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SEISMIC BEHAVIOR OF SHALLOW-FOUNDATION LONG-PERIOD STRUCTURES IN LIQUIFIABLE SOILS

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ABSTRACT

Structures in liquefiable soils are often constructed on deep foundations so that the liquefiable layer(s) could not affect the stability of the structure. This is not the case, however, if low-rise structures, such as industrial facilities or car parks, are constructed on such soils. Due to the low concentration of vertical loads, designers avoid constructing deep foundations, the cost of which could be larger than that of the superstructure itself. In case these structures are prefabricated, thus possessing a long fundamental structural period, then the effects of the liquefaction are much more evident according to the observations of the authors during recent strong shakings. It is known that the liquefaction phenomenon increases the content of the long-period waves in the recorded motion, sometimes creating bulges in the acceleration and displacement spectra around a specific period range. This period range may vary between 1 and 2 seconds, which falls in the elastic and inelastic period range of 1- or 2-story prefabricated structures. The aim of this study is to show that the long-period low-rise structures may exhibit much more damage in case they are constructed on liquefiable soils. Three real case studies, RC prefabricated structures, all from the same industrial facility built on liquefiable soil, have been used to prove this conclusion by employing real records with and without liquefaction and a series of nonlinear time-history analyses.

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Seismic Behavior of Shallow-Foundation Long-Period Structures in Liquefiable Soils

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ABSTRACT

Structures in liquefiable soils are often constructed on deep foundations so that the liquefiable layer(s) could not affect the stability of the structure. This is not the case, however, if low-rise structures, such as industrial facilities or car parks, are constructed on such soils. Due to the low concentration of vertical loads, designers avoid constructing deep foundations, the cost of which could be larger than that of the superstructure itself. In case these structures are prefabricated, thus possessing a long fundamental structural period, then the effects of the liquefaction are much more evident according to the observations of the authors during the recent strong shakings. It is known that the liquefaction phenomenon increases the content of the long-period waves in the recorded motion, sometimes creating bulges in the acceleration and displacement spectra around a specific period range. This period range may vary between 1 and 2 seconds, which falls in the elastic and inelastic period range of 1- or 2-story prefabricated structures. The aim of this study is to show that the long-period low-rise structures may exhibit much more damage in case they are constructed on liquefiable soils. Three real case studies, RC prefabricated structures, all from the same industrial facility built on liquefiable soil, have been used to prove this conclusion, by employing real records with and without liquefaction and a series of nonlinear time-history analyses.

Introduction

For reasons of economy and speed of construction, precast concrete structures are predominantly preferred for industrial facilities, warehouses, car parks etc. where large open areas are required. Such structures, typically of low height and repeated geometry, consist of consecutive frames composed of individual columns and long-span rectangular or tapered beams, both ends of which are on pinned supports. The pinned supports, constituted by one or two anchorage dowels, permit rotation but prevent lateral movement. The frames are spanned with reinforced concrete planks bolted on the beam flanges with semi-rigid connections. The non-moment-resisting beam-column connections are the reason for the longer fundamental period, as compared to a structure with monolithic connections, and the limited redundancy of the structure.

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Several researchers [1 to 7] have reported that soil softening induced by liquefaction leads to amplification of spectral accelerations at long periods (T>1.0 s), with values larger than those at soft soils without liquefaction, and in parallel, spectral accelerations at short periods tend to decrease. The changes due to liquefaction and soil softening are reflected into other features of the response spectra: peak ground acceleration reduces [6] and peak values shift towards longer periods [7]. Youd and Carter [4] have reached similar conclusions after checking the then-available liquefaction affected accelerations and corresponding spectra.

The industrial structures are particularly vulnerable to liquefaction because of mostly being located in flat fields. Recent examples of failures of industrial structures (not necessarily due to liquefaction) occurred on flat regions with high ground water table levels, such as Adapazari (Turkey), L'Aquila and Emilia-Romagna (Italy), Christchurch (N.Zealand). Moreover, such structures have rather long fundamental periods in the range of 0.5 to 1.5 seconds that coincide with the amplification range of liquefaction. Since industrial structures are short in height but large in area, the options of deep foundation or soil improvement against liquefaction would be extremely costly in terms of price per unit area of construction.

Earthquake-induced liquefaction generates two major concerns [1] that potentially can contribute to structural failure: i) liquefaction may cause excessive ground deformations or ground failure and ii) modification of seismic waves due to ground softening may adversely affect ground response. In an earlier work, Youd and Carter [4] examined bridge structures, the long period of which coincided with the range of periods affected by liquefaction, and came up with a set of suggestions for their design in liquefiable soils. Similarly to bridges, prefabricated industrial structures sit on shallow foundations often designed without consideration for the liquefaction effects.

Therefore, after the extensive damage due to liquefaction in the aftermath of Canterbury Earthquakes and in line with the recommendations of Canterbury Earthquakes Royal Commission, the Department of Building and Housing has modified the Building Code requiring specially designed foundations on ground that is prone to liquefaction or lateral spreading, encouraging site soil investigation and in-situ testing of soil properties. In particular, where liquefaction or significant softening may occur at a site even for the SLS earthquake, buildings should be founded on well-engineered, deep piles or on shallow foundations after well-engineered ground improvement is carried out. In support of the previous clause, the need of developing guidelines to address the design and use of shallow foundations is underlined in recognition of the laxness of the current design practice of foundations with respect to liquefaction [8].

Case Studies Examined

The examined real case studies are located in Istanbul, an area seismically affected by the North Anatolian Fault. The site considered lies on riverbed-formed alluvium deposits. Intermixed and alternating silty sand, silty clay, clay layers and gravel are overlain the greywacke rock, which is present at 23 m depth, while the ground water table is generally at 2.5 m depth from the ground surface. The underlying geology, as well as the soil properties, was obtained after site investigation that included boreholes, SP and CP tests and laboratory analysis of soil samples.

The results confirmed the high liquefaction susceptibility of the site strata [9].

Three case studies have been examined in this work, all from the same industrial facility. The structural details of the facility in question are modified due to confidentiality issues. The structure consists of two separate structures: i) the main production hall and storage, and ii) the loading ramp (Fig. 1). The loading ramp consists of two types of frames, the only difference of which is the height of the first floor. The frames, running in one direction and placed in every 10 m, constitute the bearing system. There is no beam between the frames in the transverse direction as often met in construction practice in most European countries. The roof consists of double tee plates which are fixed to the beams, a property that allows a rigid diaphragm behavior. C35 concrete and S220 reinforcing steel are used for modeling, characteristic values indictaed in the design.

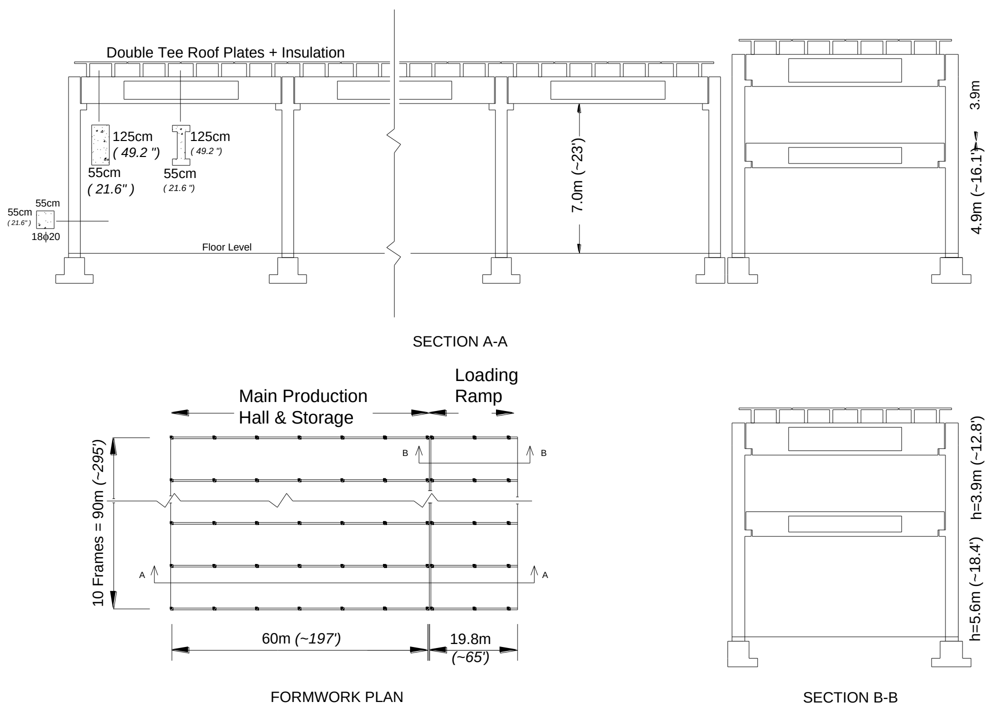

Figure 1. Formwork plan and sections of the industrial facility examined.

In summary, the facility examined comprises three different types of frames: i) the single-story frame of the main production hall (Case Study 1), ii) the 2-story frame of the loading ramp (Case Study 2a), and iii) the 2-story taller frame of the loading ramp (Case Study 2b). Once modeled with distributed plasticity elements, the three structures exhibit fundamental periods of 1.0 s, 1.22 s and 1.38 s, respectively. The periods would have been slightly shorter if concentrated plasticity models were used, however, they would be still considered long for single- and 2-story RC structures.

The structure has a relatively simple foundation system where individual columns fit into socket footings. There is no structural connection, i.e. tie beams, between the consecutive footings. Even the construction of tie-beams would be expensive given the large dimensions of the structure (a tie-beam would be approximately 10 m long) resulting in oversized elements.

The Numerical Model and the Ground Motion Records Used

The frames have been modeled in OpenSees software [10]. A representative middle frame has been extracted from each structure applying the load and mass from its tributary area. Concrete02 and Steel02 cyclic uniaxial material models have been used for modelling reinforced concrete. The connection of the columns to the beams has been modelled by using perfect flexural hinges. The columns are assumed fixed to the base. This is a rather improper modelling choice; however the issue of the soil-structure interaction was consciously left outside the scope of this research work for the sake of simplicity. Because of the connection of the roof double tee plates to the beams, the rigid diaphragm assumption is valid.

Table 1a. The acceleration records used in the analyses.

Event	Station	Site Class	PGA (g)	Liquefaction
Kobe	JMA	Soft Soil	0.85	No
	Kakogawa	Soft Soil	0.31	No
	Nishi-Akashi	Soft Soil	0.51	Yes
	Port Island	Soft Soil	0.37	Yes
	Takatori*	Soft Soil	0.30	Yes
	Amagasaki	Soft Soil	0.68	No
Kocaeli	Sakarya**	Hard Soil	0.33	No
Landers	Joshua	Soft Soil	0.30	No
Niigata	Kowagishi	Soft Soil	0.18	Yes
Supertition	Wildlife	Soft Soil	0.12	Yes
San Fernando	PUL	Hard Soil	1.49	No
Chi-chi	TCU068	Soft Soil	0.58	No
	CHY028	Soft Soil	0.69	No
Loma Prieta	LPG	Soft Soil	0.63	No
Northridge	WPI	Soft Soil	0.28	No

* It is not clear neither in the literature nor to the knowledge of the authors if liquefaction really occurred

**One horizontal component was available due to malfunction of the accelerometer, thus the dominant direction is not used

The ground motion records are chosen from different events on both soft and hard soil sites, as shown in Table 1. Soft soil records are selected so that high spectral accelerations are obtained in long period range too, something that allows the comparison with structural responses due to acceleration records with rather similar spectral demands. The distinction between the hard and soft soil cases was made based on the reported soil properties of the recorders in the literature [2, 3, 11 to 15]. Fifty-three records are used in total, seventeen of which exhibited liquefaction. The sign of liquefaction is the dominant feature of such records:

long-period cycles with reduced acceleration amplitudes occur after a threshold value of acceleration has been reached.

Table 1b. The acceleration records used in the analyses.

Event	Station	Site Class	PGA (g)	Liquefaction
ChCh	CBGS	Soft Soil	0.64	Yes
	CCCC	Soft Soil	0.49	Yes
	CHHC	Soft Soil	0.46	Yes
	CMHS	Soft Soil	0.42	Yes
	HPSC	Soft Soil	0.25	Yes
	HVSC	Hard Soil	1.50	No
	LPCC	Hard Soil	1.00	No
	PPHS	Soft Soil	0.25	No
	PRPC	Soft Soil	0.65	Yes
	REHS	Soft Soil	0.73	Yes
	RHSC	Soft Soil	0.30	No
	SHLC	Soft Soil	0.34	Yes
	SMTC	Soft Soil	0.19	No
Darf	CBGS	Soft Soil	0.18	No
	CCCC	Soft Soil	0.24	No
	CHHC	Soft Soil	0.20	No
	CMHS	Soft Soil	0.26	No
	HPSC	Soft Soil	0.16	Yes
	HVSC	Hard Soil	0.66	No
	LPCC	Hard Soil	0.37	No
	PPHS	Soft Soil	0.22	No
	PRPC	Soft Soil	0.19	Yes
	REHS	Soft Soil	0.32	No
	RHSC	Soft Soil	0.23	No
	SHLC	Soft Soil	0.19	No
	SMTC	Soft Soil	0.19	Yes
June	CBGS	Soft Soil	0.18	No
	CHHC	Soft Soil	0.22	No
	CMHS	Soft Soil	0.22	No
	HPSC	Soft Soil	0.43	Yes
	HVSC	Hard Soil	1.00	No
	LPCC	Hard Soil	0.63	No
	PPHS	Soft Soil	0.15	No
	PRPC	Soft Soil	0.30	No
	REHS	Soft Soil	0.36	No
	RHSC	Soft Soil	0.21	No
	SHLC	Soft Soil	0.22	No
	SMTC	Soft Soil	0.10	No

Soil softening due to excess pore water pressures in combination with sufficient acceleration values leads to amplification of large periods affecting a broad category of structures, as indicated by the acceleration spectra. In particular, the spectral amplification at periods exceeding 2 seconds is attributed to the fact that once liquefaction has occurred, the overlying soil "crust" oscillates with very low frequencies, causing the bulges observed in the acceleration spectra for periods of about 3 seconds [2]. The PGA value of the records used varies between 0.10 and 1.5 g. Note that the PGA values of Tables 1a and b refer to the dominant direction of the record, i.e. the direction in which PGA is the highest [2].

Analyses Results

Several nonlinear time-history analyses were conducted. In order to obtain a better insight into the individual structural response, a simple comparison is presented in Fig. 2, where Case Study Structure 1 (T = 1.0 s) is subjected to two records from the Kobe 1995 earthquake, the JMA and the Port Island record, where the former had no liquefaction trace but the latter is identified with liquefaction. It should be noted that the spectral acceleration demand for T = 1.0 s for the JMA record is higher (1.762 g) than the equivalent one for the Port Island record (1.182 g). Nevertheless, considerably higher displacement is obtained for the Port Island record with liquefaction, while the corresponding hysteretic loop area, indicative of the demand introduced in the structure, is readily larger (Fig. 2). This very observation outlines the idea behind this work: records with liquefaction cause more damage on the structures even if the elastic spectral demand of the records with liquefaction are equal to or less than that of the soft soil records without liquefaction.

Zooming in on the plots of Fig. 2, fourteen distinct cycles between the 7^{th} and 30^{th} second in the displacement time-history can be distinguished for both records. Despite the equal number of cycles, the average displacement of these peaks is 6.1 cm (2.4 in) for the JMA record (no liquefaction), whilst the same quantity is as much as double - 12.0 cm (4.8 in) - for the Port Island record that had severe traces of liquefaction. Though hard to generalize given the number of records and case studies examined, as well as the modeling approximations done, this finding indicates that the displacement demand itself is the critical parameter in determining the response of the structure, at least for the cases mentioned here.

The results of all the nonlinear time-history analyses conducted are presented in such a format that the demand of the shaking is expressed by means of a structure-dependent parameter. Thus, the spectral acceleration value at the fundamental period of the structure, $Sa(T_1)$, is utilized. In order to represent the damage on the structure, the widely used damage index of Park and Ang [16] has been employed (Eq. 1).

$$D = \frac{\delta_M}{\delta_u} + \frac{\beta}{Q_y \delta_u} \int dE \quad (1)$$

where d_M is the deformation under earthquake, δ_u is the ultimate deformation under monotonic loading found by 1^{st} mode pushover analysis in this study, Q_y is the calculated yield strength (if the maximum strength, Q_u, is smaller than Q_y, Q_y is replaced by Q_u), dE is the incremental absorbed hysteretic energy, and β is suggested as 0.15 [16].

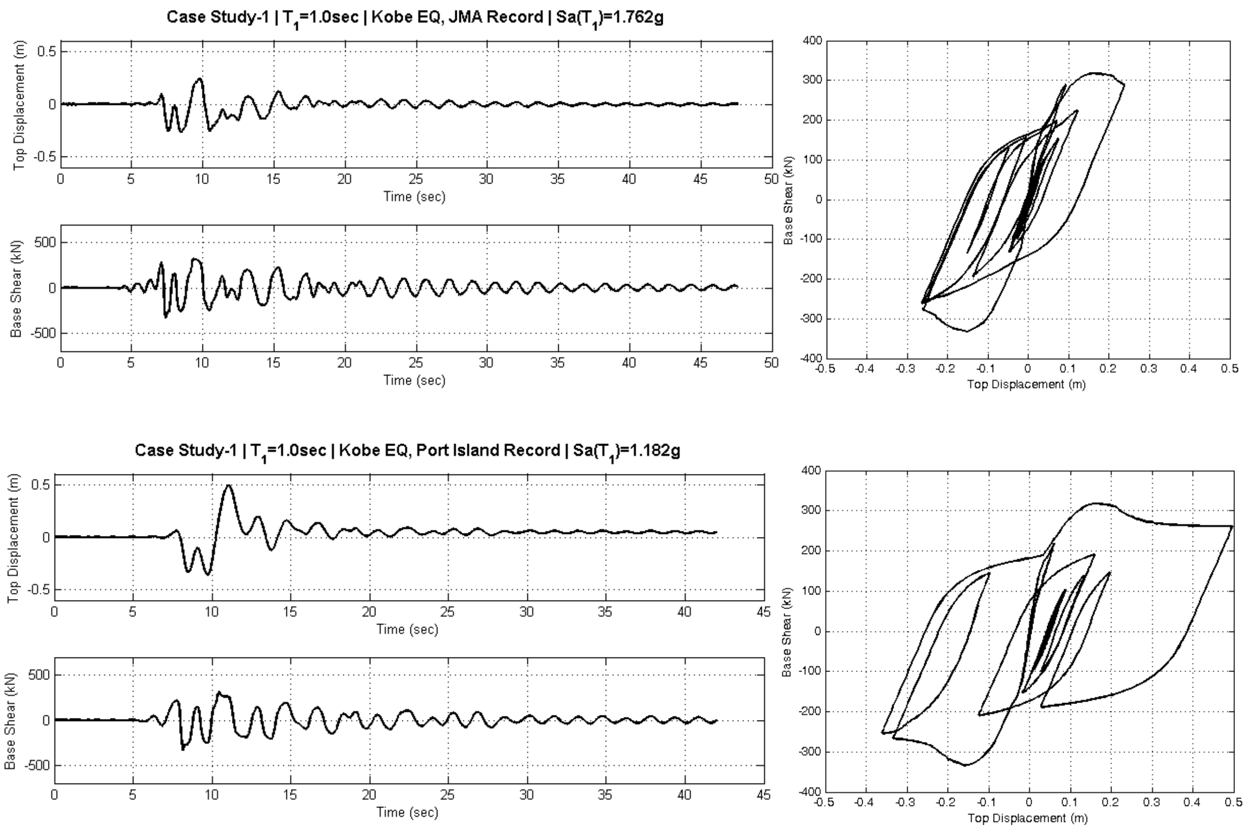

Figure 2. Base shear and top displacement response of case study 1 (top: JMA record w/o liquefaction, bottom: Port Island record w/ liquefaction)

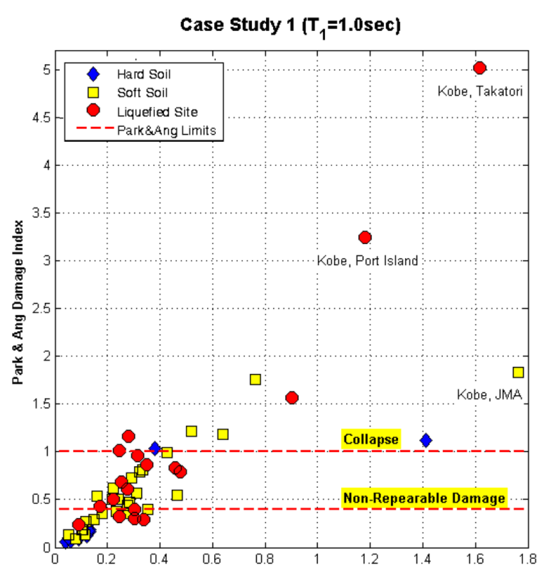

Figure 3. Comparison of the spectral acceleration at the fundamental period with the Park & Ang damage index for Case Study 1 ($T_1 = 1.0$ s).

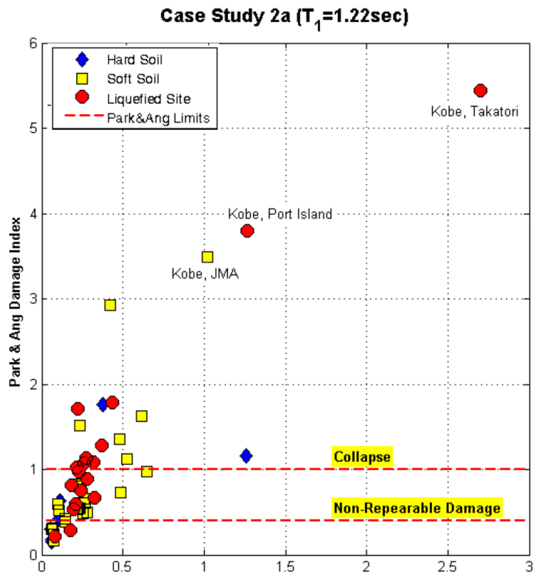

Figure 4. Comparison of the spectral acceleration at the fundamental period with the Park & Ang damage index for Case Study 2a (T_1=1.2 s).

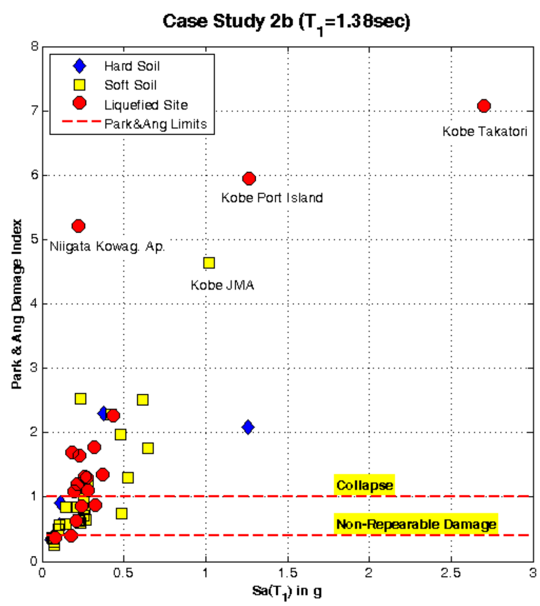

Figure 5. Comparison of the spectral acceleration at the fundamental period with the Park & Ang damage index for Case Study 2b (T_1=1.38 s).

In Figs. 3 to 5 the comparison between the Park & Ang damage index and the Sa(T1) values is illustrated. For the Main Production Hall of the facility (Case Study 1 – 1.0 s fundamental period), eleven out of the fifty-three records proved to be destructive leading to its collapse. Five of these critical records (four if Takatori record is excluded) have traces of liquefaction. Interestingly the results of Figs. 3 to 5 indicate that records with higher Sa(T_1) produce less damage than the records with liquefaction but with lower Sa(T_1). It is noted that the

Park & Ang damage index considers both the maximum nonlinear displacement and the hysteretic energy, thus the effect of number and amplitude of cycles is inherently incorporated in the results presented in Figs. 3 to 5.

Another interesting observation derived from Fig. 5 is that the Niigata record has very small spectral acceleration at the fundamental period but the relative damage index is particularly large. However, before drawing any conclusions from that record regarding long period structures, it is reminded that the Niigata record was analog and was digitized, thus the long period range of this record is not as reliable as that of the digital records. In overall, though, it can be claimed that the records with liquefaction have stronger damage potential to lead to collapse, as indicated by the case studies examined here. Even for damage levels below collapse, records with liquefaction were shown to be more damaging.

Conclusions

In the case of precast concrete structures, due to the low concentration of vertical loads, designers avoid constructing deep foundations, the cost of which could be larger than that of the superstructure itself. This fact, in conjunction to their long period characteristics, renders prefabricated structures completely exposed to liquefaction effects. It is known that liquefaction increases the content of the long-period waves in the recorded motion, sometimes creating bulges in the acceleration and displacement spectra around a long-period range. Three real case studies, all from the same industrial facility built on liquefiable soil, have been used to prove that long-period low-rise structures are particularly vulnerable to liquefaction. Real records with and without liquefaction were employed and a series of nonlinear time-history analyses were conducted during which the response of the case studies was monitored.

The results indicate that, at least for the cases examined, records with liquefaction proved to have higher damage potential, leading to either collapse or to severe damage. This finding is not explicable by solely considering the number of cycles but it is mostly attributed to the amplitude of the nonlinear displacements. Analyses also showed that records with liquefaction predominantly remain the most damaging ones although their spectral accelerations can be considerably lower that the equivalent ones of records from soft-soil sites.

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