

Seismic response of monopile-supported wind turbine systems: combination laws for kinematic and inertial interaction

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ABSTRACT

The dynamic soil structure interaction imposes both kinematic and inertial interactions on monopile-supported wind turbines. The two effects can be calculated individually, but there is no clear guidance in the current design standard for combining them. A theoretical study using the finite element method to model a simplified single degree of freedom system has been conducted to analyse the phase relationship between the two effects and obtain the combination laws through parametric studies using the enhanced hysteresis control soil constitutive model. The results show that when the fundamental frequency of the soil is smaller than the superstructure, an in-phase relationship is observed. Monopile's bending moments from the two loadings have different signums. Considering the inertial loading alone with an additional 5% modification is sufficient to capture the maximum bending moment and the profile. When the natural frequency of the soil is larger than the superstructure, an out-of-phase relationship is observed. The kinematic loading has a significant domination on the lower part of the pile. The worst-case scenario can be obtained when both loadings are combined using the simple addition rule. These findings enable engineers to combine two loadings in a simple manner, avoiding performing the lengthy dynamic time-history numerical analysis and facilitating an efficient design.

KEYWORDS Kinematic interaction; inertial interaction; hysteresis generation; soil constitutive model; combination laws; pile seismic design; offshore wind turbine

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1. Introduction

With the rapid development of offshore wind energy, it has become one of the most popular renewables for coastal countries. However, the delivery of offshore wind energy is challenging for countries located in seismic zones. With a limited understanding of the seismic response, a well-established seismic design code for offshore wind turbine foundations has not yet been developed (Kaynia, 2019). Furthermore, the existing non-codified seismic design approach is rather conservative, leading to uneconomical designs. The lack of knowledge and design guidance has led to the slow development of offshore wind energy in earthquake-prone countries.

The dynamic soil-structure interaction is usually interpreted in two parts: kinematic and inertial interactions. Figure 1 demonstrates the graphical representation of the two interactions. Kinematic interaction (KI) is the modification of the free-field soil displacement due to the stiffness contrast between the surrounding soil and the pile without considering the mass of the superstructure. Gazeta (1984) proposed the kinematic interaction factors from finite element analyses to modify the soil column displacement at far-field and imposed it as a kinematic loading. Inertial interaction (II) involves the dynamic interaction among the structure, foundation and soil. The turbine tower is treated as an equivalent single-degree-of-freedom (SDOF) structure with the rotor nacelle assembly (RNA) mass on the top and a weightless pile. The stiffness and damping from the

foundation and soil affect the dynamic behaviour of the stick-up tower, reflecting the D'Alembert force, F , acting on the superstructure. Wolf (1985) derived expressions for the natural frequency and damping ratio of the equivalent non-fixed base SDOF using macro-elements to calculate the equivalent dynamic effect for the II. Research investigations into the individual KI and II have been reported while only a limited paper has studied the coupling effect of them.

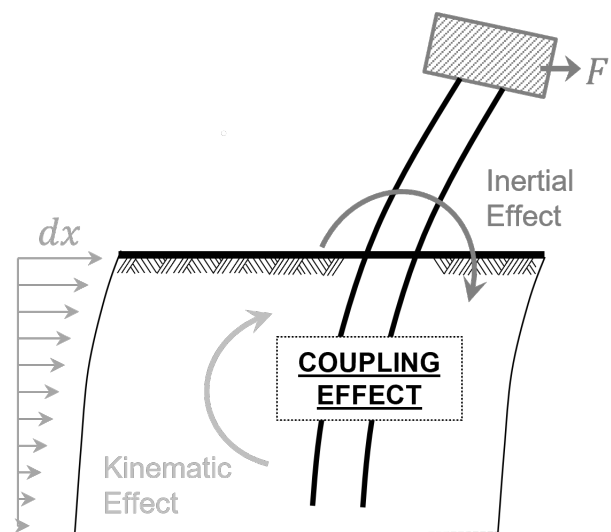


Figure 1. Coupling effect of kinematic and inertial interactions on a monopile-supported wind turbine.

Various opinions have been reported in the literature. Gazetas (1984) and Pender (1993) concluded that the contribution of KI was negligible for a soil profile with a smooth variation in shear modulus (G). Considering II alone is sufficient to capture the critical bending moment of the pile. However, this may be valid only for slender piles (aspect ratio = 40) as investigated by Gazetas (1984). The monopile has a large diameter with a low aspect ratio. The bending stiffness of a monopile is likely to be larger than that of a slender pile, thus resulting in a different conclusion for the importance of KI. Nevertheless, the numerical study conducted by Kaynia (2020) demonstrated that the acceleration of the tower and piles' bending moment are larger when the KI is taken into account.

The phase relationship of KI and II may shed light on the combination rules to investigate the importance of the two loadings. However, contradictory conclusions were found. From the large shaking table tests conducted by Tokimatsu et al. (2005), the results show that both KI and II were in phase when the natural frequency of the superstructure (f_h) was larger than the soil deposits (f_g). The maximum pile bending moment occurs when both KI and II are at the peak values and acting in the same direction at the pile head. KI and II were out of phase on the contrary natural frequency relationship. The same conclusion had been proven by Di Laora (2009) after considering the harmonic excitations in the theoretical analysis and real earthquake motions for different soil stratigraphy through a numerical study. However, Wang et al. (2017) found that KI and II always act in phase for a single pile without a pile cap from a finite-element analysis (FEA) after performing a parametric study on different masses of the superstructure and shear modulus of the deposits. In addition, Garala and Madabhushi (2021) found that KI and II are not always either in phase or out of phase when the natural frequency of the input excitation, f_i , is less than or close to f_g after performing a centrifuge test with various base excitations. In other words, f_b and f_g do not necessarily govern the phase relationship.

The existing seismic design standard for offshore wind turbines remains to be improved. Some standards recognise the importance of dynamic soil structure interaction (DSSI) but do not provide specific guidelines for KI and II quantification in monopile structures as well as their combination laws (DNV, 2021a), or some do not even mention DSSI (IEC, 2005; MOHURD, 2019; Windpal, 2018). Some seismic design standards for offshore wind turbine foundations are even based on the onshore seismic building foundation design (Kaynia, 2019) which is more conservative in terms of the response spectrum.

Although a full three-dimensional (3D) time-history dynamic FEA for a monopile can capture well the seismic response using advanced constitutive models, the significant computational cost has discouraged its use for repeated monopile assessment in common engineering practice. To perform a preliminary design with less complexity, the decoupled analysis can be used (Pappin, et

al., 1998). The KI is firstly found from the soil response in the Oasys program SIREN. In a dynamic analysis, SIREN would be able to find the time at which deflections of soil column have the maximum curvature. The corresponding deflections were imposed as a static force on a non-linear soil-pile system in the Oasys program ALP to achieve the combined KI and II force. In other words, both KI and II are considered to make full contributions for the combination. A less conservative approach by considering the critical pile force between 100% KI + 30% II and 100% II + 30% KI is adopted in some seismic designs of pile foundations (Hokmabadi, et al., 2018). More theoretical and FEA work is required to provide insights into the combination rules of the non-codified design approach.

This research aims to advance the understanding of the coupling effect of KI and II to formulate combination laws for stocky monopiles in seismic design. Due to the complicated interaction among soil, monopile and wind turbine structure, a simplified model with a representative SDOF system and fundamental mode of turbine are considered (IEC, 2005). The phase relationship between KI and II is analysed first. The contribution of KI and II to the bending moment is then related to the phase relationship to obtain the combination laws. This research performs a series of 3D time-history dynamic FEA with the hysteresis-controlled soil constitutive model. The commercial finite element modelling (FEM) software, PLAXIS 3D V22 Update 2, is used to perform the time domain analysis.

This paper consists of five sections: Section 1 outlines the motivation and existing research findings of the topic; Section 2 details the simulation setup; Sections 3 and 4 introduce model validation and discuss the four representative model results, respectively; Section 5 discusses the results of the parametric study; Section 6 demonstrates the practical application of combining kinematic and inertial loadings. The last section concludes the research findings and outlines suggestions for future research.

2. Simulation setup

2.1. Reference model configuration

The 3D full-space model has 100 m deep Nevada sand, centred at the origin with an extension to both the x and y axes of ± 100 m. The deep soil deposit can reduce the interaction caused by the mesh boundary to the monopile. A parabolic soil profile with an average of $f_g = 0.5$ Hz and shear wave velocity of 587.59 m/s is adopted. Both values start from zero and gradually increase with depth. The monopile is modelled as a constant diameter, D , of 5 m with a 0.1 m thick hollow steel structure. It has a depth, L , of 50 m below the mudline and height of 60 m above the mudline. The stick-up tower is treated as a cantilever with a natural frequency of 1 Hz.

The transitional piece and the mass of the monopile is ignored in the model due to its complexity. Although the stick-up tower would have different modes of response, this research concentrates on the first mode of response of the tower as suggested by the IEC (2005). The turbine and its blades are treated as a lumped mass represented by a 1 m thick plate placed on top of the pile. Rayleigh damping is assigned to the pile and the RNA. A water table of 20 m above the mudline is considered in the model. The cross-section of the model is shown in Figure 2.

The interface element is modelled as two elastic perfectly plastic springs to model the gap and slip displacement. Since the soil near the pile would undergo significant stress and strain changes due to the seismic response, the mesh near the pile is refined by including surface elements with an increasing radius to allow only one element for each gap. A free-field boundary is adopted to absorb incoming waves reflected by the pile structure to simulate a realistic site response and minimise the impacts brought about by wave reflection.

The free-field soil displacement is taken at $x = \pm 50$ m to minimise the free-field boundary condition effects on the mesh boundary as illustrated in Section 3. The bending moment of the plate element for the monopile is represented by one single bending moment. Two artificial ‘soft beams’ are created as shown in Figure 2 (b) to obtain the strain variation of the plate and to calculate an approximate bending moment by considering the engineering beam theory as shown in Equation (1):

$$M_y(t) = -\frac{E \times I_y}{D} \times [\varepsilon_{LHS}(t) - \varepsilon_{RHS}(t)]. \quad (1)$$

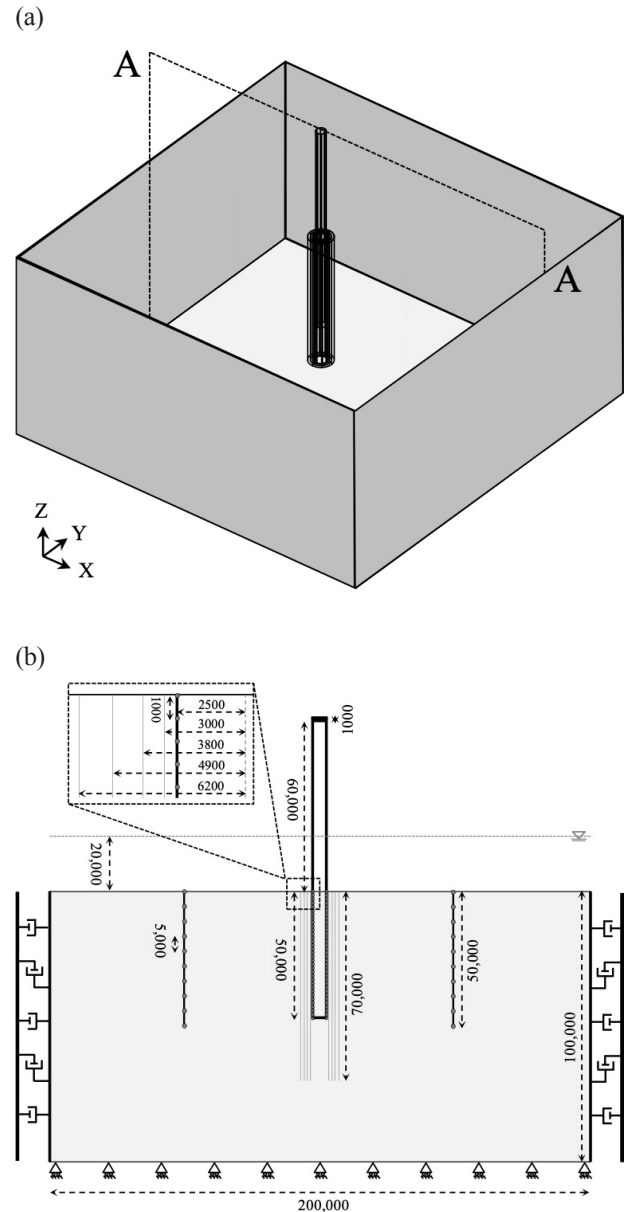


Figure 2. PLAXIS model: (a) perspective view of the reference model; and (b) cross-section AA and boundary conditions of the reference model (all dimensions are in millimetres).

2.2. Earthquake motion excitations

Kobe (PEER, 2014) and Kawasaki (NIED, 2024) motions are chosen as the two reference input base excitations as shown in Figure 3. Kobe motion is a shock-type motion which has a sudden peak that is significantly higher than the rest. Shock-type motion can be generated for near-fault records of seismic events of magnitudes larger than a 6.5 earthquake (Tsaparli, 2017). Kawasaki motion is a vibration-type motion that is characterised by cycles of similar amplitude. Kawasaki, which is a foreshock, has a high frequency of velocity and displacement variation. Both motions are fundamentally

different. Baseline correction and filtering are applied to prevent an unrealistic drift of velocity and displacement after time-history integration from acceleration traces. The base excitation with a real earthquake is applied through the prescribed displacement at the base.

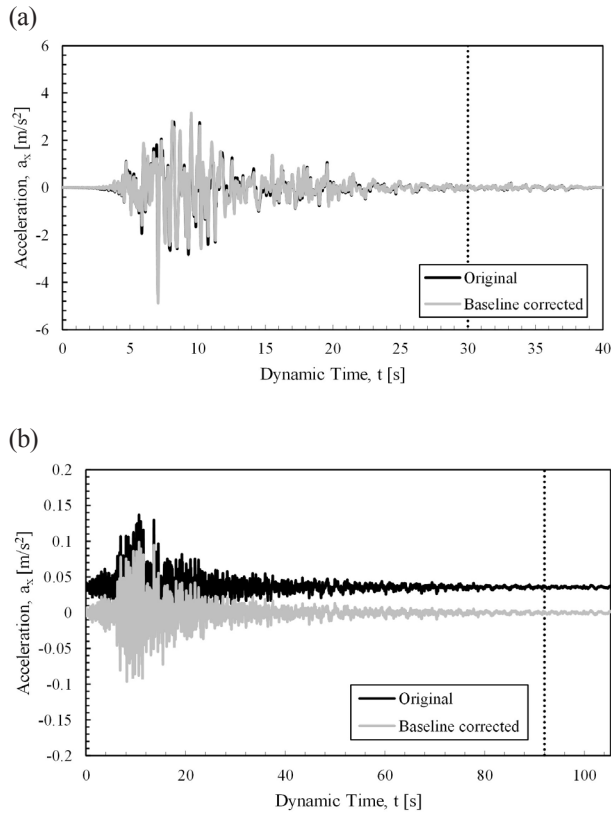


Figure 3. Variation between dynamic acceleration and time: (a) 1995 Kobe earthquake records; and (b) 2003 Kawasaki earthquake records (NIED, 2024; PEER, 2014).

2.3. Dynamic calculation

The calculation comprises three phases in total, including initial, component activation and dynamic excitation. The initial phase initialises all the stress states of the soil body using the earth pressure coefficient K_0 . The second phase is to activate all the structural components to further adjust the stress state before excitation. Note that the pile installation process is not modelled. The last phase is to apply the prescribed displacement to the base to commence the dynamic calculation. Suction is not ignored in all the phases of calculation.

2.4. Model for kinematic loading alone (K flight)

Since two loadings have been combined in the time-history analysis, the KI needs to be manually separated from the combined loading (K+I flight) described in the reference model. The KI bending moment is represented by K flight where a massless RNA is considered. This ensures the consistency of the model in calculating the far-field response.

3. Model validation

3.1. Site response analysis

The model is validated through a 3D site response analysis using a linear elastic soil model. The soil interaction with free-field BCs results in a tiny slight difference compared to the analytical solutions at the mesh boundary. Figure 4 shows the site response results of the 3D full-space model. Different meshing is adopted to check the soil acceleration at the centre of the soil deposit on the ground surface. The acceleration compares well with the analytical transfer function. Although the free-field BCs can absorb the incoming wave, the displacement of the lateral opposite boundaries is different as the dashpots are applying different normal and shear forces to the soil body. The impacts of free-field BCs can also be seen from the dropping accelerations near the mesh boundary. To reduce the impact and ensure a sufficient distance from the pile, the soil domain is set to be 20 times the monopile's diameter in the x direction and two times the pile embedded length in the y direction to minimise the damping effect from the boundary (Bhattacharya, 2019). The far-field displacement profile is decided to be taken at $x = \pm 50 \text{ m}$ as shown in Figure 2. Extra care needs to be taken with the free-field boundary for the PLAXIS 3D dynamic analysis.

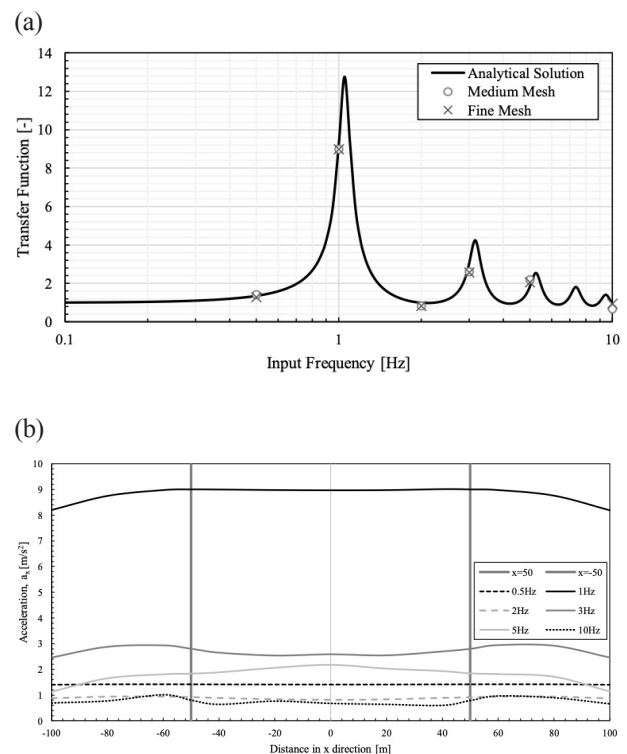


Figure 4. Three-dimensional full-space with free-field site response results: (a) comparison with the transfer function from the analytical solution; and (b) acceleration profile at the ground surface with different input frequencies.

3.2. Transmissibility analysis

After analysing the soil response, the response from the stick-up tower is verified as well. The response of an SDOF system is amplified when excitation is applied to the fixed base of the structure. The amplification of acceleration can be calculated via transmissibility as shown in Equation (2). To ensure no amplification from the site response, relatively stiff soil is used. Figure 5 shows a fair comparison between PLAXIS's results and the analytical solutions.

$$TR \equiv \frac{f_{max}(t)}{p_0} = \sqrt{\frac{1 + (2\xi\beta)^2}{(1 - \beta^2)^2 + (2\xi\beta)^2}} \quad (2)$$

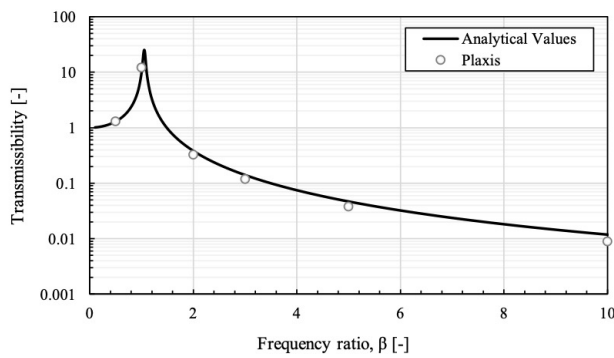


Figure 5. Comparison between the analytical solution and the numerical solution from PLAXIS.

4. Numerical study using Model 04

Imperial College Modelling and Analysis in Geotechnical Engineering (IC MAGE) Model 04 (M04) is a user-defined model implemented with PLAXIS via the IC MAGE Universal Model Interface for PLAXIS (UMIP) (Taborda et al., 2022a). M04 incorporates three main features, including the Imperial College Generalised Small Strain Stiffness (IC.G3S), Mohr–Coulomb failure criterion and enhanced hysteresis control (Taborda et al., 2022b).

4.1. IC MAGE M04 features

The main feature of M04 is its ability to model the hysteresis via a new formulation of the scaling factor, n , for the stress-strain relationship. The Masing rule is widely used to describe the soil cyclic behaviour using a constant factor which would result in the underestimation of the damping ratio in small strains and overestimation in large strains (Taborda and Zdravković, 2012). This is a concern in determining the accurate seismic response of the ground and the superstructure. However, M04 has overcome this limitation by implementing a scaling factor and introduces a set of new model parameters to improve the hysteresis generation of soil (Taborda and Zdravković, 2012).

To investigate the behaviour of M04, the cyclic direct shear test in the PLAXIS SoilTest is performed. The soil test considers a drained, isotropic with $K_0 = 1$ and initial effective vertical stress $\sigma'_{yy} = 100$ kPa conditions. The test is set to be strain controlled with 10 cycles, i.e., the same conditions as in Darendeli (2001). The control strain, γ_c , varies from $1 \times 10^{-5}\%$ to 1% in the test. Figure 6 shows the two important relationships for the soil response under dynamic loadings. The normalised shear modulus degrades with increasing deviatoric strain as shown in Figure 6 (a). The damping ratio curve compares well with the experimental curve from Darendeli (2001).

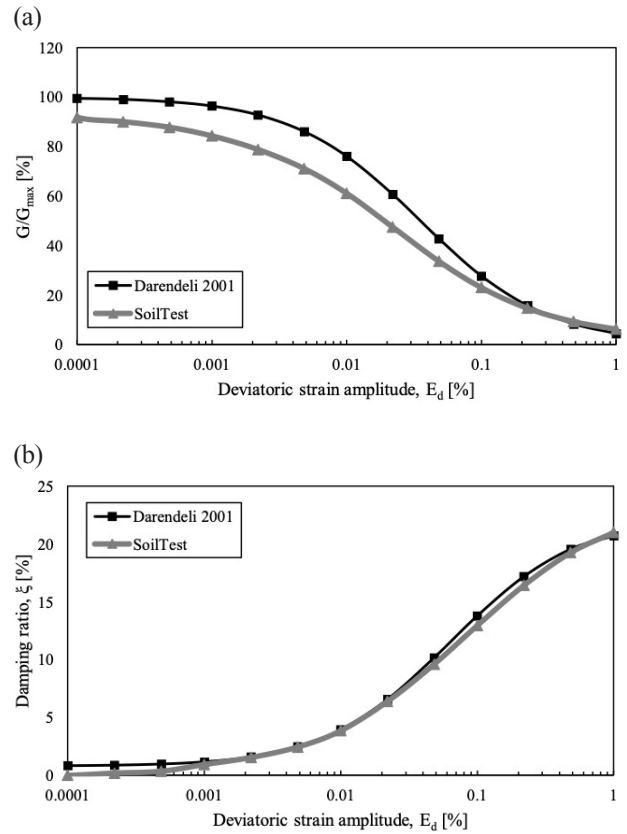


Figure 6. Cyclic response simulated by the M04 and experiments (Darendeli, 2001) – variations with deviatoric strain of (a) secant shear modulus; and (b) damping ratio.

4.2. Results of the four standard Kobe-excited and Kawasaki-excited models

This section considers the four standard cases using M04 with a soil deposit natural frequency (first number of naming) and natural frequency of the superstructure (second number of naming), M04_Kobe_0.5_0.1, M04_Kobe_0.5_0.3, M04_Kawasaki_0.5_0.1 and M04_Kawasaki_0.5_0.1 based on the configuration mentioned in Section 2, to conclude some observations by varying the pile dimensions and different intensities of the motion excitation to verify the proposed rules.

4.2.1. Phase relationship analysis

The phase relationship of the two loadings is first analysed. Three labels are put on each phase relationship diagram to study the time relationship among the three critical values. The maximum inertial force (Max F) represents the maximum dynamic response contributed by the II. The maximum relative ground surface displacement at the far-field (Max GDisp) represents the maximum KI. The maximum bending moment (Max BM) is the resultant loading from KI and II.

Figure 7 shows the phase relationship diagram of the Kobe-excited models. The two loadings at two scenarios are not exactly in phase or out of phase but they tend to show the trend. The engagement of the stress state to the yield surface is observed as the plastic strain is developed. Note that the plastic behaviour can reduce the correlation of the phase relationship due to irrecoverable strain. The average damping ratio for M04_Kobe_0.5_0.3 and M04_Kobe_0.5_1 is 11.76% and 10.16%, respectively. The Max F and Max BM in Figure 7 (a) are very close and occur in the same cycle. This has shown that the time of achieving the maximum II is close to the maximum bending moment. Therefore, II is dominating the overall bending moment. For M04_Kobe_0.5_0.3, the Max F and Max BM is overlapped which means that the maximum bending moment is almost fully contributed by the II alone. However, the Max GDisp for both cases is on the other cycle of the trace which means that the Max GDisp does not occur at the same time as Max BM or the Max F.

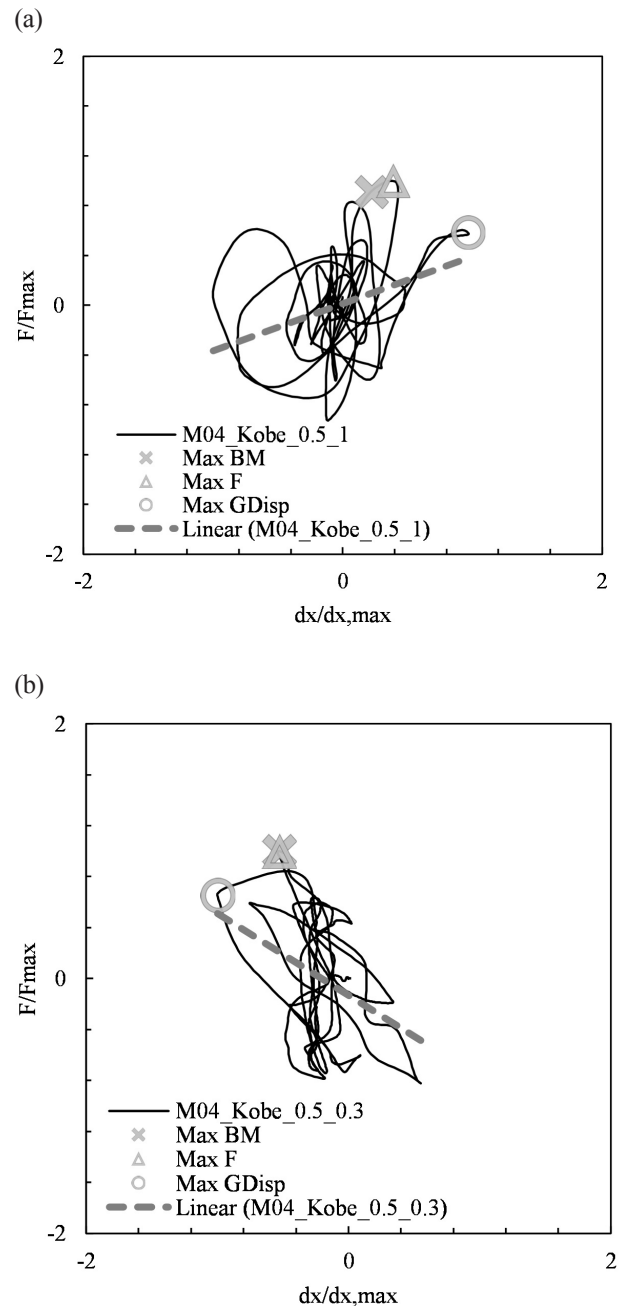


Figure 7. Phase relationship diagrams of KI and II in: (a) M04_Kobe_0.5_1; and (b) M04_Kobe_0.5_0.3.

The phase relationship diagram of the two Kawasaki-excited models as shown in Figure 8 is clearer and has a higher correlation in the relationship than the Kobe-excited models. For the two models, no engagement of the stress state at the far-field to the yield surface is observed due to zero $E_{d,p}$ developed. In addition, the average damping ratio at the far-field of the mudline surface is only 1.05% and 1.63% for M04_Kawasaki_0.5_1 and M04_Kawasaki_0.5_3 cases, respectively. The Max F and Max BM are close to each other, meaning the contribution of the II is high. The Max GDisp does not occur at the same time as either Max F or Max BM.

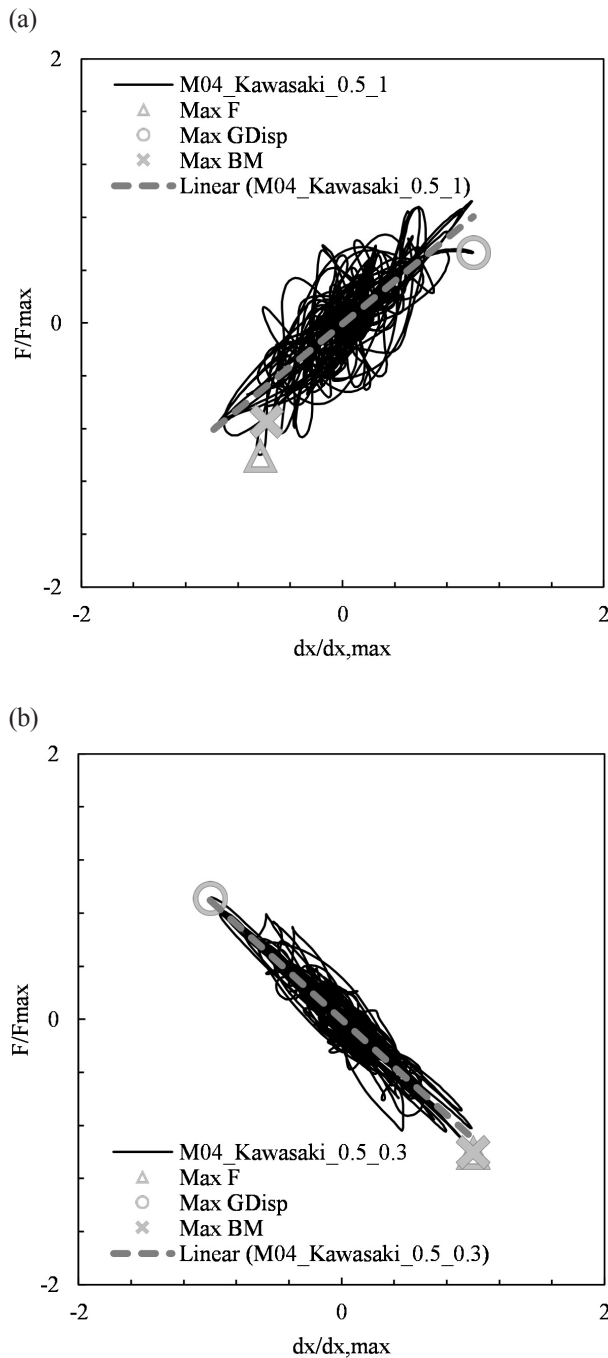


Figure 8. Phase relationship diagram of KI and II in: (a) M04_Kawasaki_0.5_1; and (b) M04_Kawasaki_0.5_0.3.

4.2.2. Bending moment profile analysis

This sub-section concentrates on analysing the bending moment profile of the four standard cases to link the phase relationship to the combination laws. In theory, at the time of the maximum GDisp, the K bending moment should reach its maximum value since the KI is controlled by the displacement load. The maximum bending moment occurs at the part where the II is dominating.

Figure 9 shows the bending moment of the four standard cases. The maximum II bending moment usually occurs near the mudline surface ($-1.01 \sim -5.56 \text{ m}$). However, the KI has its maximum value at around the middle of the pile ($-25.76 \sim -31.31 \text{ m}$). The bending moment profiles show that the bending moment of the pile is dominated by II for the first 5 m in the non-liquefiable soils. The II reduces but the KI increases. The combination of them forms the part named the coupling effect section. With further depth, the II reduces to zero, and the KI solely dominates the overall bending moment for around 10 m .

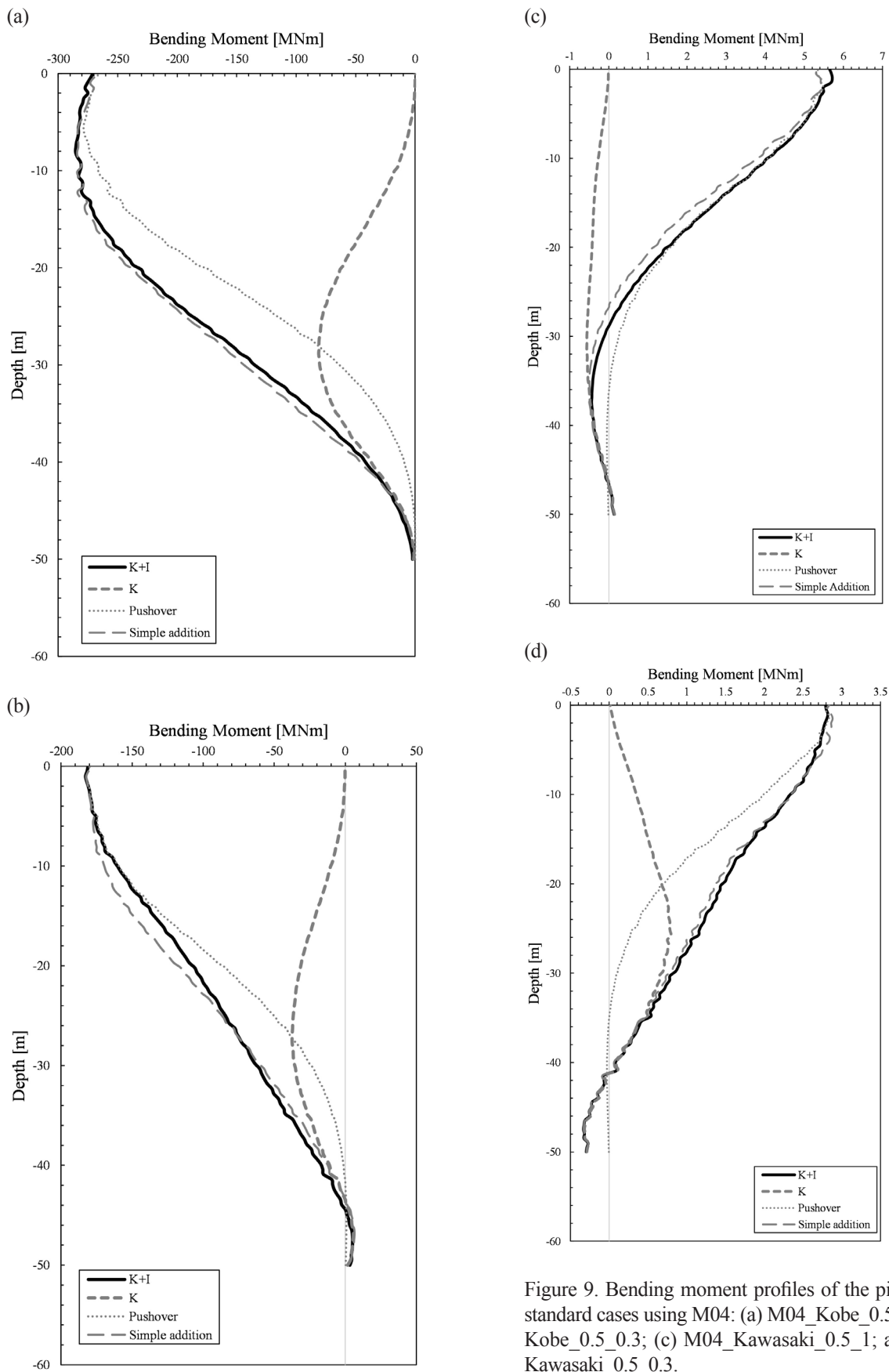


Figure 9. Bending moment profiles of the pile of the four standard cases using M04: (a) M04_Kobe_0.5_1; (b) M04_Kobe_0.5_0.3; (c) M04_Kawasaki_0.5_1; and (d) M04_Kawasaki_0.5_0.3.

Three of the models achieve their maximum bending moment near the surface (-1.01 m) but the maximum bending moment depth for M04_Kobe_0.5_1 is -8.08 m . The lowering of the interface with high G contrast may contribute to the deeper depth of the maximum bending moment. The maximum bending moment in $f_g < f_b$ is larger than $f_g > f_b$.

4.2.3. Combination laws for kinematic and inertial loadings

The above analysis sheds insights into the contribution of KI and II. In this research, the combination laws are first related to the phase relationship. Although the conclusions from the above analysis have shown the dependency of the phase relationship on f_g and f_b , the correlation is not always 1 or -1. However, in this study, the combination laws are largely based on the principal rules when the correlation is 1 or -1 but with slight modification. The II from the pushover and the final bending moment using the simple addition are plotted in Figure 9.

For $f_g < f_b$, the II from the pushover analysis with 5% modification can give the worst-case scenario. Models with an in-phase relationship show that the bending moment signs of KI and II are different. Note that for cases where two curves are observed in a K flight bending moment, only the sign of the upper curve is considered since the maximum bending moment always occurs at the upper part of the deposit. The results with simple addition and the square root of the sum of the squares (SRSS) combination are compared with the true K+I result. Standard deviation is used to measure the scattering of the difference between the combined result and the true K+I bending moment at different depths. Although SRSS has a slightly smaller standard deviation for the Kobe-excited models, the standard deviation differences with SRSS in the Kawasaki-excited models are much larger than the simple addition rule. Therefore, the simple addition rule is considered the better way of combination.

Note that M04_Kobe_0.5_1 is a special case since the bending moment signs are the same for II and KI flights. However, even if only II is considered, the difference from the true K+I bending moment at its maximum value is only 4.15% less. To ensure that the combination law can capture the maximum bending moment value, an additional 5% of II with slight flexibility is suggested. Therefore, the combination law for the case of $f_g < f_b$ in a 100 m deep deposit is to consider 1.05 times the inertial loading alone from the pushover analysis to account for the worst-case scenario.

For $f_g > f_b$, both KI and II with equal weighting needs to be added up via the simple addition rule to obtain the worst-case scenario. In an out-of-phase relationship, the bending moment of II and KI have the same sign. When the two loadings are combined by the simple addition rule, the difference from the true K+I bending moment at its maximum at the same level is below 1.5%. The difference

could be due to the numerical errors in calculation and the small nonlinearity effect of combining KI and II using M04. Therefore, the combination law for the case of $f_g > f_b$ in a 100 m deep deposit is to consider combining equal weights of KI and II using the simple addition rule to account for the worst-case scenario.

5. Parametric study using M04

A conclusion for the phase relationship as well as the combination laws has been given in Section 4.2. However, the conclusions may be valid only for the four standard cases. This section conducts a parametric study to validate the conclusion by changing the pile dimension and adjusting the input motion amplitude at the soil base.

5.1. Stocky monopile ($L/D = 5.9$ & 5) under Kobe and Kawasaki excitation

Two sets of models with two low slendernesses have been performed with both Kobe and Kawasaki motion excitations. The first model set with $L/D = 5.9$ is to change the pile diameter to 8.5 m and the stick-up tower to be 100 m and 240 m to obtain $f_b = 1\text{ Hz}$ and 0.3 Hz respectively. The rest of the parameters remain unchanged. The second model set with $L/D = 5$ is to change the length of the pile from 25 m to 50 m and the height of the stick-up tower to 60 m and 140 m to achieve $f_g < f_b$ and $f_g > f_b$. The dependency of f_g and f_b on the phase relationship is still maintained. No significant change in the bending moment shape is observed. The Max GDisp does not occur at the same time as Max F and Max BM. The time for Max GDisp is not the same as the Max BM of K flight as well.

Similar to Section 4.2, only the case for M04_Kobe_0.5_1 has the maximum bending moment further below the ground surface at -17.68 m and -4.59 m for $L/D = 5.9$ and $L/D = 5$, respectively. For all the Kawasaki-excited models, the maximum bending moment is all at either -1.52 m or -1.02 m . Since plastic strains are experienced by the Kobe-excited models, soil near the surface is degraded, thus causing the lowering down of the interface of high stiffness contrast. This shows that the excitation intensity can also affect the maximum bending moment location. However, the KI only dominates the last 5 m for $L/D = 5.9$ while no KI domination is found for $L/D = 5$.

The bending moment signs are different when $f_g < f_b$ while the bending moment signs are the same for $f_g > f_b$. The results show that the simple addition method has generally smaller differences compared to SRSS. For the $f_g < f_b$ case, the errors are all below 3%. Even if all the in-phase models have the same bending moment signs, the difference with simple addition is below 3%. The modification of 5% can ensure the maximum bending moment to be captured. For $f_g > f_b$, the maximum difference with the true value is around 5%. Therefore, the proposed combination laws are

able to capture the maximum bending moment and the profile of the critical part.

5.2. Models with different scaled Kobe excitations

To verify the impact of the motion intensity on the results, a series of scaled Kobe motions have been applied as the base excitation. The scaling factors are 0.25, 0.5, 0.75, 1.25, 1.5, 1.75 and 2.

The dependency of f_g and f_b on the phase relationship is maintained. However, the shape and the correlation of the phase relationship vary with different intensities. The correlation of the phase relationship is strong at a low intensity and weak at a high intensity. For amplitudes below the scaling factor of 1.25, the trace can cover the region near 1 and -1 of the normalised displacement. However, with a higher intensity, the normalised displacement trace cannot reach close to 1 for both in-phase and out-of-phase cases. The time of Max F and Max BM are close, indicating the large domination of the II of the bending moment. The Max GDisp and Max K do not occur at the same time.

The bending moment signs are not the same when $f_g < f_b$ apart from the cases of the scaling factors of 0.5, 0.75 and 2. In all the in-phase relationships, if only II is considered, the difference in the exact K+I bending moment varies from 2.08% to 4.86%. The difference is due to the position of the maximum bending moment not being close to the mudline surface, varying from -3.54 m ~ -12.12 m. The larger the scaling factor, the deeper the maximum K+I bending moment location. This is due to the degradation of the shear stiffness under the seismic loading such that the interface of large stiffness contrast is lowered. The proposed rule for $f_g < f_b$ with a modification of 5% can still capture the maximum bending moment. Furthermore, no KI dominating effect is observed at the bottom pile for all $f_g < f_b$ models such that considering solely the II should be sufficient. Therefore, the proposed combination law is appropriate for in-phase relationship models.

The bending moment signs are the same when $f_g > f_b$. No special cases are observed. The maximum bending moment depth of the 0.5_0.3 model sets is always -1.01 m. The contribution of KI at -1.01 m ranges from 0.07% to 0.25% of the K+I bending moment, which is negligible. However, the KI dominates at around the last 10 m of the pile while the II is zero at the same depth. By adding both loadings together, the maximum bending moment and the effect from the KI at the bottom pile can be captured together.

5.3. Slender pile ($L/D = 40$) under Kobe excitation

Apart from analysing the stocky monopiles, changing the pile dimension to be very slender can confirm the validity of the proposed combination laws for the slender piles. The slender pile dimension changes to $D = 0.75$ m $L = 60$ m, achieving a slenderness ratio of 40. The heights

of the stick-up towers are also changed to 18 m and 42 m for the $f_g < f_b$ and $f_g > f_b$ cases, respectively.

The results show that the phase relationship dependency is the same as proposed. The maximum bending moment of $f_g < f_b$ is larger than $f_g > f_b$. The II dominates the upper deposit for the first 5 m. The contribution of the K bending moment to the maximum K+I bending moment is only 1.14% and 0.2% for $f_g > f_b$ and $f_g < f_b$. The maximum bending moment is governed by the II. However, the analysis of the time evolution on the K and K+I bending moment shows that KI dominates the pile's bending moment at the depth of around 25 m and 30 m for the cases of $f_g > f_b$ and $f_g < f_b$. This means that more than half of the pile bending moment is fully controlled by the KI which is different from the conclusion that the KI can be ignored given by Gazata (1984).

To capture the bending moment profile of a slender pile, the combination laws also need to consider the KI when $f_g < f_b$. However, if the maximum bending moment value is the governing factor for the design, only II needs to be considered for the two relationships.

5.4. Conclusions of the M04 parametric study results

Soil degradation is the key modelling feature provided by M04 to perform realistic hysteresis control under different seismic loadings. The maximum GDisp does not occur at the same time as the maximum K flight bending moment due to the free-field BCs. An in-phase and out-of-phase relationship is observed in $f_g < f_b$ and $f_g > f_b$ respectively for a 100 m soil deposit. However, the correlation reduces with intensity due to the development of plasticity. The maximum bending moment in $f_g < f_b$ is larger than in $f_g > f_b$. The combination law for $f_g < f_b$ is to consider II alone since KI does not dominate the lower part of the pile. A 5% modification to the maximum II bending moment can ensure the maximum pile bending moment to be captured for the worst-case scenario in which the bending moment signs are the same. For $f_g > f_b$, both KI and II need to be considered as the KI dominates the last 10 m of the pile as the most conservative approach.

6. Comparison with the rules used in practice

The results from this research can be applied to the decoupled analysis for monopile-supported wind turbine foundation design. The kinematic load of the pile can be obtained by applying the free-field soil column displacement to the pile. The bending moment of the pile can be obtained through the pushover analysis using the pseudo-static acceleration obtained from the seismic hazard analysis at the top of the stick-up tower. Hokmabadi, et al. proposed to use a 100%:30% load combination rule for non-liquefied conditions (2018). Kinematic and inertial load is always out of phase regardless of the different soil deposit natural frequencies.

With the proposed combination law, the shear velocity of soil deposits needs to be obtained to assess the natural frequency and compare it with the superstructure. If $f_g < f_b$, the two loadings are out of phase. The bending moment from the decouple analysis can simply be added up to obtain the maximum bending moment of the pile. If $f_g > f_b$, the maximum bending moment can be obtained by considering the inertial loading alone with 5% modification to capture the worst-case scenario. The proposed law has proved that the contribution of kinematic loading to a monopile's bending moment is limited, thus reducing the conservatism in the seismic design of monopiles.

7. Conclusion

As there is only limited research on this topic and the findings are different from each other, this research performs a series of parametric studies to formulate the combination laws for KI and II to avoid the lengthy 3D numerical dynamic time-history analysis and the over conservatism in the preliminary design.

IC MAGE M04, which adopts a new formulation for hysteresis generation and enhanced hysteresis control, is used to model the soil degradation due to seismic loading. A parametric study of different pile dimensions, motion intensity and motion type models is conducted. The dependency of the phase relationship on f_g and f_b remains unchanged regardless of the variation. When $f_g < f_b$, an in-phase relationship is observed. The out-of-phase relationship is observed when $f_g > f_b$. The correlation of the phase relationship reduces with higher motion intensity. The Max GDisp does not occur at the same time as the maximum bending moment on the pile. The soil degradation effect due to seismic excitation in all in-phase relationship models is more significant as the maximum bending moment has a deeper depth. However, for all out-of-phase models, the maximum bending moment is located very close to the mudline surface which may be due to the flexible pile on a stiff soil deposit. The KI is significant for all out-of-phase model cases but not the in-phase relationship models.

To obtain the worst-case scenario for the combination, the phase relationship is a decisive factor. The in-phase relationship indicates that the bending moment signs for the two loadings are not the same. Therefore, only II needs to be considered. An extra 5% to the maximum II is proposed to capture the maximum bending moment to include special cases. Both KI and II need to be combined by the simple addition rule for the out-of-phase relationship to capture the KI at the lower part of the pile.

This project focuses on the theoretical study. A statistical study can be considered to further investigate other combination methods for KI and II. More advanced models such as bounding surface plasticity can be used to simulate the soil liquefaction to observe the contribution of KI. Other variations such as layered soil and tri-directional

excitation can be performed to validate the phase relationship and the combination laws.

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