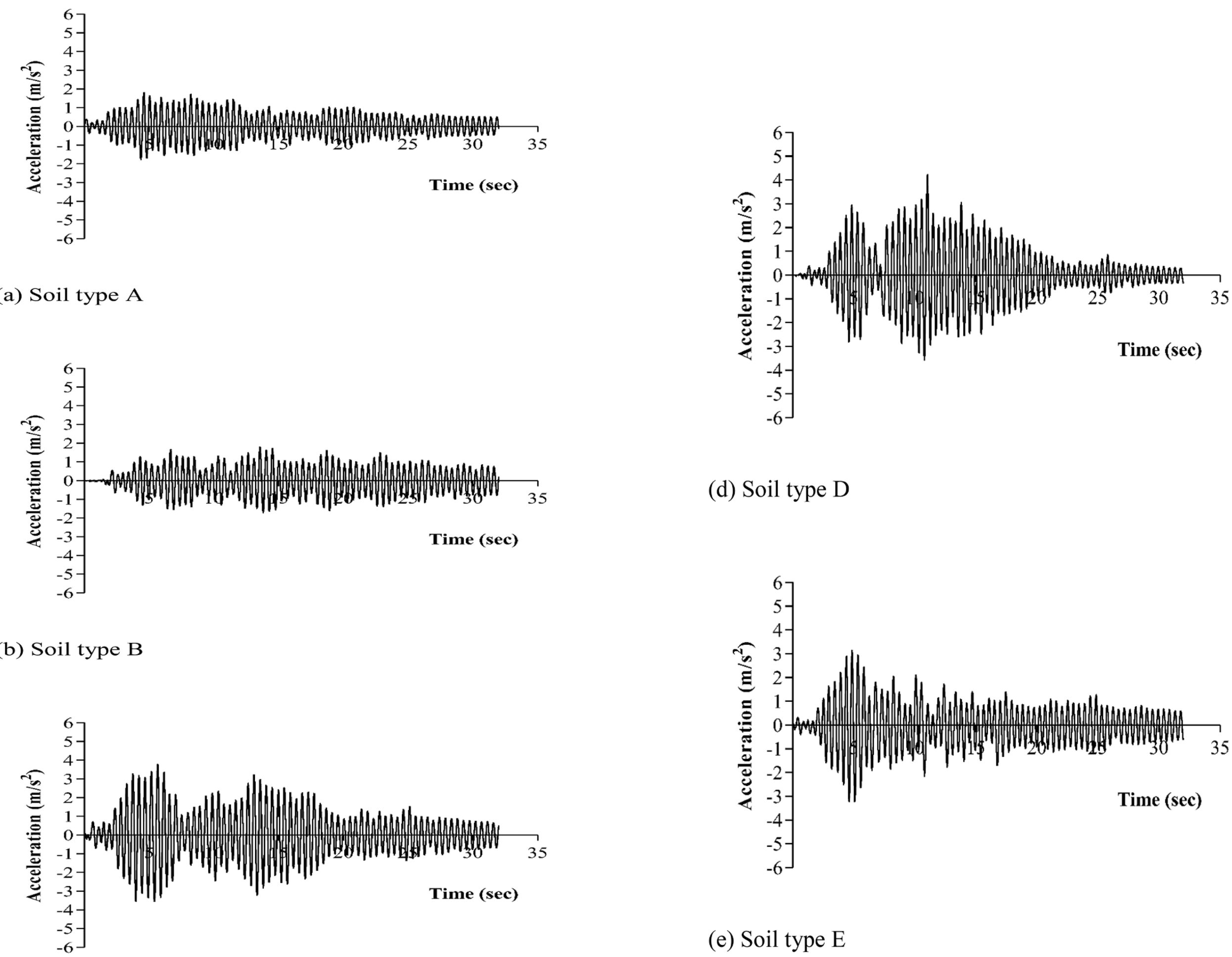
Fig. 6. Acceleration time history of Model 2 under different soil types exposed to the Kobe earthquake for the no pounding case.

on soil type E. Indeed, the soil type that leads to the maximum peak acceleration is not the same in all cases and the results are different for various pounding scenarios and ground motions. The same also applies to the minimum peak acceleration. For instance, Model 1 exposed to the Kobe earthquake has experienced the maximum peak acceleration when it was founded on soil type C for cases 1 and 3 and when it was founded on soil type B for case 2. Indeed, Model 1 under the Parkfield earthquake has experienced the maximum peak acceleration when it was founded on soil type E for case 1 and when it was founded on soil type D for cases 2 and 3. Moreover, Model 1 exposed to the Kobe earthquake has experienced the minimum peak acceleration when it was founded on soil type B for case 1 and when it was founded on soil type A for cases 2 and 3. Therefore, it can be concluded that the soil type has a significant effect on the response of colliding structures. Also, the maximum and minimum peak acceleration differs for various soil types, pounding scenarios, seismic gaps and earthquakes.

4. Conclusions

This study investigated experimentally the effect of the soil type on the response of colliding structures exposed to earthquakes. Two single-storey models of steel buildings with different dynamic parameters were considered in the shaking table tests. Three pounding scenarios were taken into account by applying different seismic gaps (0.5 cm, 1 cm and 1.5 cm as well as the no pounding case). First, the effect of pounding on the response of these two structures was analysed by comparing several pounding scenarios with the no pounding case. Then, the effect of the seismic gap on the response of colliding structures was studied. Finally, the effect of soil type on the response of structures exposed to interactions was investigated. Five soil types were considered in the study, which are the five soil types defined in the ASCE 7–10 code (hard rock, rock, very dense soil and soft rock, stiff soil and soft clay soil). The main conclusions of this study are:

- Pounding significantly increases the level of accelerations of structures during the whole time of vibrations as well as the peak acceleration itself. The ratio between the peak acceleration of the colliding structure to that of the no pounding case has ranged between 1 and 12 for Model 1 and 1 and 6 for Model 2.
- Pounding is more significant for the flexible structure (Model 1), as compared to the stiff one (Model 2).
- The seismic gap has a significant effect on the acceleration response of colliding structures. Increasing the gap does not necessarily lead to lower responses unless it is large enough to eliminate collisions at all.
- The soil type has a significant effect on the response of colliding structures. The soil type effect is more significant when pounding takes place.



(a) Soil type A

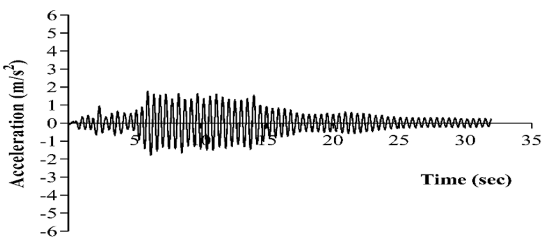
(b) Soil type B

(c) Soil type C

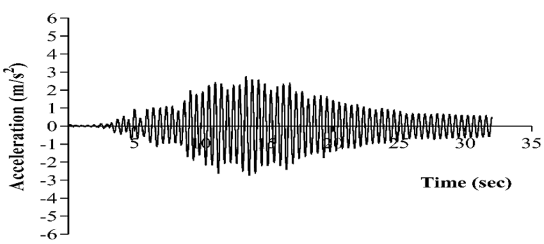
(d) Soil type D

(e) Soil type E

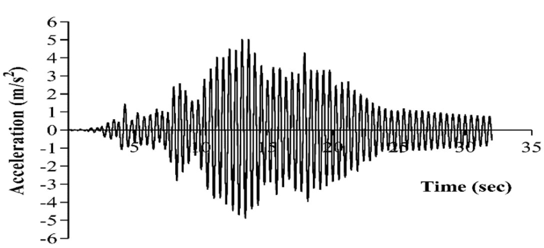
Fig. 7. Acceleration time history of Model 2 under different soil types exposed to the Parkfield earthquake for the no pounding case.



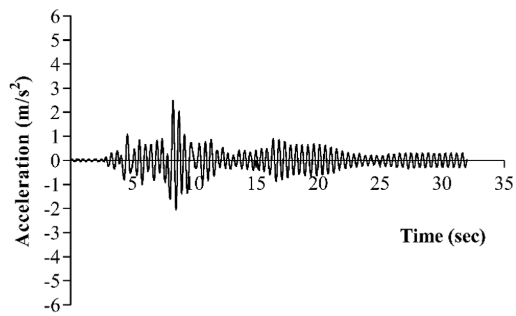
(a) Soil type A



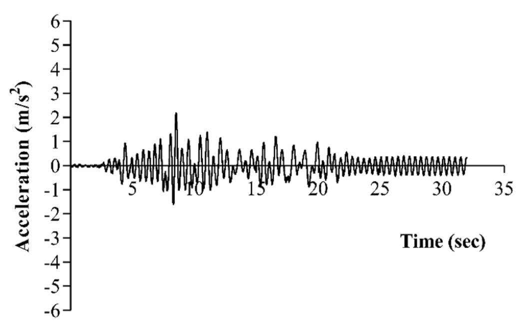
(b) Soil type B



(c) Soil type C



(d) Soil type D



(e) Soil type E

Fig. 8. Acceleration time history of Model 2 under different soil types exposed to the Imperial valley earthquake for the no pounding case.

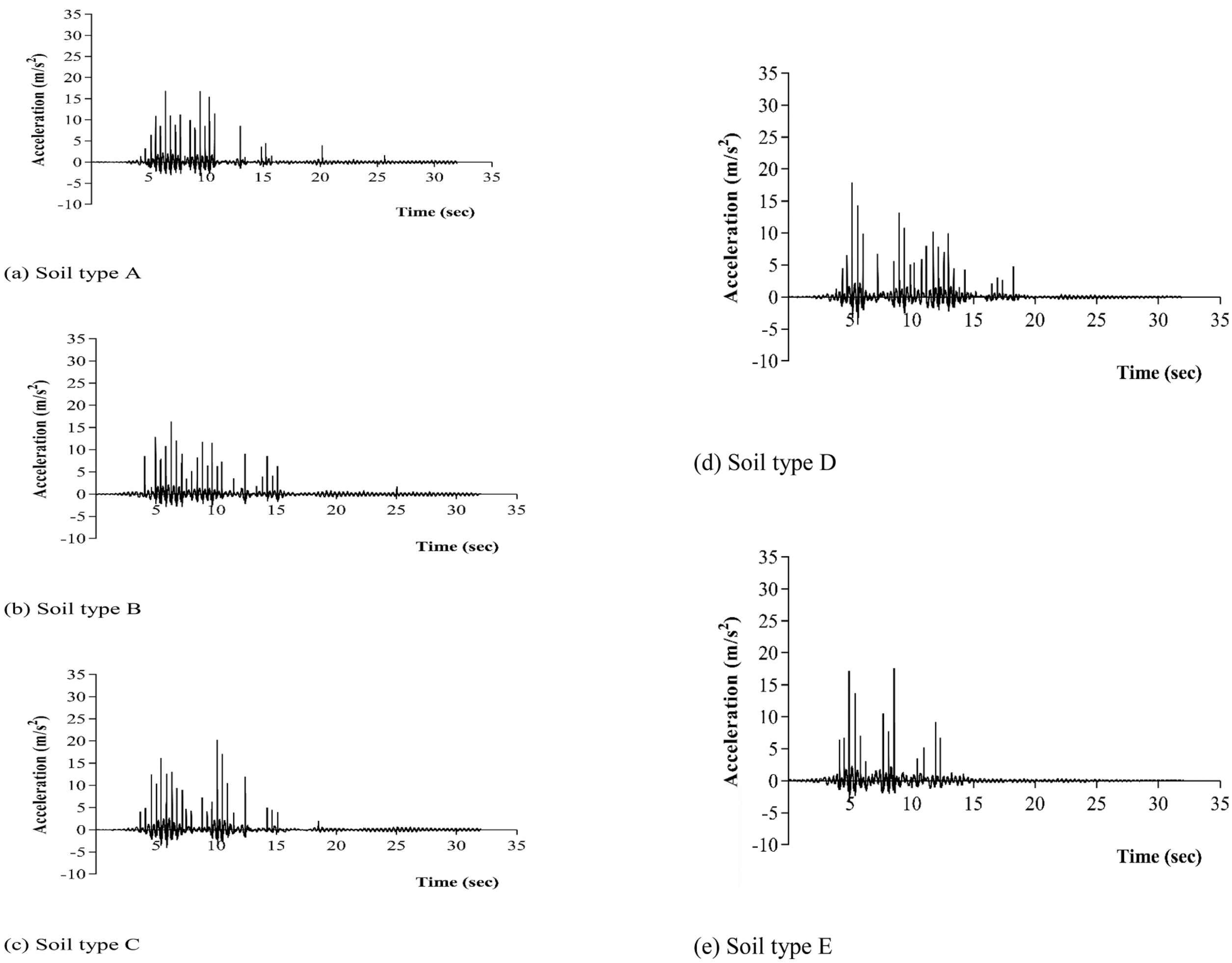
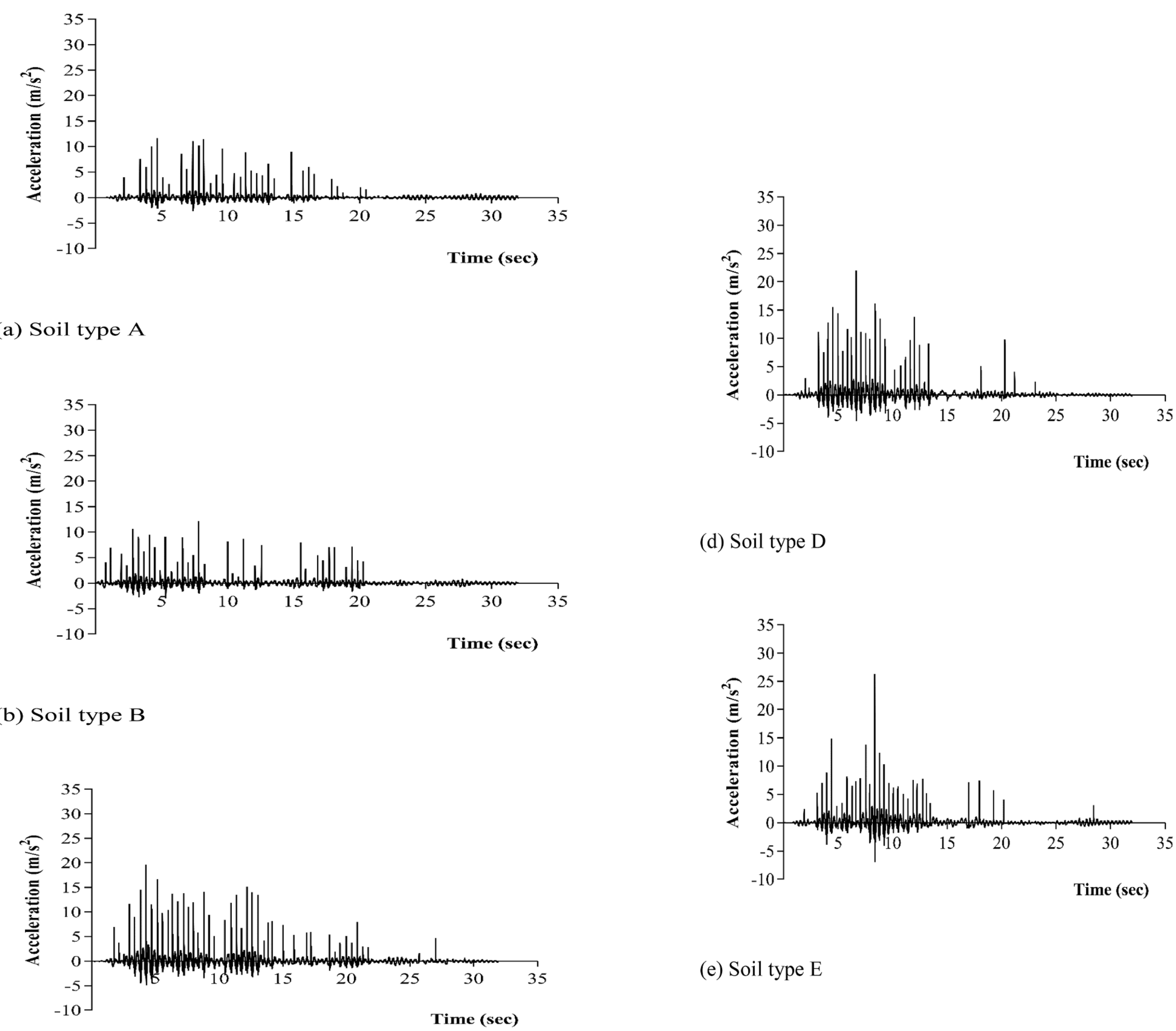


Fig. 9. Acceleration time history of Model 1 under different soil types exposed to the Kobe earthquake for case 1.



(a) Soil type A

(b) Soil type B

(c) Soil type C

(d) Soil type D

(e) Soil type E

Fig. 10. Acceleration time history of Model 1 under different soil types exposed to the Parkfield earthquake for case 1.

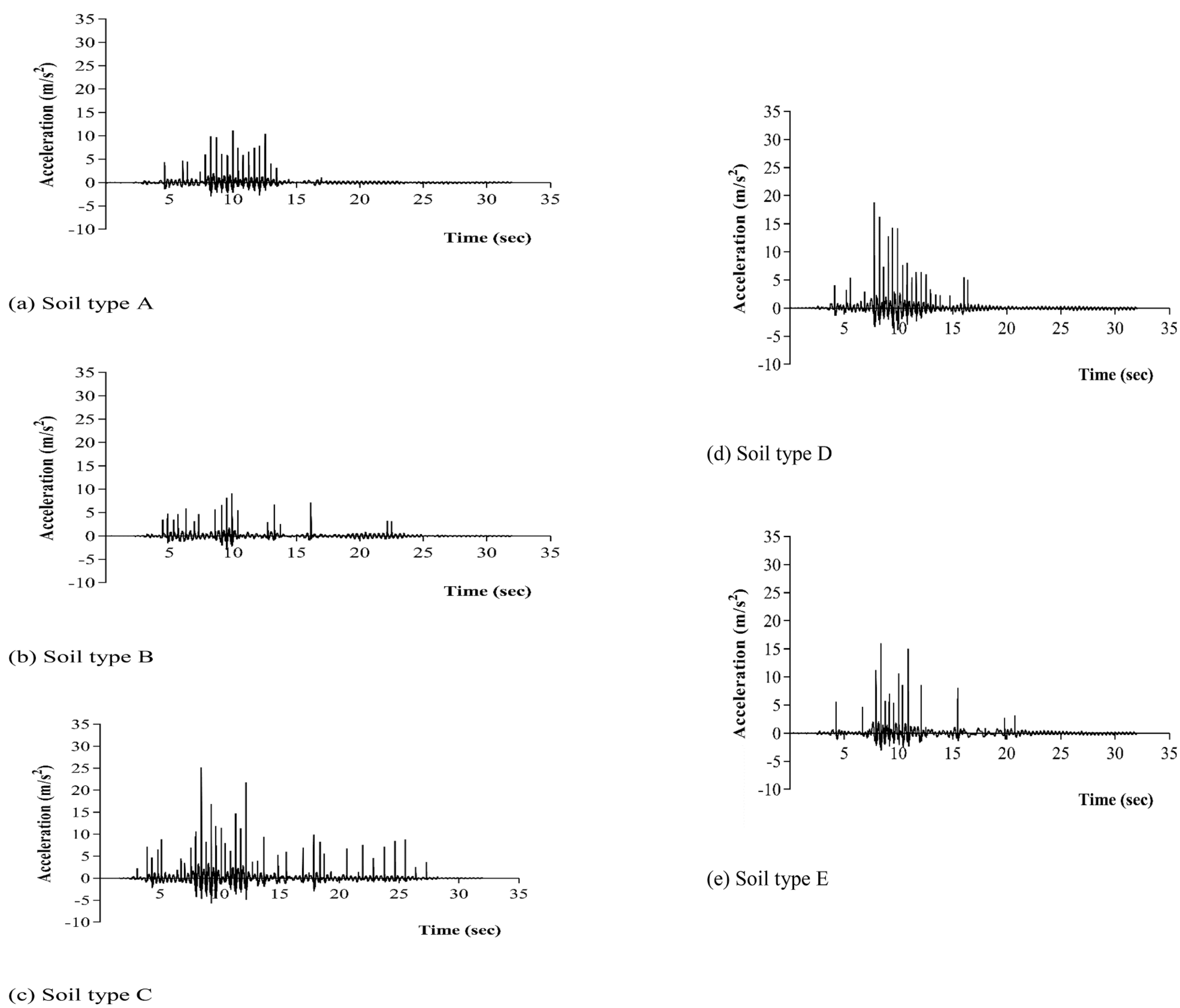
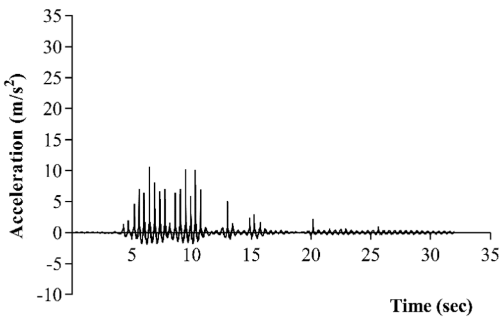
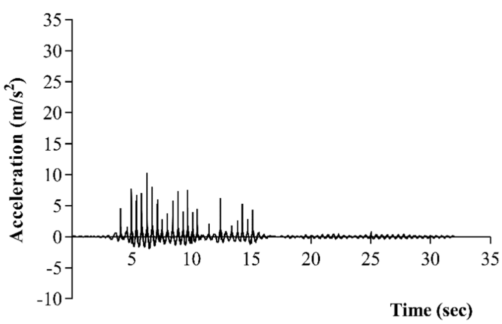


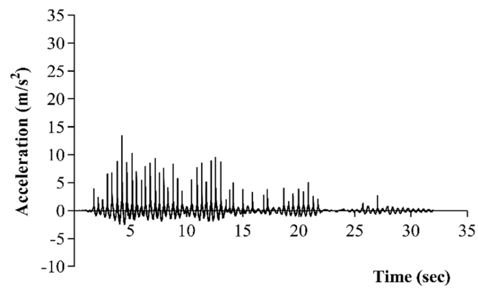
Fig. 11. Acceleration time history of Model 1 under different soil types exposed to the Imperial valley earthquake for case 1.



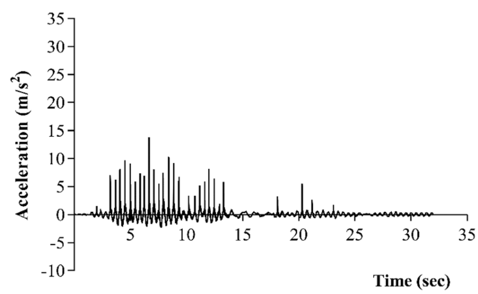
(a) Soil type A



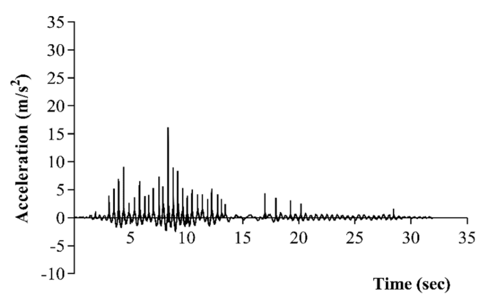
(b) Soil type B



(c) Soil type C

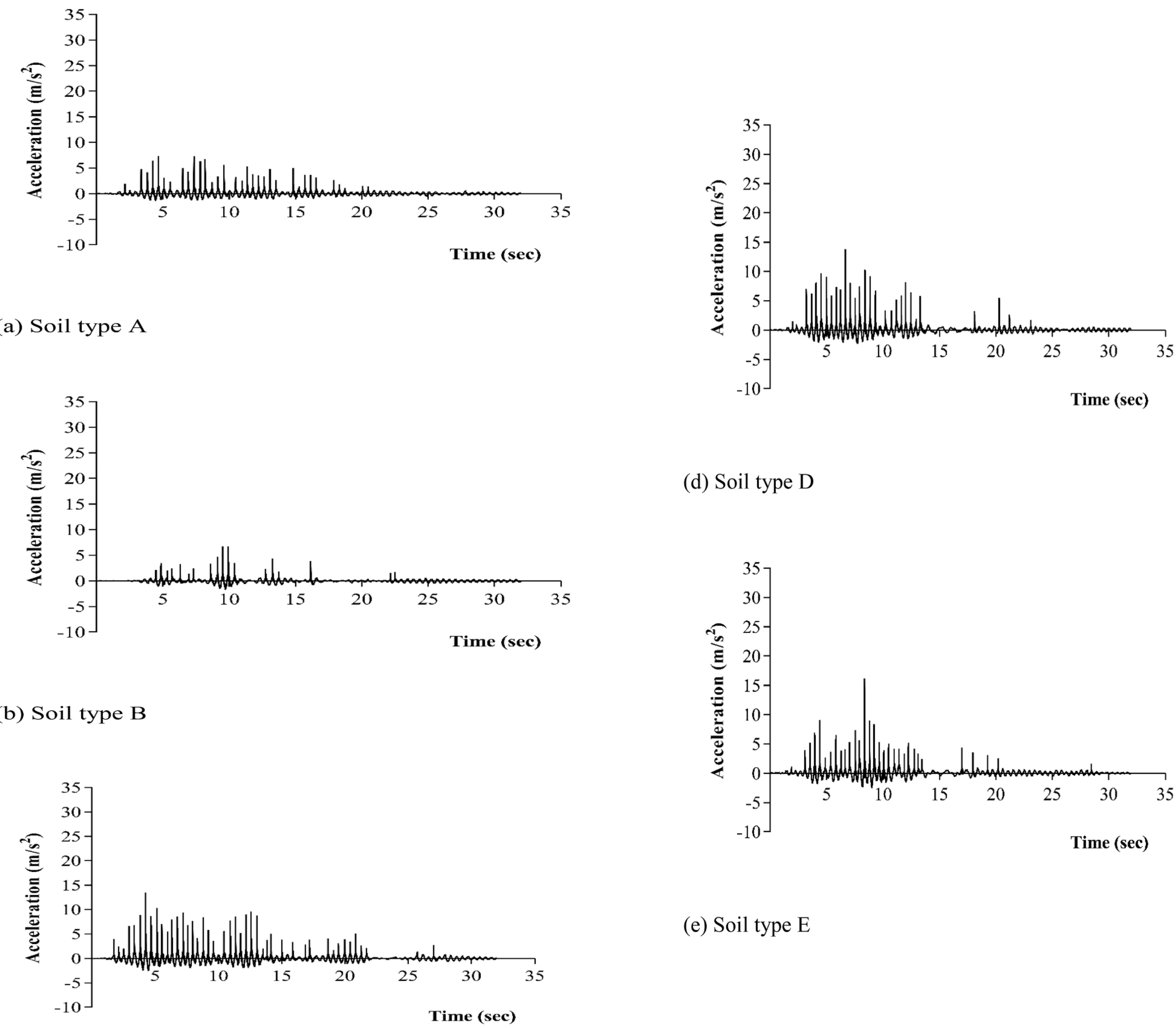


(d) Soil type D



(e) Soil type E

Fig. 12. Acceleration time history of Model 2 under different soil types exposed to the Kobe earthquake for case 1.



(a) Soil type A

(b) Soil type B

(c) Soil type C

(d) Soil type D

(e) Soil type E

Fig. 13. Acceleration time history of Model 2 under different soil types exposed to the Parkfield earthquake for case 1.

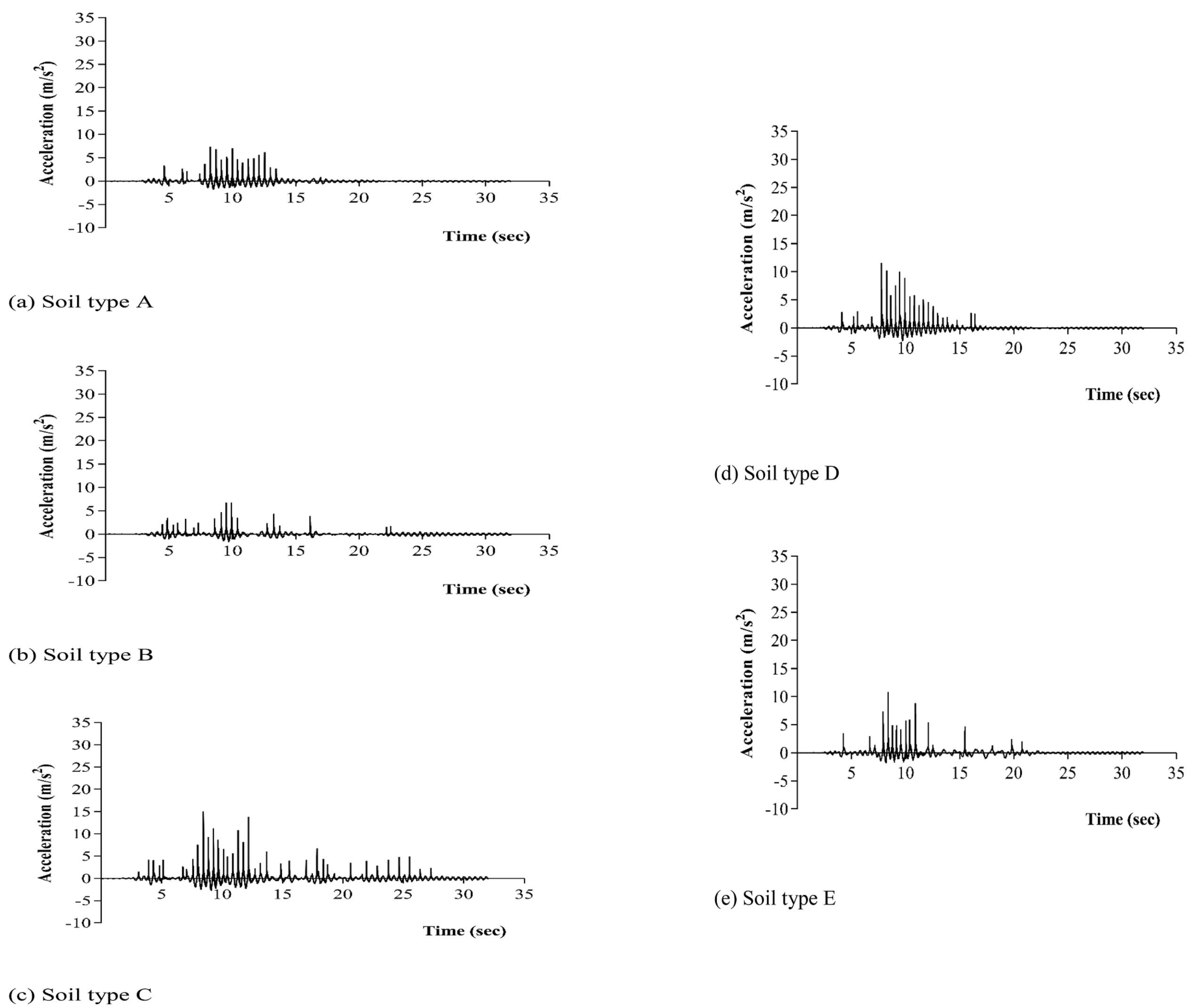
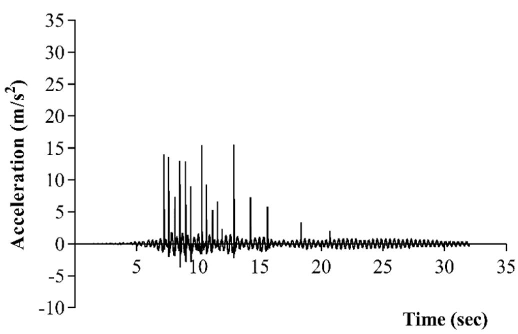
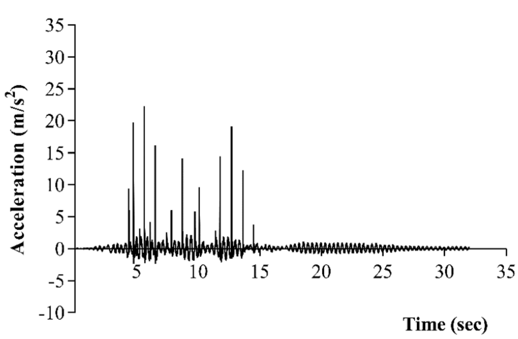


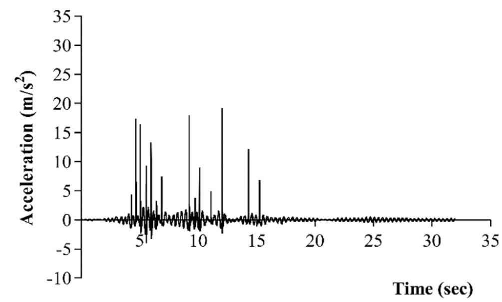
Fig. 14. Acceleration time history of Model 2 under different soil types exposed to the Imperial valley earthquake for case 1.



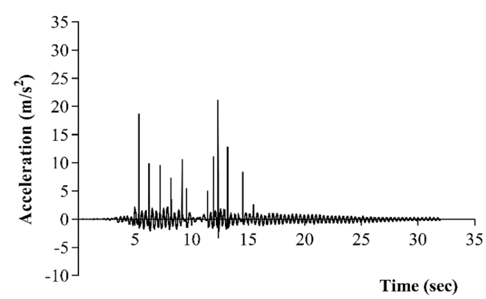
(a) Soil type A



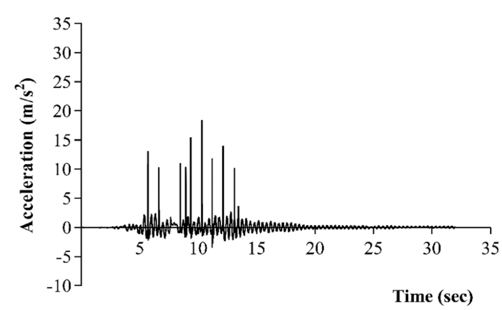
(b) Soil type B



(c) Soil type C

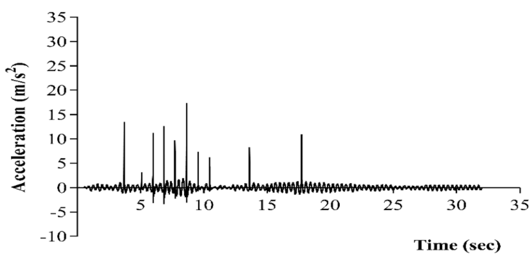


(d) Soil type D

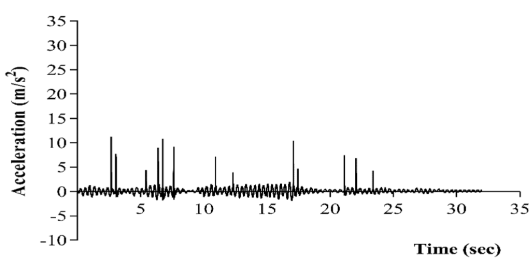


(e) Soil type E

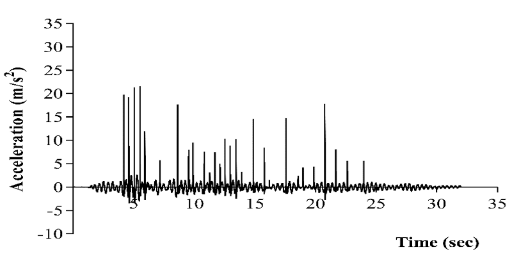
Fig. 15. Acceleration time history of Model 1 under different soil types exposed to the Kobe earthquake for case 2.



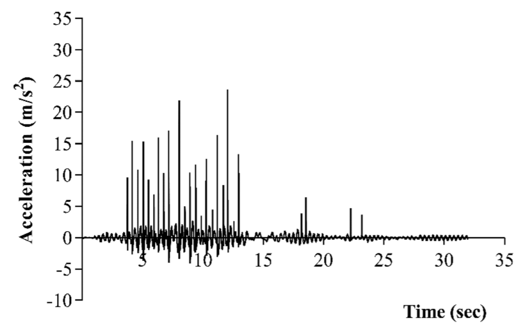
(a) Soil type A



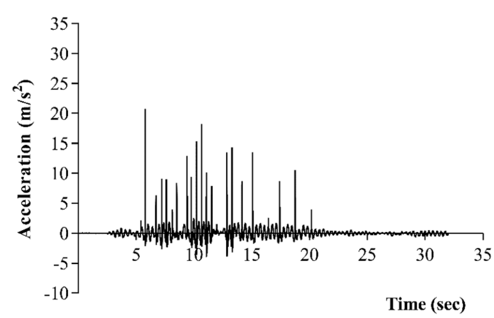
(b) Soil type B



(c) Soil type C

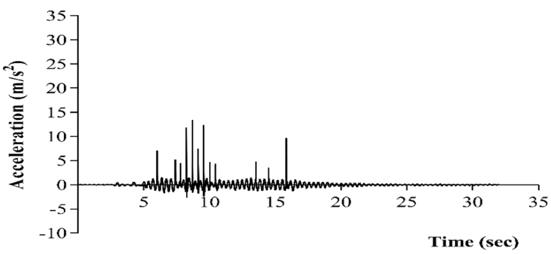


(d) Soil type D

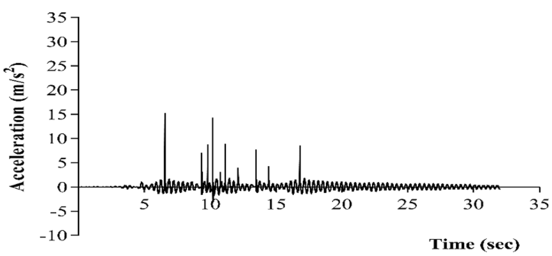


(e) Soil type E

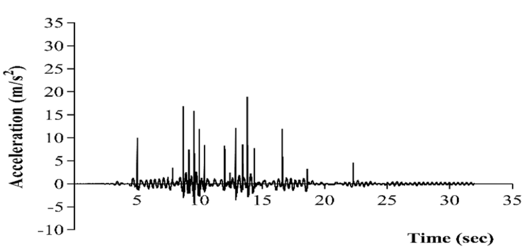
Fig. 16. Acceleration time history of Model 1 under different soil types exposed to the Parkfield earthquake for case 2.



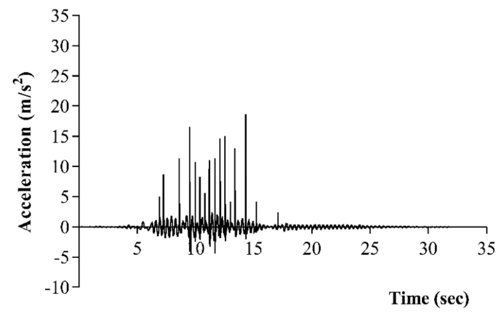
(a) Soil type A



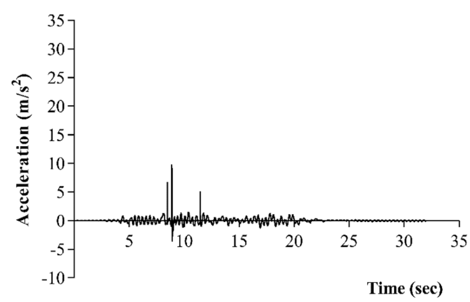
(b) Soil type B



(c) Soil type C

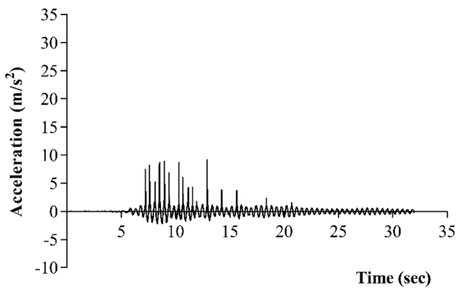


(d) Soil type D

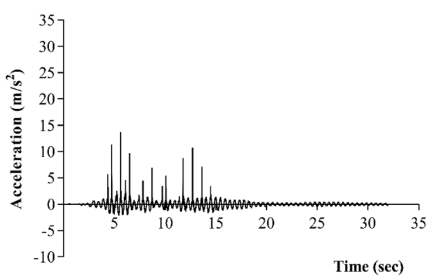


(e) Soil type E

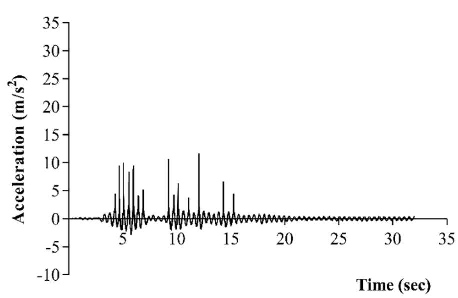
Fig. 17. Acceleration time history of Model 1 under different soil types exposed to the Imperial valley earthquake for case 2.



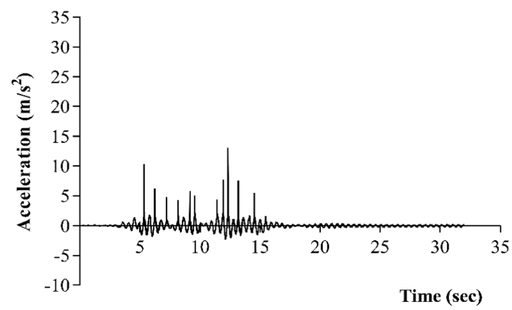
(a) Soil type A



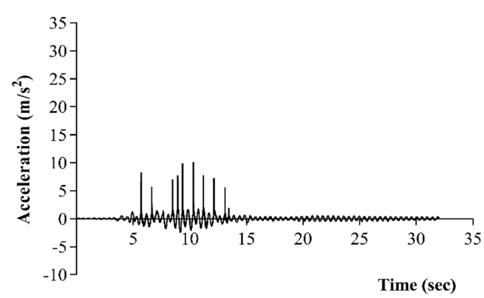
(b) Soil type B



(c) Soil type C

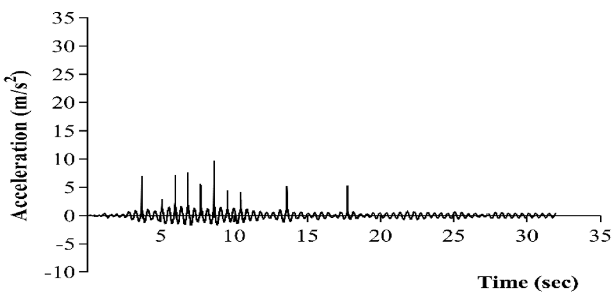


(d) Soil type D

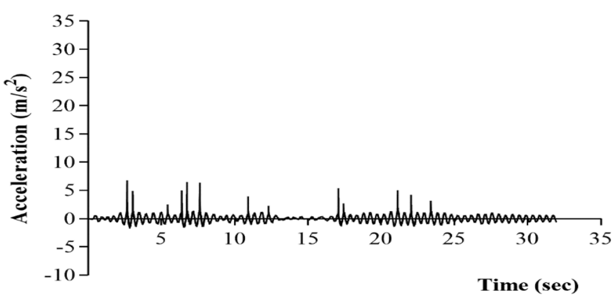


(e) Soil type E

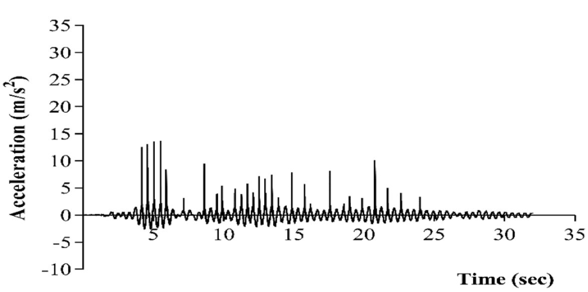
Fig. 18. Acceleration time history of Model 2 under different soil types exposed to the Kobe earthquake for case 2.



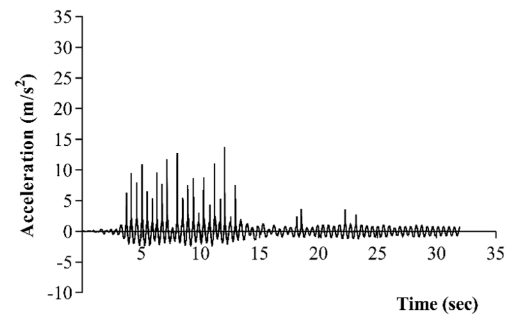
(a) Soil type A



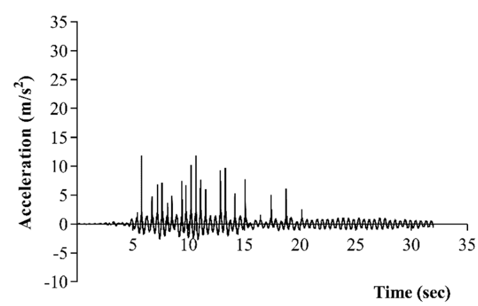
(b) Soil type B



(c) Soil type C



(d) Soil type D



(e) Soil type E

Fig. 19. Acceleration time history of Model 2 under different soil types exposed to the Parkfield earthquake for case 2.

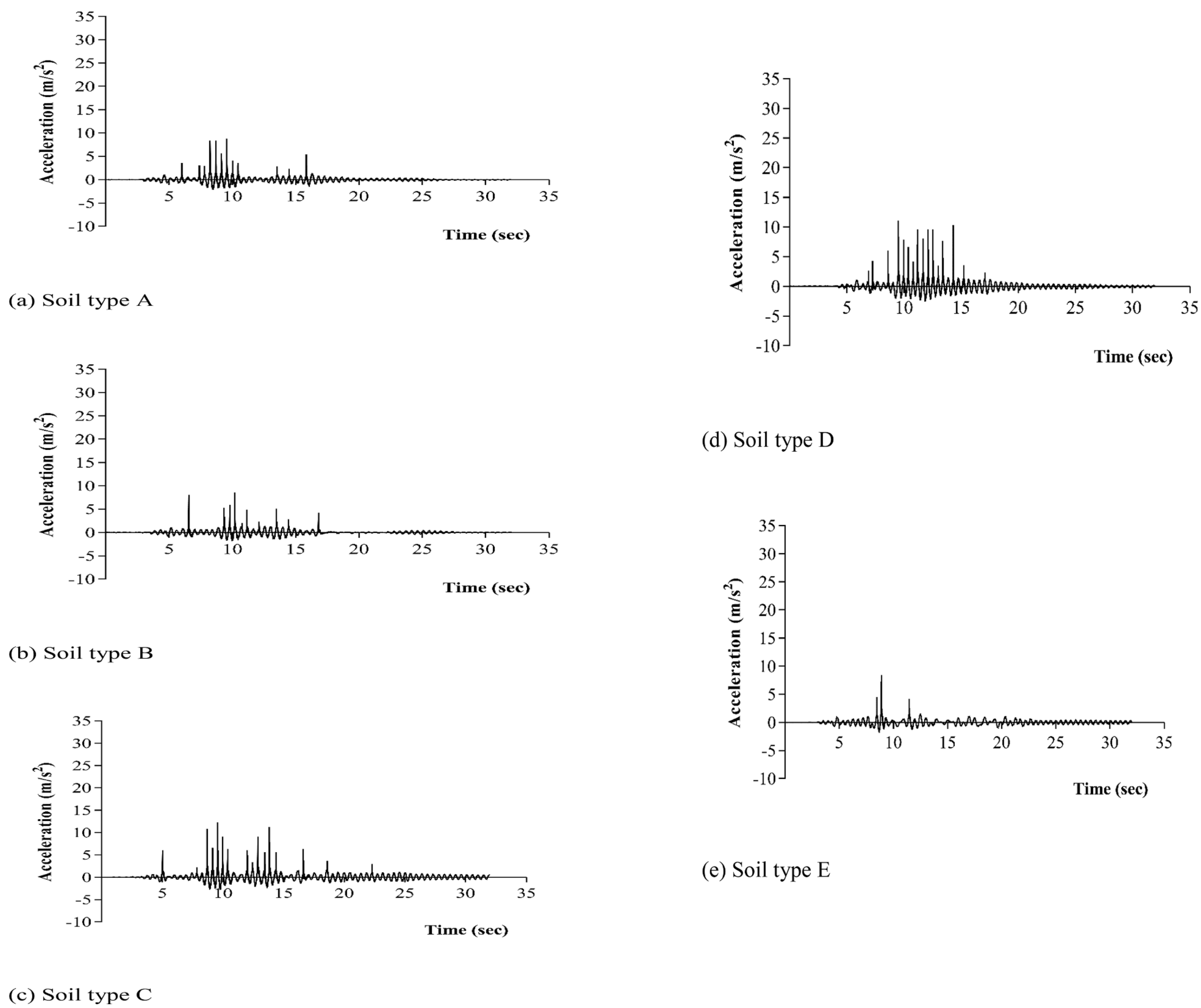
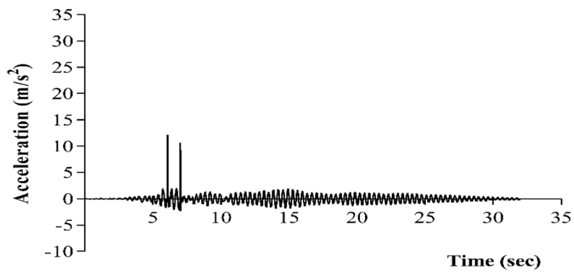
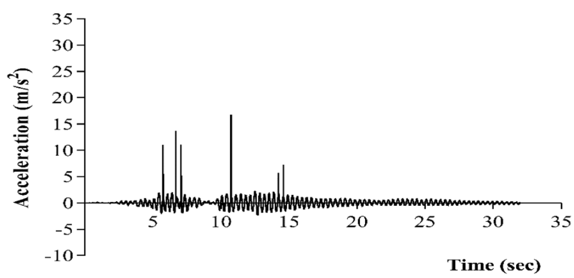


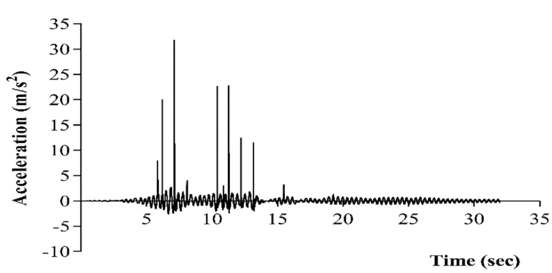
Fig. 20. Acceleration time history of Model 2 under different soil types exposed to the Imperial valley earthquake for case 2.



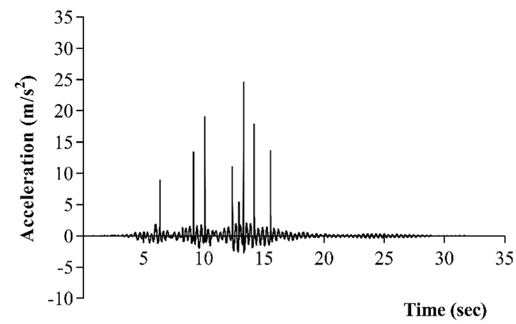
(a) Soil type A



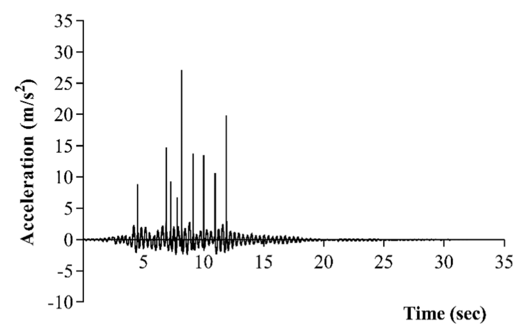
(b) Soil type B



(c) Soil type C

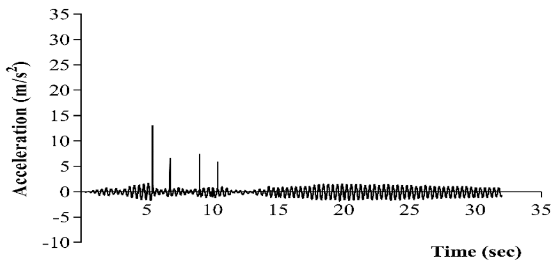


(d) Soil type D

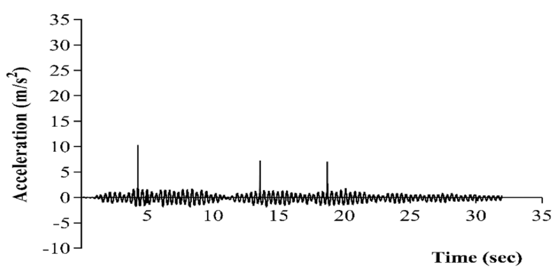


(e) Soil type E

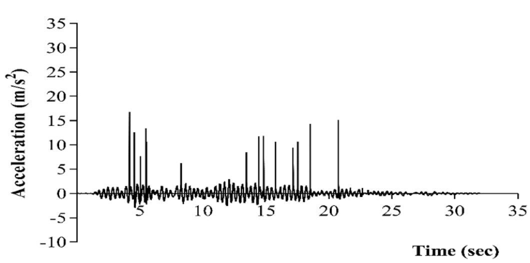
Fig. 21. Acceleration time history of Model 1 under different soil types exposed to the Kobe earthquake for case 3.



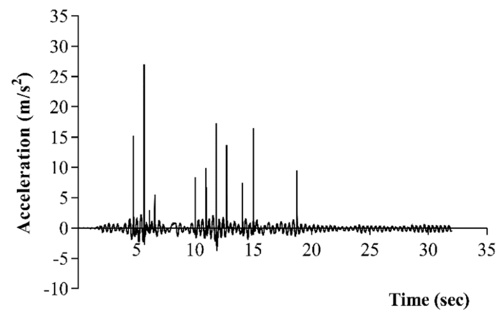
(a) Soil type A



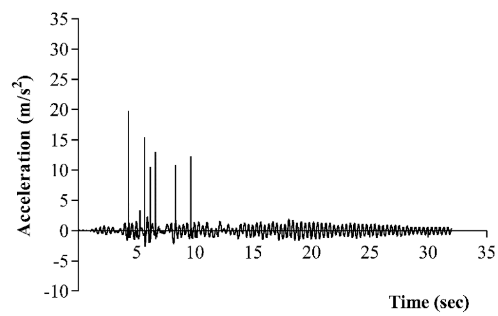
(b) Soil type B



(c) Soil type C

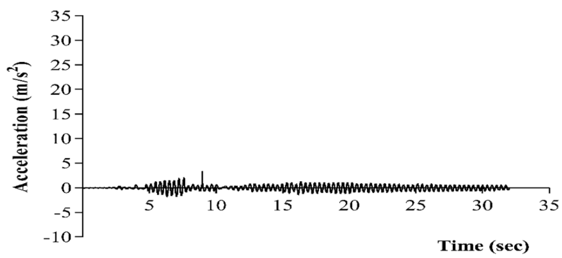


(d) Soil type D

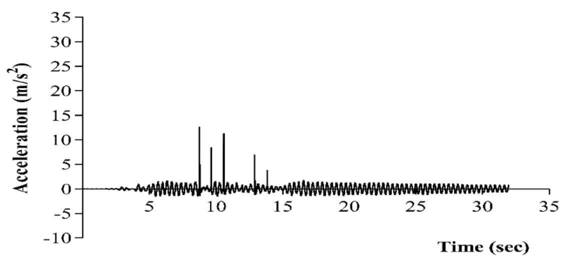


(e) Soil type E

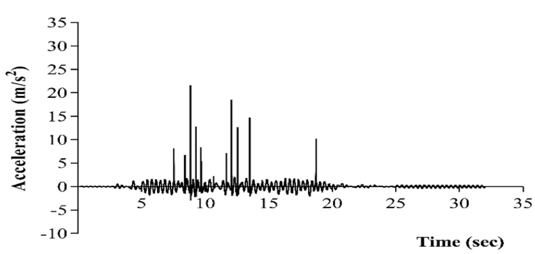
Fig. 22. Acceleration time history of Model 1 under different soil types exposed to the Parkfield earthquake for case 3.



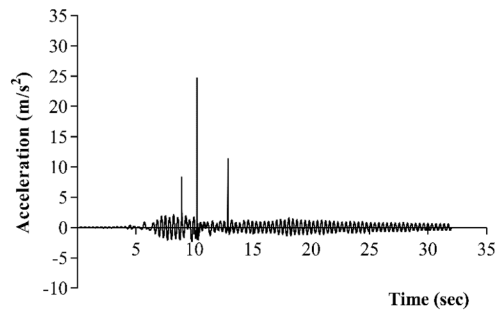
(a) Soil type A



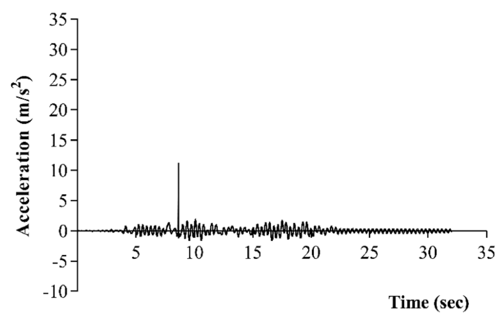
(b) Soil type B



(c) Soil type C

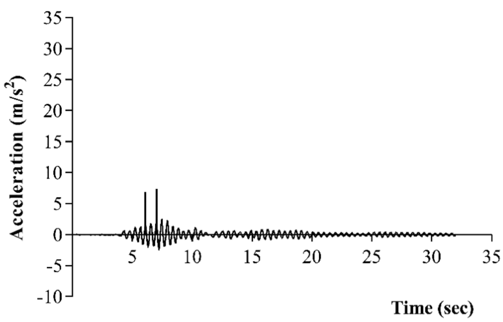


(d) Soil type D

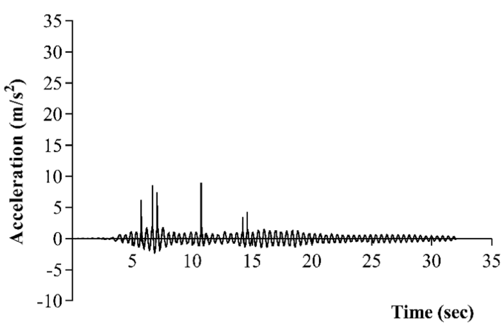


(e) Soil type E

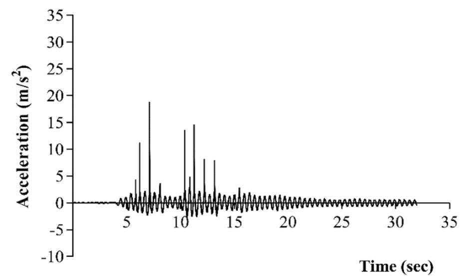
Fig. 23. Acceleration time history of Model 1 under different soil types exposed to the Imperial valley earthquake for case 3.



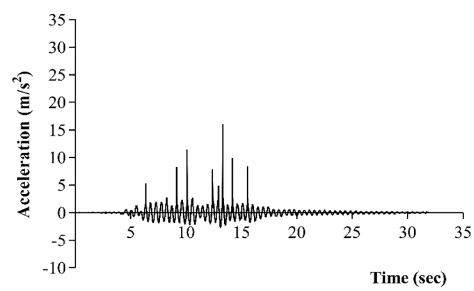
(a) Soil type A



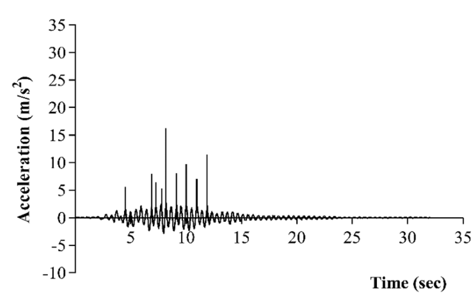
(b) Soil type B



(c) Soil type C



(d) Soil type D



(e) Soil type E

Fig. 24. Acceleration time history of Model 2 under different soil types exposed to the Kobe earthquake for case 3.

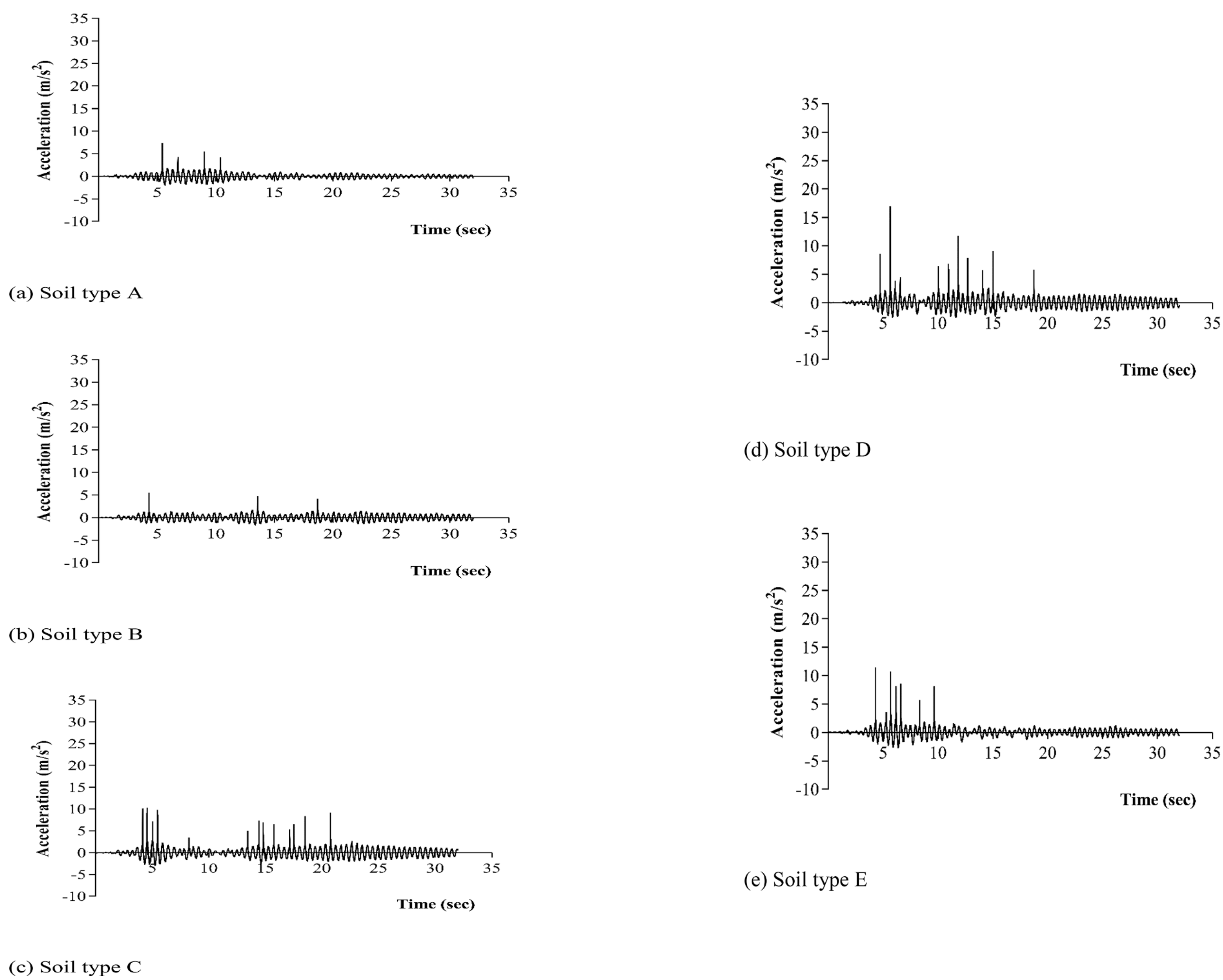
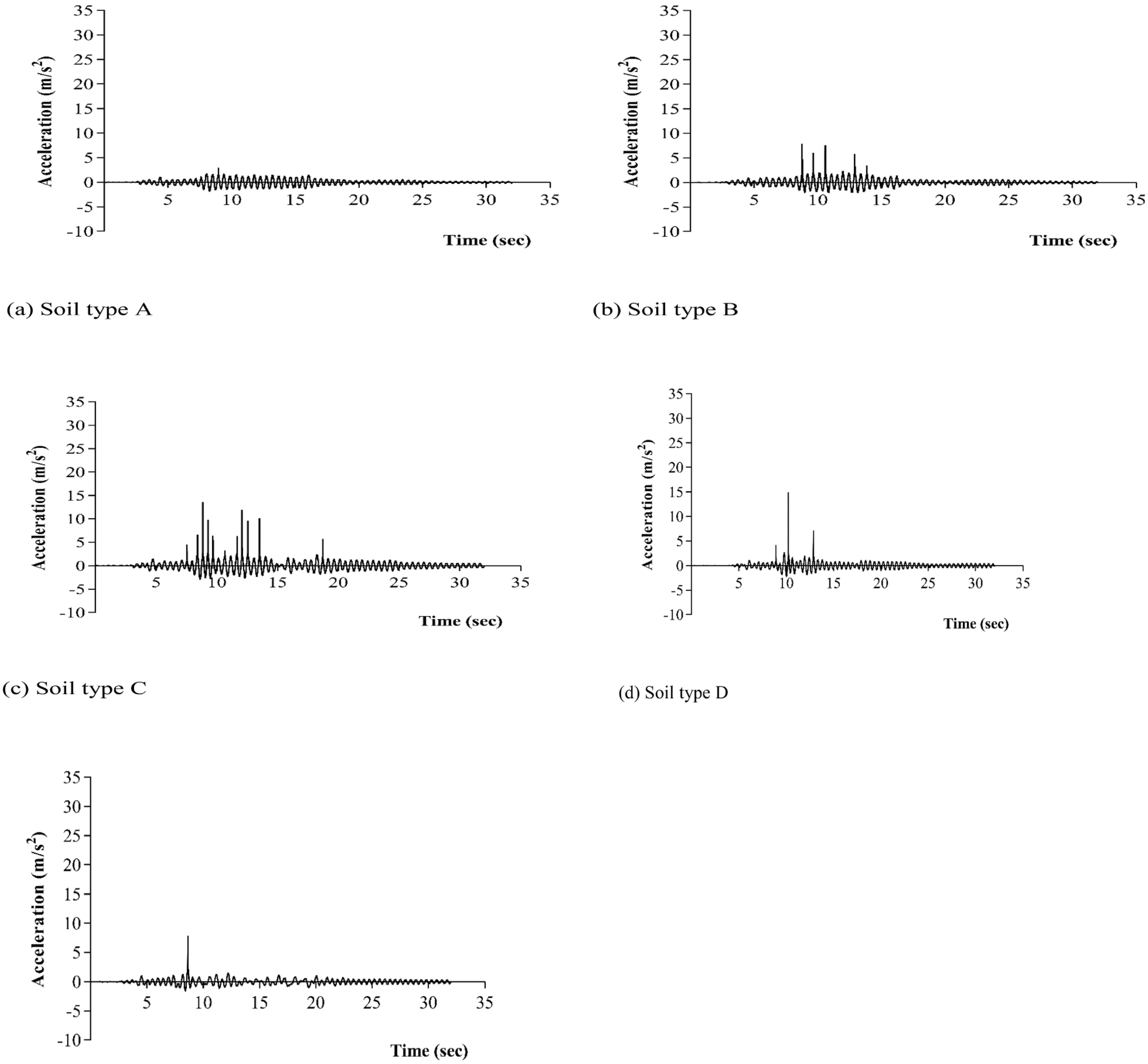


Fig. 25. Acceleration time history of Model 2 under different soil types exposed to the Parkfield earthquake for case 3.



(a) Soil type A

(b) Soil type B

(c) Soil type C

(d) Soil type D

(e) Soil type E

Fig. 26. Acceleration time history of Model 2 under different soil types exposed to the Imperial valley earthquake for case 3.

Table 3
Peak accelerations and acceleration ratios for different poundings scenarios for Model 1.

Earthquake	Soil type	No pounding	Case 1		Case 2		Case 3	
		Peak acceleration (m/s^2)	Peak acceleration (m/s^2)	Ratio	Peak acceleration (m/s^2)	Ratio	Peak acceleration (m/s^2)	Ratio
Kobe	A	2.81	16.92	6.03	15.54	5.54	12.18	4.34
	B	2.70	16.29	6.03	22.28	8.25	16.64	6.16
	C	2.77	20.26	7.31	19.26	6.95	31.79	11.47
	D	3.39	17.76	5.24	21.08	6.22	24.54	7.24
	E	2.72	17.61	6.49	18.40	6.78	27.01	9.95
Parkfield	A	1.99	11.68	5.87	17.25	8.67	13.11	6.59
	B	2.20	12.16	5.53	11.17	5.07	10.32	4.69
	C	3.49	19.67	5.64	21.63	6.20	16.66	4.78
	D	2.93	22.01	7.51	23.63	8.06	26.90	9.17
	E	2.34	26.21	11.19	20.73	8.85	19.81	8.45
Imperial Valley	A	1.79	11.15	6.24	13.26	7.41	3.29	1.84
	B	1.86	9.14	4.91	15.10	8.11	12.71	6.82
	C	3.38	25.10	7.42	18.88	5.58	21.65	6.40
	D	2.64	18.86	7.14	18.68	7.07	24.61	9.32
	E	2.81	15.89	5.66	9.73	3.47	11.12	3.96

Table 4
Peak accelerations and acceleration ratios for different poundings scenarios for Model 2.

Earthquake	Soil type	No pounding	Case 1		Case 2		Case 3	
		Peak acceleration (m/s^2)	Peak acceleration (m/s^2)	Ratio	Peak acceleration (m/s^2)	Ratio	Peak acceleration (m/s^2)	Ratio
Kobe	A	2.56	10.66	4.16	9.25	3.61	7.38	2.88
	B	2.36	10.19	4.31	13.72	5.81	8.83	3.74
	C	3.64	12.85	3.53	11.66	3.20	18.87	5.18
	D	3.33	10.98	3.29	12.97	3.89	15.91	4.77
	E	3.90	10.55	2.71	10.09	2.59	16.17	4.15
Parkfield	A	1.82	7.37	4.06	9.64	5.31	7.37	4.06
	B	1.81	7.65	4.22	6.65	3.67	5.55	3.06
	C	3.78	13.51	3.58	13.70	3.63	10.29	2.73
	D	4.21	13.82	3.28	13.77	3.27	16.90	4.01
	E	3.22	16.05	4.99	11.89	3.69	11.42	3.55
Imperial Valley	A	1.81	7.37	4.07	8.65	4.78	2.85	1.57
	B	2.77	6.76	2.44	8.50	3.06	7.89	2.85
	C	5.05	14.88	2.95	12.26	2.43	13.59	2.69
	D	2.51	11.54	4.60	11.08	4.41	14.84	5.91
	E	2.20	10.69	4.86	8.42	3.83	7.77	3.53

Table 5
Peak accelerations (m/s^2) of Model 1 founded on different soil types.

Earthquake	Pounding scenario	Soil Type A	B	C	D	E
Kobe	No pounding	2.81	2.70	2.77	3.39	2.72
	Case 1	16.92	16.29	20.26	17.76	17.61
	Case 2	15.54	22.28	19.26	21.08	18.40
	Case 3	12.18	16.64	31.79	24.54	27.01
Parkfield	No pounding	1.99	2.20	3.49	2.93	2.34
	Case 1	11.68	12.16	19.67	22.01	26.21
	Case 2	17.25	11.17	21.63	23.63	20.73
	Case 3	13.11	10.32	16.66	26.90	19.81
Imperial Valley	No pounding	1.79	1.86	3.38	2.64	2.81
	Case 1	11.15	9.14	25.10	18.86	15.89
	Case 2	13.26	15.10	18.88	18.68	9.73
	Case 3	3.29	12.71	21.65	24.61	11.12

Table 6
Peak accelerations (m/s^2) of Model 2 founded on different soil types for different poundings scenarios.

Earthquake	Pounding scenario	Soil Type A	B	C	D	E
Kobe	No pounding	2.56	2.36	3.64	3.33	3.90
	Case 1	10.66	10.19	12.85	10.98	10.55
	Case 2	9.25	13.72	11.66	12.97	10.09
	Case 3	7.38	8.83	18.87	15.91	16.17
Parkfield	No pounding	1.82	1.81	3.78	4.21	3.22
	Case 1	7.37	7.65	13.51	13.82	16.05
	Case 2	9.64	6.65	13.70	13.77	11.89
	Case 3	7.37	5.55	10.29	16.90	11.42
Imperial Valley	No pounding	1.81	2.77	5.05	2.51	2.20
	Case 1	7.37	6.76	14.88	11.54	10.69
	Case 2	8.65	8.50	12.26	11.08	8.42
	Case 3	2.85	7.89	13.59	14.84	7.77

- The maximum and minimum peak acceleration differ for various soil types, pounding scenarios, seismic gaps and earthquakes.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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