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Assessment of geotechnical capacities of spread footings due to settlements induced by tunnelling and excavation

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ABSTRACT: This paper presents a framework for the impact assessment of structures on spread footings, taking into account the potential reduction of contact pressure where settlement occurs and the re-distribution of foundation loads due to differential settlement. The models demonstrate that statically determinate structures will likely settle more than the ground, and are capable of accommodating ground settlements with only modest increase in support reaction. The models for stiff statically indeterminate structures suggest that (i) there is greater load re-distribution to neighbouring supports, (ii) there is likely a lag in the first appearance of building settlement in relation to the ground settlement, and (iii) the building settlement as a percentage of ground settlement lags behind but increases with ground settlement. The models here are discussed through two case histories. They function well with observational methods, as both structural stiffness and foundation response are considered, resulting in more realistic settlement predictions.

1 INTRODUCTION

The impact of underground works to adjacent structures consists of both settlement and differential settlement, and the latter is normally considered more critical. Differential settlements also induce load re-distribution between supports but this is normally ignored to-date.

For buildings with deep foundations, the loss in mobilized resistance along the pile due to ground settlement could be recovered by mobilizing more resistance from the soil layers which are not affected along the pile body. The displacements required to remobilize the imposed load on the pile can be assessed based on load transfer analysis, i.e. *t-z* and *q-z* analysis (Boon & Ooi, 2016).

For buildings with shallow foundations, the loss in mobilized resistance due to ground settlement can also be recovered but requires the foundation to move with the ground profile, as the resistance has to be generated through the contact pressure at the spread footing. If the structure is stiff and the unloading of the foundation created by the settling ground cannot be closed, the support reactions have to be re-distributed.

However, to-date, to obtain a first assessment of the impact of tunnelling and excavation to structures on shallow foundations, the solutions for masonry structures based on Burland & Wroth (1974) and Boscardin & Cording (1989) are still used widely, and the damage is quantified using tensile strains. The calculations to estimate the tensile strains assume that the deformations of the structure are compatible with the ground, and the bearing capacity of the ground is not affected. For framed structures supported on spread footings, these calculations are less representative, as the loads applied on the ground are discrete rather than uniformly distributed. Attempts to predict the settlements of discrete footings of a framed structure have been reported in Franza & DeJong (2017). Nevertheless, the study of load re-distribution between supports is limited. Although the phenomenon of load re-distribution and loss in mobilized resistance of shallow foundations are generally considered less critical than the consequence of structural deformations, this depends on a few factors. The appreciation of the mechanism of load re-distribution would be useful in practice to facilitate engineering judgement.

The objective of this paper is to set out a framework which helps engineers to evaluate in a more systematic manner whether the bearing capacity of the foundations are compromised, and evaluate if more rigorous analyses or protective works may be required potentially based on the predicted settlements.

2 PROPOSED MODELS

In this paper, it is assumed that the two main parameters affecting the performance of the spread footings are the contact stiffness and bearing capacity, and it is important to identify if these are affected at the outset of the assessment. The models here are developed based on statics. The advantage is that it can be adapted easily to similar problems, without the concern of inaccurate calibration factors due to changes in boundary conditions, and can be solved immediately by commonly available structural program with familiar input parameters.

2.1 *Structure-Foundation Response*

The models for a statically determinate and indeterminate structure in response to ground settlement are distinguished here, as the mechanism of load re-distribution is different.

2.1.1 *Statically Determinate Structure*

For a statically determinate structure, the bending stiffness of the structure has negligible influence to the foundation reaction loads, because the structure will have to accommodate the ground movement. Ground settlements inducing differential settlements are capable of inducing eccentricity and this can cause an increase in support loads at the edge where the structure is leaning forward. A model to approximate this mechanism is shown in Figure 1.

The building settlement is a function of both the ground settlement and the contact compression at the foundation. If the distortion of the ground due to settlement is defined as i, and the additional foundation rotation to develop more contact pressure is defined as θ, then the total building rotation is $i+ \theta$. The calculation procedures are discussed in Appendix A. The model here suggests that the building settlement is likely to be greater than the ground settlement for a determinate structure, and will be apparent if the contact stiffness of the foundation is compromised due to ground settlements induced by migration of fines.

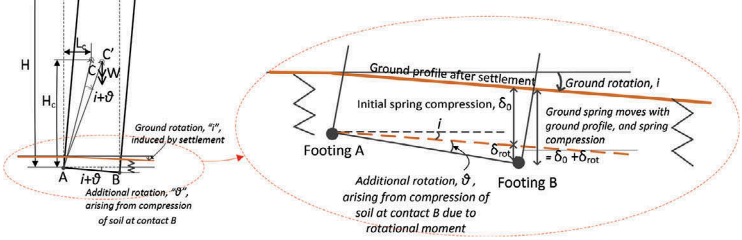

Figure 1. Model for a statically determinate structure with two spring supports, A and B, where the moment due to eccentricity of the structure is resisted by additional compression at support B.

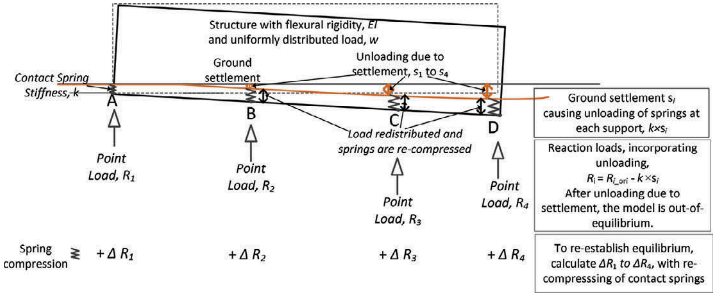

Figure 2. Adopted model for a statically indeterminate structure with multiple supports.

2.1.2 *Statically Indeterminate Structure*

For a statically indeterminate structure, there are redundant supports, and the loss in mobilized contact pressure at a foundation, can be compensated through load re-distribution to neighbouring foundations.

For a stiff structure, the structure is more efficient in re-distributing loads to neighbouring supports, and the movement of the structure will be less compatible with the ground. For a more compliant or flexible structure, the structure is less efficient in re-distributing loads, and the structure will accommodate the ground movement resulting in more compatible movement with the ground. The support reactions are not likely to experience large changes in magnitudes for a compliant structure.

The model adopted is shown in Figure 2, where the reaction loads, after taking into account of the unloading due to soil settlement, are assigned as upward point loads. To recover the reaction loads to re-establish equilibrium, the contact springs are compressed, based on the flexural rigidity of the equivalent beam. The calculation procedures are discussed in Appendix B, but common structural programs can also be used by assigning similar boundary conditions. It is noted that conventional p-Δ frame analysis may overestimate building strains, because either incomplete boundary conditions are specified, or the existing reaction loads are ignored.

3 CASE HISTORIES

Two case histories comprising statically determinate and indeterminate structures are discussed.

3.1 *Pylon Structure*

A pylon structure was located nearby an excavation in the karstic Kuala Lumpur Limestone Formation (see Figure 3). The foundation of the pylon consists of pad footings and they were monitored prior to the adjacent excavation (Figure 4a). During excavation, settlement was

Figure 3. Nearby pylon structure adjacent to a deep excavation.

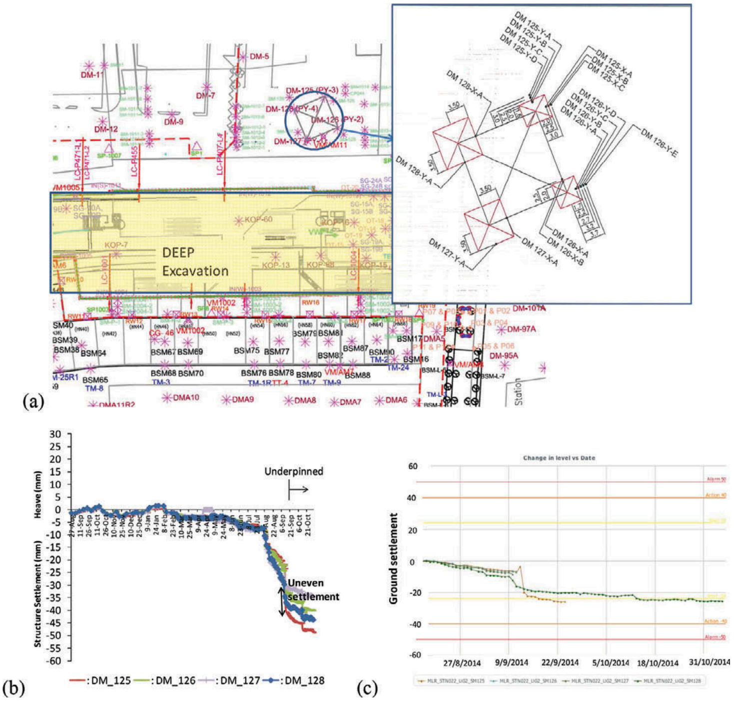

Figure 4. Settlement of pylon nearby a deep excavation: (a) layout plan, (b) settlement of structure, (c) settlement of ground.

measured. The difference in settlement between supports was increasing up to approximately 15 mm (Figure 4b).

The implication of differential settlement to the foundation forces can be approximated and simplified in 2-D as a rigid beam with two spring supports. For a beam with two supports, the structure is statically determinate. The estimated ultimate bearing capacity of the pad footing

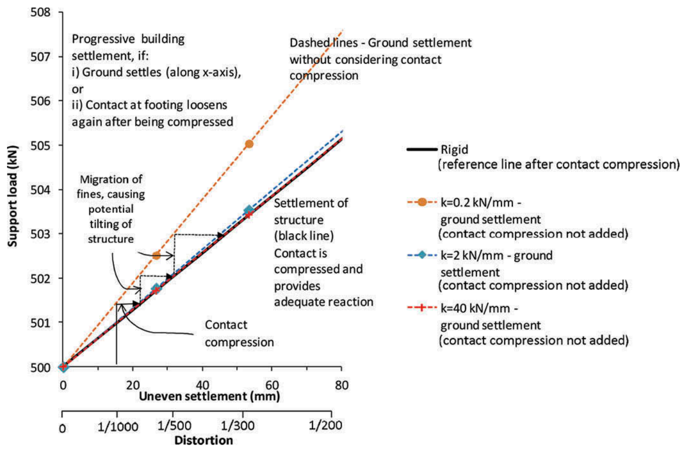

Figure 5. Influence of uneven settlement on support load, using the solution in Appendix A. It was assumed that each support was originally carrying 500 kN. The contact stiffness of 40 kN/mm was estimated using Young's modulus 10000 MPa for a pad footing of 4 × 4m. The contact stiffness of 0.2 kN/mm shows the influence of loss of contact stiffness. Span of structure is 15.3 m.

(4 × 4 m) for an undrained shear strength of 20 kPa for the soil was estimated to be around 1600 kN, or an allowable load of 533 kN. This is coherent with the estimated foundation design, with the pylon weight estimated to be around 2000 kN.

Using the model discussed in Section 2.1, it is shown in Figure 5 that an uneven settlement between the supports of 15 mm induce only marginal additional loads (y-axis). The model shows that the support loads of determinate structures are not very sensitive to small magnitudes of differential settlements especially below the typical allowable tolerance ranging between 1/500 and 1/300. Figure 5 shows how progressive movement of the structure may happen due to loosening of the contact pressure (staircase lines in Figure 5), if there is migration of fines underneath the footing. As the settlement measured at the structure was greater than the pylon (Figure 4 (b) and (c)), this was believed to be a plausible mechanism of settlement.

In the case history here, Tube-A-Manchette as well as compaction grouting was carried out. This was followed by the installation of underpinning micropiles socketed 4.5 m into limestone rock. The pylon however was subsequently relocated to accommodate a future underground entrance to a commercial development.

3.2 *One-Storey Structure*

A one-storey structure was in the path of the tunnel boring machine (TBM) in the karstic Kuala Lumpur Limestone Formation. There were two incidents. In the first incident, a depression was detected, and settlement was recorded at the edge of the building, as the TBM stopped for intervention, as shown in Figure 6. In the second incident, after tunneling, ground penetration radar scanning was carried out, and it was found that separation developed between the ground and the slab midspan. This was confirmed also through coring.

The ultimate bearing capacity was estimated to be 900 kN for a footing size of 3×3 m for a ground with an undrained shear strength of 20 kPa. The original support loads are estimated to be in the range of 320 – 420 kN depending on the location of the supports.

In this study, two cross sections were analysed (Figure 7). The first cross section (A-A) analyzed with ground settlement at the first edge foundation shows that the support immediately

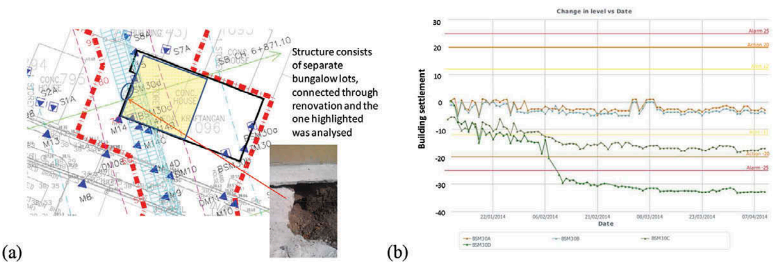

Figure 6. Settlement of a one storey structure due to tunnelling: (a) layout plan, (b) settlement trend of 3 building settlement markers, (c) settlement trend of the settlement marker with the largest magnitude.

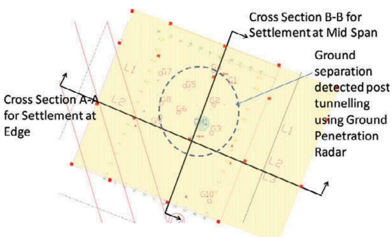

Figure 7. Analysed Cross-Sections. Red dots are foundation positions.

adjacent to the edge may increase by approximately 300 kN, with a total reaction load of 670 kN (Figure 8 (b) and (c)). This happens when the ground settlement is large enough until separation occurs and no load is transferred to the ground from the structure at the edge support. Unloading was calculated at the opposite far end, due to the flexural rigidity of the structure. However, the measurement of heave is unlikely to occur in practice. The predicted settlement of 16 mm difference in magnitude between the two end supports was calculated, and this is in the same order of magnitude compared to field measurements (17mm and 32mm at the two ends respectively). The larger magnitude of measured settlement may be possibly due to the greater extent of ground settlement affecting more than one foundation support (related to the second incident).

Using the model in Section 2.2, the building settlement as a fraction of ground settlement is plotted in Figure 9. The results show that the first 10 mm of ground settlement will result in

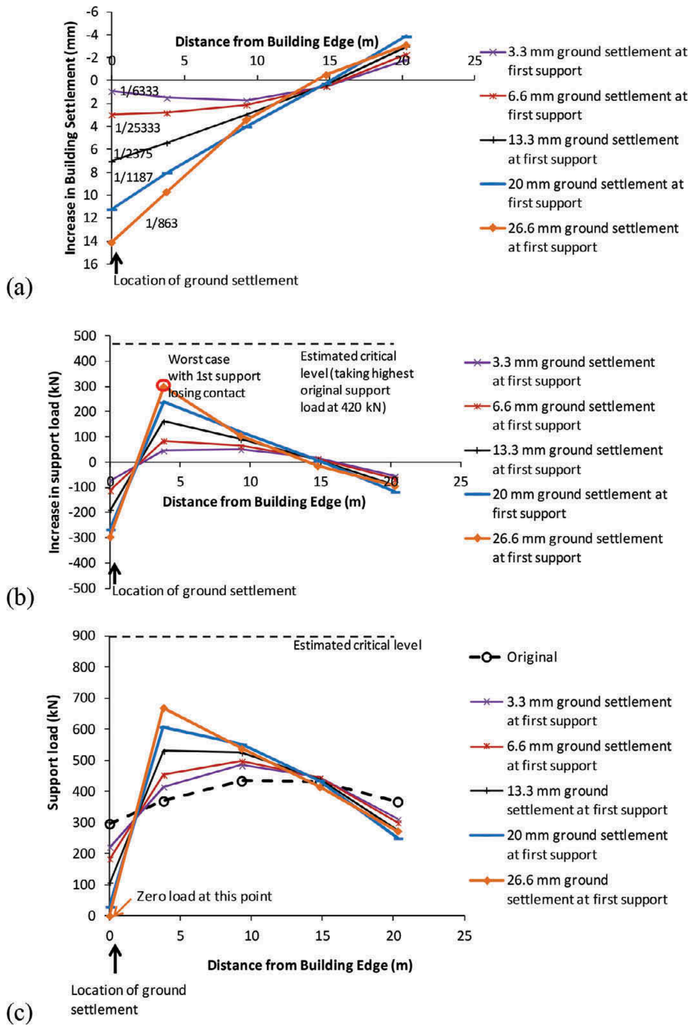

Figure 8. Impact of loss of foundation support stiffness at the edge of the building (Cross Section A-A): (a) increase in building settlement, (b) increment in support loads, and (c) support loads. Solution was obtained using solution in Appendix B, with spring support stiffness of 30 kN/mm assuming that the ground Young's modulus was 6.5 MPa and footing size of 3 × 3 m. The building stiffness was estimated using a 200 mm thick slab and with a parallel axis theorem using the midheight of the structure (3m).

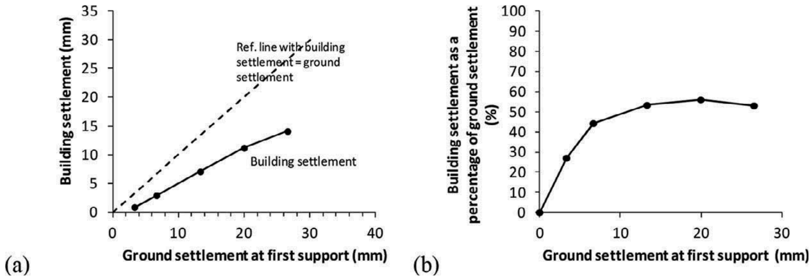

Figure 9. Building settlement as a function of ground settlement in terms of (a) magnitude, (b) percentage.

less than half of building settlement. At even smaller magnitudes, i.e. approximately less than 5 mm of ground settlement, the building settlement may not manifest with the tolerance of measurement accuracy. The building settlement as a percentage of ground settlement increases with larger magnitudes of ground settlement up to approximately 60%. This lag of building settlement compared to greenfield ground settlement predictions has been also observed in other project sites (Boon et al., 2016).

In the cross section (B-B) analysed for the second incident, the loss of support stiffness at the midspan led to an increase of edge support by 280 kN, and it also led to en-block settlements of the buildings (Figure 10), due to greater soil compression at the foundations (now taking larger loads).

The influence of flexural rigidity of the structure was studied using the first cross-section (A-A) with the edge foundation compromised, and the results are presented in Figure 11. The flexible structure is more prone to distortion, as it is more sensitive to ground settlements. The stiff structure experiences less settlement and distortion, exhibiting more linear settlements across the building, but distributes loads to adjacent supports.

In Figure 12, the response of the structure with decreasing stiffness at the edge support is shown, for different magnitudes of structural flexural stiffnesses. The flexural stiffness was estimated using the parallel axis theorem, taking the neutral axis at the midheight, for different ground slab thicknesses and assuming that the roof truss offers little rigidity. The case where the neutral axis is taken at the ground level was also compared. The results show that the ground slab thickness has little impact to the structural response, but the location of neutral axis has a major impact. The location of the neutral axis depends on the framing of the reinforced concrete structure, and is beyond the subject of study in this paper. The contribution of column stiffness to the flexural rigidity is discussed in Goh & Mair (2014). The results in Figure 12 show that flexible structures are more responsive to changes in ground stiffness and settlements.

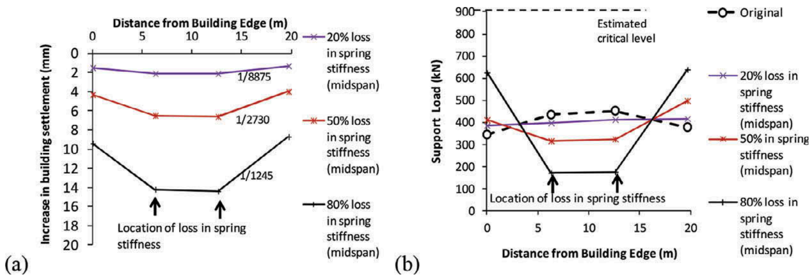

Figure 10. Impact of loss of foundation support stiffness at the midspan of the building (Cross Section B-B): (a) increase in settlement, and (b) support reaction loads.

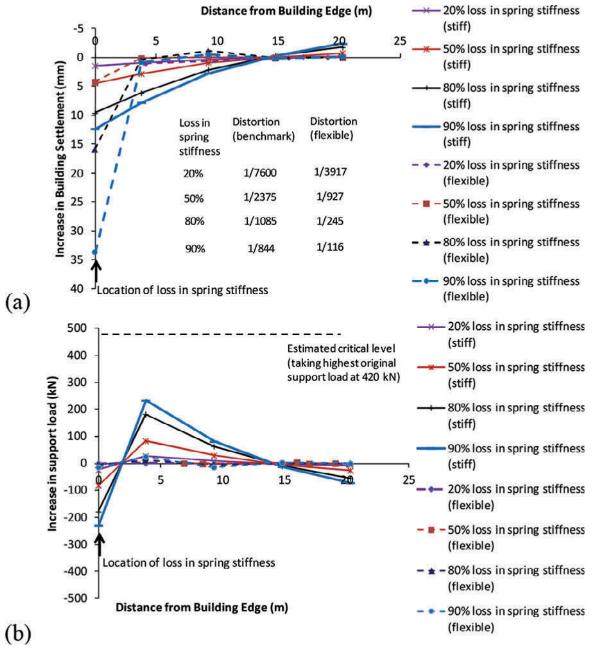

Figure 11. Influence of flexural stiffness of structure with EI = 12000 MPa/m^4 and EI=17.3 MPa/m^4 in terms of (a) settlement and (b) reaction loads, due to settlement at edge foundation.

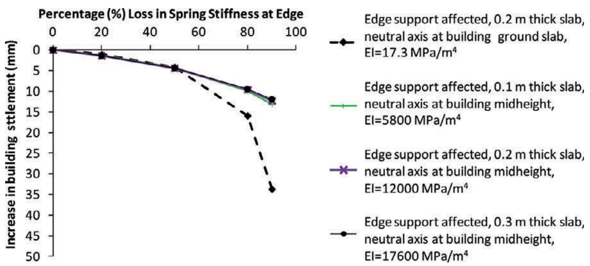

Figure 12. Influence of flexural stiffness of structure to the building settlement.

4 CONCLUSION

The impact of ground settlement to structures with discrete loads on spread footings was studied, and two case histories were discussed. The response of the structure depends on the mechanism of the ground settlement, the flexural rigidity of the structure, and whether the structure is statically determinate or indeterminate.

For statically determinate structures, the building settlement will likely exceed the ground settlement when there is distortion of the structure. Most statically determinate structures, depending on the location of centre of gravity and span, are likely able to accommodate differential settlements, within the typical allowable tolerance ranging between 1/500 and 1/300 adopted in most projects, with modest increase in support reaction force. This range of differential settlements are two orders in magnitude smaller compared to those discussed in Burland et al. (2003), with an angle of rotation of ~5.4° or ~1/10 for the leaning tower of Pisa. However, as the structure is statically determinate, the loss of bearing capacity and stiffness at any support due to migration of fines underneath the foundation has to be reviewed. Progressive settlements may occur as a result of continual loss of contact pressure due to the migration of fines.

For statically indeterminate structures, the response of the structure depends on the flexural rigidity of the structure. For compliant structures, the settlement of the structure will be more or less compatible with the ground settlement, and the structure is more prone to distortion. For typical reinforced concrete structures with sufficient flexural rigidity, the structure is less prone to distortion. However, some of the support loads can almost double in magnitude as a result of load re-distribution for plane-strain problems, when the contact at the neighbouring foundation is lost. However, this may be within the bearing capacity, provided an original factor of safety of 3 was available. This is likely the reason why incidences of bearing capacity problems associated to underground works are uncommon. The model here can capture the delay in the appearance of building settlement in relation to settlement. It was found that the building settlement as a fraction of ground settlement increases and goes up to 60% for a moderate one-storey reinforced concrete structure. Attempts were also made to model the impact of a depression where contact pressure is lost in one of the foundations.

The calculations in this paper assume that both the structure and foundation respond elastically. The influence of moment and horizontal loads on the bearing capacity (Nova & Montrasio, 1991; Houlsby, 2016), and the influence of plasticity and creep have not been studied. Nonetheless, the models discussed here have been able to reflect typical observations encountered in practice, and will function better with observational approaches during construction.

5 APPENDICES

5.1 *Appendix A: Load-settlement response of determinate structure*

In the model in Figure 1, i is the distortion induced by the ground settlement, and θ is the additional rotation for soil compression to develop the additional reaction. Taking moment at support A, the driving moment M_{D} for a body with weight W can be expressed as:

$$M_D = W[H_c sin(\theta + i) + L_c \cos(\theta + i)] \tag{1}$$

where H_{c} and L_{c} are the horizontal and vertical distance to the centre of gravity. The resisting moment M_{R} can be expressed as:

$$M_R = R_{\mathrm{new}}[L \cos(\theta + i)] \tag{2}$$

where R_{new} is the reaction force to maintain equilibrium. For a rigid foundation, R_{new} can be calculated easily with θ being nil. For a compliant foundation, R_{new} can be calculated as:

$$R_{\mathrm{new}} = R_{\mathrm{ori}} + kL(sin(\theta + i) - \sin\theta) \tag{3}$$

where k is the support spring stiffnesses. To satisfy equilibrium, i.e. $M_{\mathrm{D}} = M_{\mathrm{R}}$, the unknown θ can be calculated using the MS Excel Solver tool.

5.2 *Appendix B: Indeterminate structure subject to changes of support reactions*

An example for a beam with three spans (l_1, l_2 and l_3) and four supports (R_1, R_2, R_3 and R_4) with uniform load, w, is discussed (see Figure 2). The equations can be modified to incorporate more spans and more supports. From Castigliano's theorem, the energy, U, can be expressed as (Boresi et al., 1993):

$$U = \int_0^{l_1} \frac{M_1^2}{2EI} dx + \int_{l_1}^{l_1+l_2} \frac{M_2^2}{2EI} dx + \int_{l_1+l_2}^{l_1+l_2+l_3} \frac{M_3^2}{2EI} dx + \frac{R_1^2}{2k_1} + \frac{R_2^2}{2k_2} + \frac{R_3^2}{2k_3} + \frac{R_4^2}{2k_4} \tag{4}$$

where M is the bending moment at each span, EI the bending stiffness, and k is the contact spring stiffness. It is assumed that there are no moments at the two ends of the beams, i.e.

$$\frac{w(l_1+l_2+l_3)^2}{2} = R_1(l_1+l_2+l_3) + R_2(l_2+l_3) + R_3 l_3 \tag{5}$$

$$\frac{w(l_1+l_2+l_3)^2}{2} = R_4(l_1+l_2+l_3) + R_3(l_1+l_2) + R_2 l_1 \tag{6}$$

Eq. (5) and Eq. (6) allows R_1 and R_4 to be substituted into Eq. (4). The remaining supports are redundant supports. This leads to the following equations $\frac{\partial U}{\partial R_2} = 0$ and $\frac{\partial U}{\partial R_3} = 0$, solving which gives R_2 and R_3. Once R_2 and R_3 are obtained, R_1 and R_4 can be calculated from Eq. (5) and (6). These values are the original reaction loads R_{i_ori} before the presence of settlement.

To model the influence of ground settlements (see Figure 2), point loads are assigned at the supports $R_i = R_{i\_\mathrm{ori}} - k_i \times s_i$ where $k_i \times s_i$ are the unloading of springs due to ground settlement. If R_i is negative, it indicates the presence of a gap that has to be closed before compressive reaction forces can be developed. The point loads R_i are treated as external loads in the subsequent calculation. To obtain equilibrium, the contact springs have to be compressed by ΔR:

$$U = \int_0^{l_1} \frac{M_1^2}{2EI}\,dx + \int_{l_1}^{l_1+l_2} \frac{M_2^2}{2EI}\,dx + \int_{l_1+l_2}^{l_1+l_2+l_3} \frac{M_3^2}{2EI}\,dx + \frac{\Delta R_1^2}{2k_1} + \frac{\Delta R_2^2}{2k_2} + \frac{\Delta R_3^2}{2k_3} + \frac{\Delta R_4^2}{2k_4} \tag{7}$$

At equilibrium, when any support load $R_i + \Delta R_i$ is negative, the analysis is repeated by replacing the spring stiffness with a very small number, and the point load (simulating the foundation reaction) at the support is removed from the calculation. In this paper, a symbolic mathematical toolbox Sympy operating in the Linux system was used to calculate the solutions. Eq. (4) has been benchmarked with StaadPro and the same solutions in Figure 11 were obtained.

Alternatively, the effects of settlement in Figure 2 can also be calculated using commonly available structural programs, in which case the springs are active at the outset, and are also interacting with the assigned reaction loads (in the opposite direction) and imposed load, w, in contrast to the formulation in Eq. (7). The differences in the solutions were found to be small.

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