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The effects of low-rise building mat on liquefiable site

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ABSTRACT
Introduction. The effects of low-rise building mat on liquefiable site were investigated in this study.
Materials and methods. One-dimensional ground response analyses of a layered sand model profile were conducted using Midas. The UBCS and soil model was used.
Results. The excess pore water pressure and the stress/strain time histories as well as the ground deformation of the numerical model were examined.
Conclusions. With the comparison of the free-field solutions, if the liquefaction occurred, the mat can cause the subsoils to settle more and push the side soils to move laterally, which will yield sway motions of the mat. The deeper embedment of the mat sometimes would help to minimize such phenomenon.

KEYWORDS: low rise building, mat foundation, free field, layered sand, soil liquefaction

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Влияние плит оснований малоэтажных зданий на разжижаемость грунтов

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АННОТАЦИЯ
Введение. Изучается влияние плит оснований малоэтажных зданий на разжижаемость грунтов.
Материалы и методы. С помощью ПО Midas проведен одномерный анализ реакции слоистой песчаной модели. Использовались модель UBCS и модель грунта.
Результаты. Исследованы избыточное давление поровой воды, временные зависимости напряжений и деформаций, а также деформации грунта в условиях численной модели.
Выводы. При сравнении решений в свободном пространстве в случае разжижения наличие плиты основания может привести к большему оседанию грунтов основания и боковому смещению, что приведет к раскачиванию основания. Более глубокое заглубление основания иногда помогает минимизировать это явление.

КЛЮЧЕВЫЕ СЛОВА: малоэтажное здание, плитный фундамент, свободное пространство, слоистый песок, разжижаемость грунта

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INTRODUCTION

This study is a pilot study for investigations of the influences of foundation type in liquefiable sites. It aims to understand the impact of liquefied ground on the mat foundation of low-rise building. The study uses two-dimensional finite element analysis to simulate the response of free-site ground and the same site with the existence of mat under the shakings of one-dimensional horizontal earthquakes. The changes of the shear stress and strain, the excess pore water pressure and the permanent displacement under the earthquakes affected by the seismic intensities, the design geometry of the mat as well as the dead load applied onto the mat will be discussed [1–5].

MATERIALS AND METHODS

Numerical model and seismic concerns

In this study, the finite element software MIDAS-GTS NX (Midas, 2014) was used to simulate the seismic behavior of the soil-raft foundation, whereas the UBCS and model (Byrne et al., 2004) was used to simulate the possible liquefaction of the sands. For comparison, the free-field solution was obtained first, and the analysis of the mat at the same site was monitored. The study was following the site example revealed at the National Science Council's joint research project, and then taking the soil parameters as the basis. The numerical model was planned as a layered sand profile, which has a depth of 50 m. Fig. 1 and Table 1 illustrate the numerical model. In order to include the influence of seismic forces, the seismic record during 1999 Chi-Chi earthquake in zone-1 of Taipei basin was adopted. The record was selected and examined to ensure that its main frequency is similar to the natural frequency of the site (Fig. 2). The original earthquake records are calibrated through the scaling method for the target motion, and the baseline correction was used to eliminates the source error. The peak ground acceleration (PGA) used in the study is corresponding to the levels of seismic intensity at IV, V^-, V^+ and VI^- issued by the Central Weather Administration in Taiwan, which is 0.08, 0.11, 0.195 and 0.25 g respectively. The UBCS and mode parameters are organized as shown in Table 2. The mat foundations were mainly assumed with the embedment depths of 2 and 5 m, and the width of the foundation was 20 and 30 m. The load applied at the mat was 30 and 60 kPa to simulate the low-rise building weights. The boundary conditions on both sides of the finite element model are set to Free Field, in which the infinite boundary condition is set up on the *XY* plane to eliminate the reflected waves from the boundaried. The bottom node of the model is set as the hinge support to simulate the rigid base of the underlain bedrock. From the examination of the stability of the solutions, the overall analysis width of the model was set to 300 m.

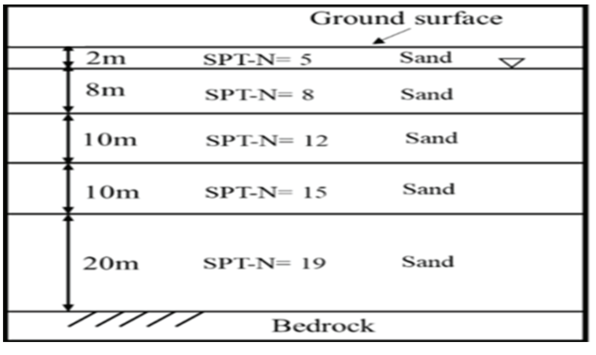

Fig. 1. Layout of the numerical model of layered sands

Рис. 1. Макет численной модели слоистых песков

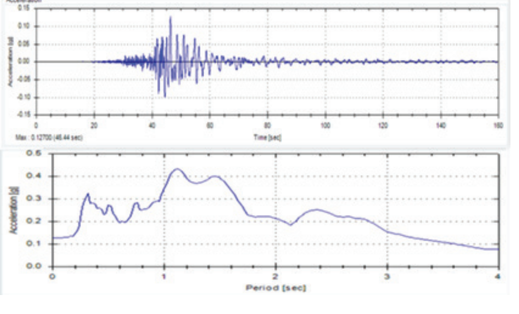

Fig. 2. Record from TAP003 station in Chi Chi earthquake

Рис. 2. Запись со станции TAP003 во время землетрясения в Чи-Чи

Table 1. Material parameters of the numerical model

Табл. 1. Параметры материала численной модели

Depth (m)	r_d (kN/m³)	r_{sat} (kN/m³)	V_s (m/s)	N	$(N_1)_{60}$	Ø (°)	G (kPa)	E (kPa)	ν
0~2	18	20	144	5	10	28	38000	99000	0.3
2~10	18	20	165	8	10	30	55000	155000	0.4
10~20	18	20	186	12	12	32	71000	199000	0.4
20~30	18	20	201	15	12	33	82000	230000	0.4
30~50	18	20	215	19	12	35	94000	264000	0.4

Table 2. Model parameters of UBCSand model

Табл. 2. Параметры модели UBCSand

Parameter	Symbol	Midas (Byrne, 1998)
Elastic (Power Law)		
Elastic shear modulus number	K_G^e	$K_G^e = 21.7 \times 20.0 \times (N_1)_{60}^{0.333}$
Elastic shear modulus exponent	ne	0.5
Plastic / Shear		
Peak Friction Angle	$\varnothing_p$	$\varnothing_p = \varnothing_{cv} + (N_1)_{60}/$ $10.0+\max\left(0.0, \frac{(N_1)_{60}-15}{5}\right)$
Constant Volume Friction Angle	$\varnothing_{cv}$	$30° < \varnothing_{cv} < 34°$
Plastic shear modulus number	K_G^p	$K_G^p = K_G^e (N_1)_{60}^2 \times 0.003 + 100.0$
Plastic shear modulus exponent	np	0.4
Failure ratio	R_f	$R_f = 1.1 \times (N_1)_{60}^{-0.15}$ 0.7~0.98 (< 1)
Post Liquefaction Calibration Factor	F_{post}	0.1
Soil Densification Calibration Factor	F_{dens}	0.45

Interface element

In the MIDAS-GTS NX program, the interface elements are defined using the Coulomb friction method. The stresses at the interface elements are mainly

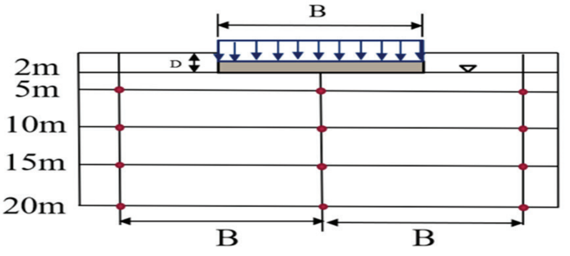

Fig. 3. Observation zone in this study

Рис. 3. Зона наблюдений в исследовании

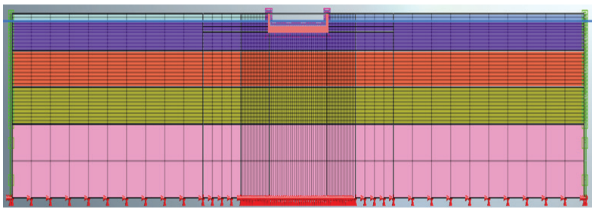

Fig. 4. Layout of the FE Mesh used in this study

Рис. 4. Схема используемой в исследовании сетки конечных элементов

the friction between the concrete and the soils. According to such type of mechanism, the interface element parameters will be dominated by the weaker material which is the soil. With the suggestions from Midas Manual, the spring coefficient k_n in the normal direction and the spring coefficient k_t in the shearing direction at the contact face were set to 10 E_s (Young's coefficient of the soil) and E_s respectively, and the ultimate shear stress τ_{max} is computed as $\sigma \tan\delta$, where δ is the friction angle between the two materials (taking as 2/3 of the friction angle φ of sand), and σ is the normal stress acting upon the contact plane.

Elements and mesh

Four-node supralateral elements were adopted in this study. The finite element mesh was determined with the concern that the aspect ratio of element length-width should be 1:1. Since the main observation in the study is the physical quantities at the centre part of the soil profile and the mat with the nearby soils around the mat (which can be taken with a spacing distance of 1B from the centre line) as shown in Fig. 3, the elements close to the centre were kept smaller, whereas the farther out elements were expanded with larger aspect ratios to reduce the computation time. Fig. 4 shows an example of foundation mesh.

RESULTS

Free-field observations

1. The shear stress of the soil layer will increase with the increase of earthquake intensity, and the deeper the depth, the greater the shear stress; the shear stress will be eliminated by the generation of EPWP.

2. The permanent shear strain of the soil layer increases with the earthquake intensity. At the intensities studied, the maximum shear strain of the soil layer can reach 2.5 %.

3. The permanent ground displacement also increases with the earthquake intensity. When PGA = = 0.195 g, the permanent horizontal and vertical ground displacements are 18 and 25 cm; when PGA = 0.25 g, the permanent horizontal and vertical ground displacements reach 27 and 47 cm. Fig. 5–7 show the changes in shear stress, shear strain, and excess pore water pressure of the model site with depth when PGA = 0.195 g. Fig. 8 shows the surface horizontal and vertical displacement time histories of the site.

4. The soil liquefaction will become more severe as the earthquake intensity increases. Comparing with

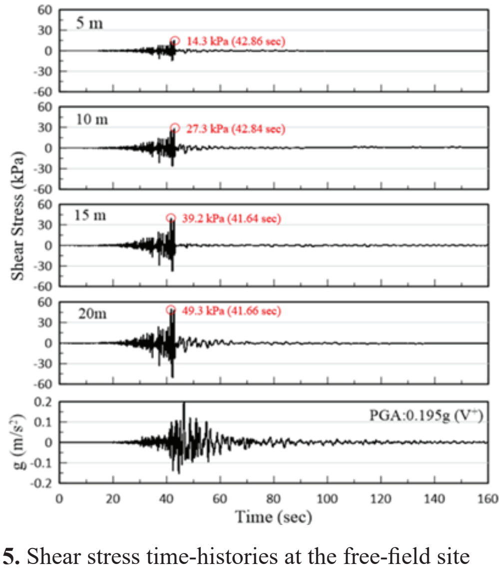

Fig. 5. Shear stress time-histories at the free-field site

Рис. 5. Временные зависимости напряжений сдвига на участке свободного пространства

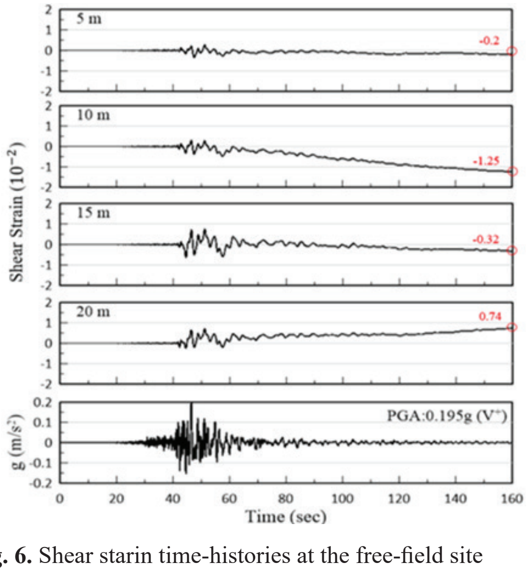

Fig. 6. Shear starin time-histories at the free-field site

Рис. 6. Временные зависимости сдвиговых напряжений на участке свободного пространства

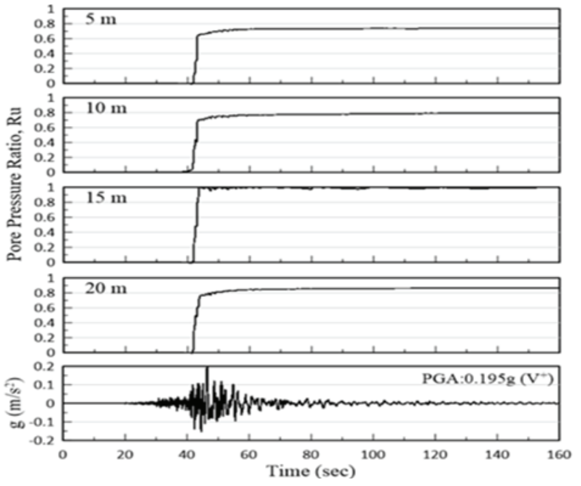

Fig. 7. Time histories of the ratios of excess pore water pressures at the free-field site

Рис. 7. Временные зависимости соотношений избыточных давлений поровой воды на участке свободного пространства

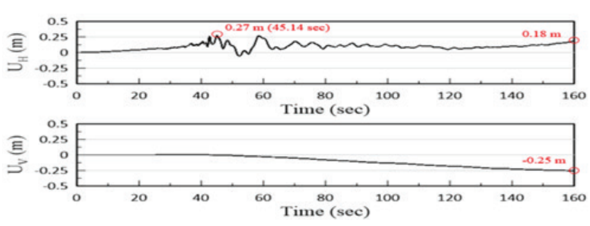

Fig. 8. Horizontal and vertical displacement time histories at the free-field ground surface

Рис. 8. Временные зависимости горизонтальных и вертикальных перемещений на поверхности земли в свободном пространстве

the empirical analysis of the liquefaction potential assessment suggested in Taiwan, the results showed that if PGA < 0.195 g, the safety factor suggested by the empirical method would be higher; if PGA ≥ 0.195 g, the safety factor of the empirical method can approximate the results of 1D ground response analysis.

5. By computing the ground settlement from the empirical methods and treating the vertical strain of the soils as 1/3 of the volumetric strain or 1/2 of the volumetric strain (assuming ν = 0.5), the settlements were compared with those obtained from the FE analyses. It was found that the results from Tokimatsu and Seed (1987) are higher than FE solutions; while those from Ishihara and Yoshimine (1992) are smaller than FE solutions when PGA < 0.195 g and higher than FE when PGA ≥ 0.195 g.

Mat foundation

1. Comparing with the free-field responses, the mat will increase the shear stress and decrease the shear strain of the soils underneath it, however the side soils will be pushed out by the mat. The mat will decrease the liquefaction potential of the soils underneath it due to the influences of overburden pressures, however if the surrounding soils were tending to liquefy during the ground shaking, the mat will easily trigger unbalanced ground motions which could cause the mat to sway during the seismic impact. The mat could also accelerate the EPWP dissipations during the excitations. Fig. 9–11 depict the comparison of mat foundation and free field responses when PGA = 0.195 g.

2. When the PGA remains unchanged, reducing the size of mat will reduce the shear stress increment of the underlying soil layer (compared with the free field one), and the soil strain remains similar to the free field. The horizontal displacements of the soils below the foundation were suppressed and similar to the free field, and the vertical displacements were larger than the free field. The soils aside the smaller size mat had less displacements com-

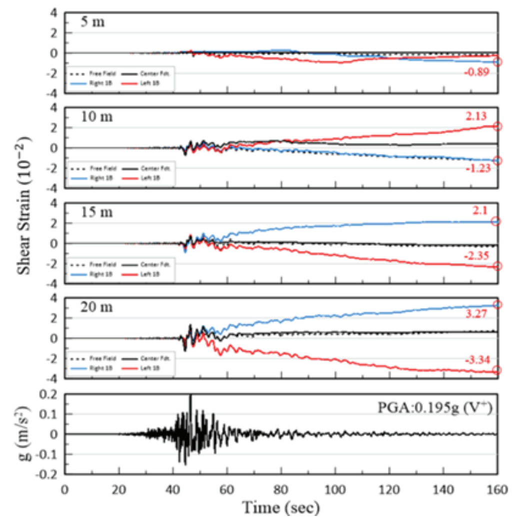

Fig. 9. Effects of the mat to shear strain time histories

Рис. 9. Влияние основания на временные зависимости деформации сдвига

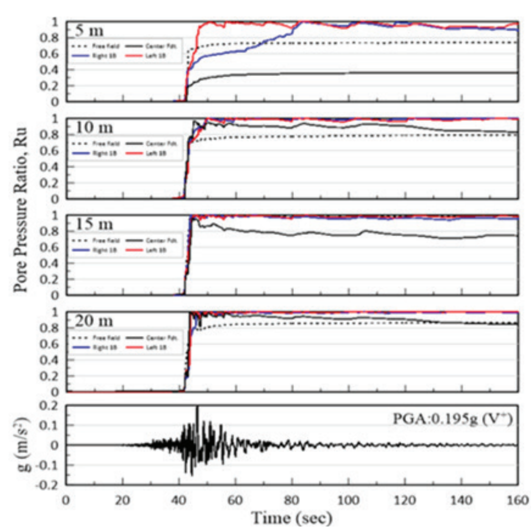

Fig. 10. Effects of the mat to Ru time histories

Рис. 10. Влияние основания на временные зависимости коэффициента порового давления

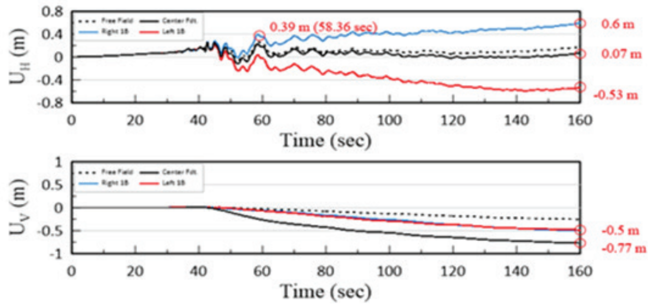

Fig. 11. Effects of the mat to the surface ground displacement time histories

Рис. 11. Влияние плиты основания на динамику смещения поверхности грунта

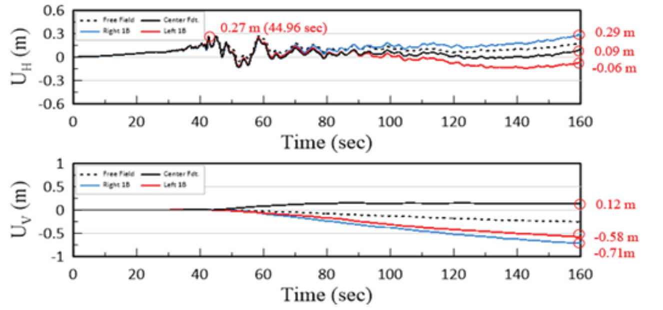

Fig. 12. Effects of embedment depth at 5 m to the surface ground displacement time histories

Рис. 12. Влияние глубины заложения на 5 м на временные зависимости смещений грунта на поверхности

pared to those found at larger size, but much larger than those found at the free field site.

3. By putting the low-rise building mat deeper will increase the maximum shear stress of the soil layer. The increase in embedment depth of the mat (2~5 m from the ground surface) will push the side soils move outwards during the earthquake, which will cause easily the foundation to tilt and collapse. Although the seismic resistance of mat increases with depth, the possible destructiveness of the low rise building mat foundation may be intensified. This deserves the engineer's attention. Fig. 12 shows the impact of increase the embedment depth.

4. The increase in foundation load will increase the maximum shear stress of the shallow soil layer; the permanent shear strain of the soil layer will decrease. Comparing the duration of the excess pore water pressure ratio, it can be found that the excess pore water pressure ratio under the foundation is suppressed and the dissipation situation becomes more obvious.

5. The effects of shear stress, shear strain, excess pore water pressure ratio, and horizontal and vertical displacements with respect to the mat width, embedment depth, and load of foundation are summarized in Table 3.

More details of the observations and discussions can be found in Lian (2024).

CONCLUSIONS

Free-field response

The study focuses on layered sand profile where low to medium strength parameters were selected to yield conservative results for the liquefaction influences on low-rise building mat foundation. The study was mainly conducted using 2D Midas analysis. Each case study can be accomplished within three hours. The re-

Table 3. The effects on mat on the measures affected by soil liquefaction

Табл. 3. Влияние показателей, затронутых разжижением грунта, на плиту основания

Influence factors / Measures	Increase *B* (20~30 m)	Increase *D* (2~5 m)	Increase load (30~60 kPa)
Max. shear stress	Increased	Increased	Increased
Max. shear strain	Increased	Increased	Increased
Max. Ru	Decreased	Increased	Decreased
U_H at fdt. centre	Decreased	Decreased	Increased
U_V at fdt. centre	Decreased	Decreased	Increased
U_H outsides fdt.	Increased	Increased	Increased
U_V outsides fdt.	Decreased	Increased	Increased

sults were compared with those from empirical liquefaction potential analysis and the seismic settlement prediction methods. The study found that seismic intensities PGA at 0.08 and 0.11 g will not induce the soil liquefaction, while intensity PGA larger than 0.195 g will cause the soil liquefaction to occur especially for the sand layers at shallower depth from the ground surface.

Mat foundation response

The analysis of the mat at the same profile reveals that the mechanism became quite complex when the foundation exists. The liquefaction potentials of the soils right below the mat can be reduced, but the corresponding influences in the side soils around the mat may cause the foundation to tilt and sway. The embedment depth indeed provides the advantage to resist the liquefaction induced damages. However, for low rise building foundation, the embedment depth of the mat is maximum up to 5 m. Therefore, the mat foundation for low-rise building has its limited strength to overcome the soil liquefaction induced damages.

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Ting-Lun Hsu — helped to prepare the editorial work of this paper.
Diyar Mukhanov — help to prepare and translate work of this paper.
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