Design loading of deep foundations subject to sinkhole hazard

Charges de calcul appliquées aux fondations profondes soumises à des risques de cratères d'effondrement

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ABSTRACT

Deep foundations such as piles or caissons are often designed to withstand significant lateral forces. This is the situation for most offshore structures and for foundations situated on slopes for example. It is often assumed that the maximum lateral pressure that soil will exert under such conditions is three times the Rankine passive pressure, $3K_p$.

This paper presents the findings of a study into deep foundations subjected to loading from soil collapsing into a deep sinkhole in dolomitic ground. Typically the resulting hole will have near vertical sides which are quasi-stable. In designing deep foundations in such ground conditions, it is necessary to consider the lateral forces the soil can exert as it flows past the foundations when these quasi-stable sides collapse. Three-dimensional non-linear finite element techniques were used to model the flow of soil around the foundation into the sinkhole. This differs from typical lateral loading problems in that the horizontal stresses within the soil normal to the edge of the sinkhole reduce to zero at the edge.

For the deep foundations it was found that the peak pressure was significantly lower than $3K_p$, and only acted over a relatively narrow depth range on the foundations. The results of the finite element work were then used to develop simplified rules on the loading of these foundations from sinkholes of any dimension and at any location relative to the foundation for use in design.

RÉSUMÉ

Les fondations profondes, telles que les pieux et les caissons, sont souvent conçues pour supporter des efforts latéraux importants. C'est le cas, par exemple, pour la plupart des structures offshores et pour les fondations situées sur pentes. Il est généralement admis que la pression latérale maximale exercée par les sols dans ces conditions est égale à trois fois le coefficient de butée de Rankine, $3K_p$.

Cet article présente les résultats d'une étude concernant des fondations profondes soumises aux effets de glissements des sols lors de l'apparition de cratères d'effondrement. Généralement, la cavité résultant de l'affaissement des terres possède des parois proches de la verticale et quasi-stables. Lors de la conception de fondations profondes dans des sols de ce type, il est nécessaire de prendre en compte les forces générées par le glissement des terres au contact des fondations lorsque les parois quasi-stables du cratère s'affaissent. Des méthodes éléments finis 3D ont été utilisées pour modéliser l'écoulement des sols autour des fondations lors de l'effondrement. Ceci diffère des problèmes de chargements latéraux habituels car les contraintes horizontales dans le sol deviennent nulles sur la surface libre du cratère.

Dans le cas de fondations profondes, on observe que la pression maximale est sensiblement inférieure à $3K_p$ et qu'elle s'exerce sur une section des fondations de profondeur limitée. Les résultats de ces analyses éléments finis ont été utilisés pour développer des règles de calcul simplifiées pour le dimensionnement de fondations soumises à l'apparition de cratères d'effondrement de dimensions et de positions quelconques.

Keywords : karst, dolomite, limestone, 3D finite element, deep foundations, piles, caissons

1 SINKHOLE FORMATION AND CONSEQUENCES FOR DESIGN

Regions of karst present particular geotechnical hazards, of which one is the collapse of sinkholes that form in residual soils above chemically weathered rock. The formation of sinkholes in areas of karst is discussed in the literature (Waltham et al. 2005). This study was concerned with subsidence sinkholes of the 'dropout' type. Initially a cavity forms in underlying limestone / dolomite due to chemical erosion by the passage of groundwater. With percolation of water from the surface the residual soils can be eroded into the lower cavity to form a void in the soil. This void expands by continued erosion and collapse until the roof of the void is unable to support itself and collapses, resulting in a sinkhole at the surface.

The nature of the overburden determines whether a 'suffosion' or 'dropout' sinkhole occurs. Suffosion sinkholes occur in coarse soils and are characterised by a gradual surface subsidence. Dropout sinkholes result from erosion causing a void within the residuum which enlarges over time until the roof collapses, at which point the void becomes exposed at the surface as a sinkhole. Sinkholes require the formation of a stable or quasi-stable void within the residuum, so are found in cohesive and / or cemented soils, and often require support by karstic pinnacles of rock.

Dropout sinkholes tend to result in a hole with near-vertical sides that were supported by the arching stresses in the soil prior to the collapse of the roof. These vertical side then collapse into the hole over a period of time. For piled foundations to rock, the opening of the sinkhole is not the hazard, but rather the lateral soil movements that follow when the perimeter of the initial sinkhole begins to collapse.

In order to design deep foundations in karstic ground it is necessary to understand their response to sinkholes that could

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occur in their immediate vicinity. Sinkholes adversely affect different foundation types in different ways. Shallow foundations and floating piles may lose vertical support. Floating or rock-socketed piles lose lateral support and are also subjected to vertical and lateral earth pressures caused by adjacent soil collapsing into the sinkhole. In all cases, a thorough analytical treatment of the effects of sinkhole formation is complex.

This paper presents a study which considered the magnitudes of the lateral loading on piled foundations when the near vertical sides of a dropout sinkhole collapse. The piles were assumed to be socketed into the underlying dolomite to provide a robust anchor for the foundation that can not be undermined by the development of the sinkhole.

2 LATERAL LOADING OF PILES

Broms (1964) determined that the ultimate resistance of a pile moving through soil was related to $3*K_p$, where K_p is Rankine's coefficient of passive pressure. Later, Reese et al. (1974) proposed a different relationship based on the testing of offshore hollow steel piles. The results were similar, but slightly higher.

All of this work was based on piles fully embedded in soil in which vertical and horizontal stresses increase with depth. Where piles are affected by sinkholes, the lateral soil stresses are significantly altered in the ground. In the extreme location at the edge of the sinkhole, the radial stress in the soil is reduced to zero. With the minor principal stress so low, the soil can not generate the large pressures predicted by Broms (1964) or Reese et al. (1974). Thus, if the findings of these authors were adopted for the design of piles affected by sinkhole collapse, then the design would be 'safe', but probably very uneconomic.

Because of the complex three-dimensional geometry and the transient nature of the loading, resolving this problem is beyond simple analysis. 3D finite element (FE) analysis was thus undertaken for some general cases, the results of which were used to determine general 'empirical' rules which could be implemented in more standard analysis software.

3 FINITE ELEMENT MODEL

3.1 *Model Parameters*

This problem was modelled using LS-DYNA, a general purpose 3D nonlinear FE program (Livermore Software Technology Corporation 2007) that is particularly suited to large 3D models involving high degrees of nonlinearity.

The purpose of the FE model was to provide generic detail on how the soil around a dropout sinkhole moves into the hole when the arching stresses are released.

The soil was modelled as a dry Mohr-Coulomb material with effective stress strength parameters of $\varnothing' = 30°$ and $c' = 0$kPa. This constitutive model was appropriate for these analyses since highly plastic behaviour was to be modelled in soils that were very heterogeneous and not well understood. Since the analyses were undertaken to develop a simplified design method rather than to assess a particular situation, a more sophisticated constitutive model was not warranted. The sensitivity of the results to the soil parameters was reviewed but in respect of providing generic design cases the actual value used was found not to be important. In fact, in order to develop a sinkhole of this type, significant cohesion is required, and to model this loss of cohesion suddenly was most probably conservative.

The formation of sinkholes was modelled by instantaneously removing a cylinder of soil. This is extreme but shows how the soil flows in to the hole in the worst case. Elements were permitted to fall into the hole; once the confining stress on these elements disappears they become highly deformed but are then irrelevant to the solution because they carry no stress.

The piles were modelled as linear elastic beam elements, with a "skin" of non-structural shell elements supported at the outer diameter of the pile by rigid elements. There was no mesh connectivity between soil and pile, but the 'skin' around the pile provides a contact surface for the soil.

Before creation of the cavity the soil was assumed to be normally consolidated. Additional initial horizontal stress was provided around the piles which increased K_0 to approximately 0.5 local to the piles. The piles were assumed to be rigidly fixed into the dolomite rock below the residuum material.

3.2 *Geometry of Problem*

A foundation comprising four 1.2m diameter rock-socketed piles was modelled explicitly. The sinkhole was centred on the symmetry plane of the pile group, which results in the most onerous lateral displacements of the pilecap and which simplified the modelling. Figure 1 shows the FE model in its initial condition.

Only the piles and pilecap were modelled with no restraint being applied to the pilecap. Bedrock was selected to be at 11m because it was necessary to have a deep enough model to be able to investigate the mechanisms of failure, but not so deep that the analyses took a prohibitive amount of time to run. The sinkhole was modelled with a diameter of 15m, extending from the ground surface to bedrock. Parametric studies were then undertaken to review the results of this analysis. They are beyond the scope of this paper, but gave confirmatory results.

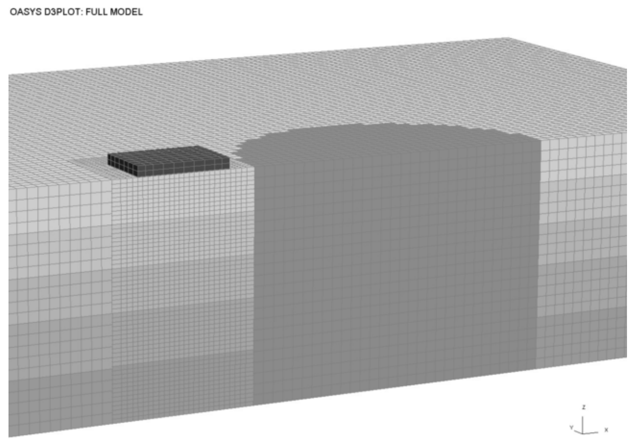

Figure 1. Finite element model in its initial condition

4 RESULTS OF FE MODELLING

4.1 *Observations on mode of failure in model*

Figure 2 shows a snapshot from the FE analysis. It shows how the soil failed into the sinkhole during the analysis. The soil formed a 'wedge' of strain around the sinkhole, with maximum soil movement associated with a comparatively thin region near the soil surface. As the failure progressed the location of peak soil strain moved down the piles over time.

Initially the angle of the failing wedge was approximately 60° to the horizontal, the angle of an active failure in soil with $\varnothing' = 30°$. As the failure progressed the angle of the failing wedge of soil to the horizontal reduced. Pessimistically, the analysis shown in Figure 2 was permitted to proceed until the inverted cone of failing soil reached its angle of repose. Subsequent analyses were refined to include choking of the sinkhole.

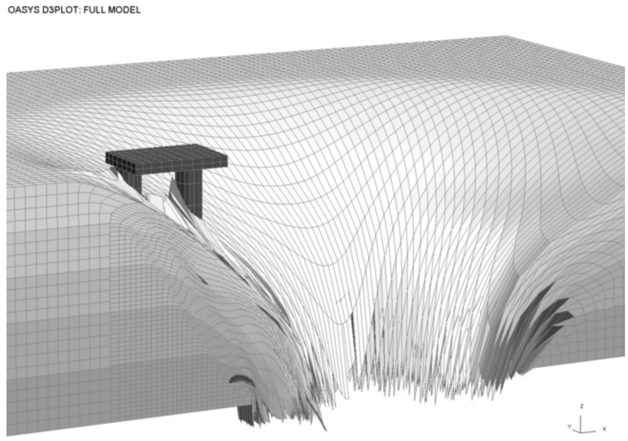

Figure 2. Deformation of soil around foundation as sinkhole collapses

4.2 *Lateral soil pressure on piles*

The pressure applied to each pile over the duration of the analysis was calculated and was resolved into radial (towards the centre of the sinkhole) and circumferential (around the sinkhole) directions. This pressure changed as the soil around the sinkhole failed, and it was calculated at several intervals through the run. Figure 3 shows the radial pressures on the piles and the line of horizontal soil pressure assuming twice the Rankine active ($2{*}K_a$) conditions for horizontal stress.

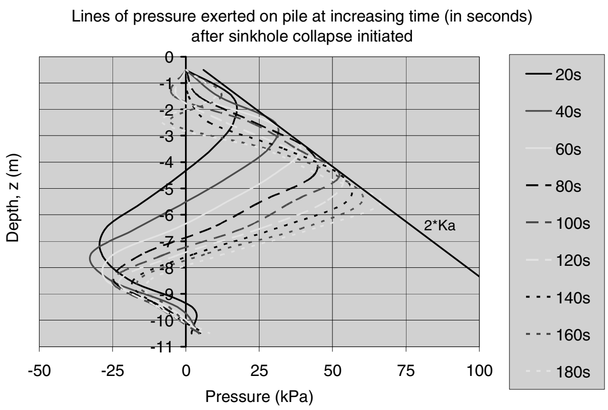

Figure 3. Radial soil pressure exerted on pile during sinkhole collapse

Figure 3 shows the following important features:

- The peak soil pressure on the piles was approximately $2K_a\sigma'_v$
- The peak soil pressure acted over a comparatively short length of pile
- As the soil failed into the sinkhole, the peak soil pressure acted over a lower portion of pile, but increases in magnitude (since the confining stress increases)
- The total lateral force on the foundation increased as the failure developed since the volume of soil acting in aggregate on both piles increased
- Where the soil was not exerting lateral force on the pile due to movement into the sinkhole, it still offered significant lateral resistance to support the foundation.

The most significant result of the analyses, therefore, was that the total lateral force on the piles was much lower than could have been predicted before the FE work was undertaken. The loading was much less onerous than any method that applied a lateral pressure calculated as a proportion of vertical stress, since the peak pressure acted over a relatively short amount of pile, and in particular it was much less than $3K_p$.

The results of these analyses are only applicable to foundations socketed into rock, or a competent non-sinkhole susceptible stratum. It is fundamental to the results that the capability of the foundation to carry vertical load was not compromised by the formation of the sinkhole.

5 EMPIRICAL MODEL

The 3D FE analyses were then used to determine a generalised design methodology presented only in terms of the geometry of the problem and the strength of soil. This methodology was then applied within an elastic beam and subgrade model as described below. This enabled a wide variety of pile group geometries to be analysed and superstructure movements be established, without recourse to time-consuming 3D FE continuum analyses for every situation.

5.1 *Elastic beam and subgrade model*

In an elastic beam and subgrade model, piles are divided into elastic elements connected to the ground by springs. Soil can be modelled as moving past the pile by removing the spring at a particular node and by applying an appropriate load instead.

For the problem under consideration,, soil moving plastically past the pile into the sinkhole loads the pile. Beneath the zone of moving soil, the pile is pushed into the soil which resists the loading. Plastic yield occurs in the soil for a number of pile diameters before the displacements reduce and soil deformations become elastic. Refinements in the spring model in the elastic region are unlikely to result in significant benefits so for this problem elastic-plastic springs were recommended based upon the work of Broms (1964, 1972), Poulos (1971) and Reese et al. (1974). General rules were developed for calculating spring parameters below the wedge of failing soil, noting that the vertical effective stress at the base of the sinkhole is very low and it is the soil in this location that provides the resistance to overturning of the pile.

5.2 *Determination of design lateral soil pressures*

The lateral soil pressures to be used within the beam and subgrade model in the region where the soil was flowing were determined from the results of the FE modelling.

Figure 4 presents the basic interpretation of the FE results, showing the fundamental geometry of the pressure distribution on the piles, as determined from results such as those presented in Figure 3. The 'wedge' angle, α, was defined as the angle measured from the bottom corner of the sinkhole to the bottom of where lateral soil pressure was applied to the pile at a point in time. This angle was initially approximately the active angle and it then reduced during the analysis. The 'pressure' angle, β, was defined as the angle measured from the bottom corner of the sinkhole to point on the pile where the peak lateral pressure was exerted at a point in time, z_f.

Both the 'wedge' angle and the 'force' angle decrease with time through the run. The relationship between these angles was determined empirically from the results of the nine parametric studies undertaken, but both were constrained to be greater than the angle of friction of the soil at all times

The force distributions on the piles shown in Figure 3 were approximated by two parabolic curves that join at depth z_f, where the lateral pressure was taken to be $2K_a$, from Figure 3.

They were defined such that at all times they encapsulated the pressure distributions found in the FE modelling.

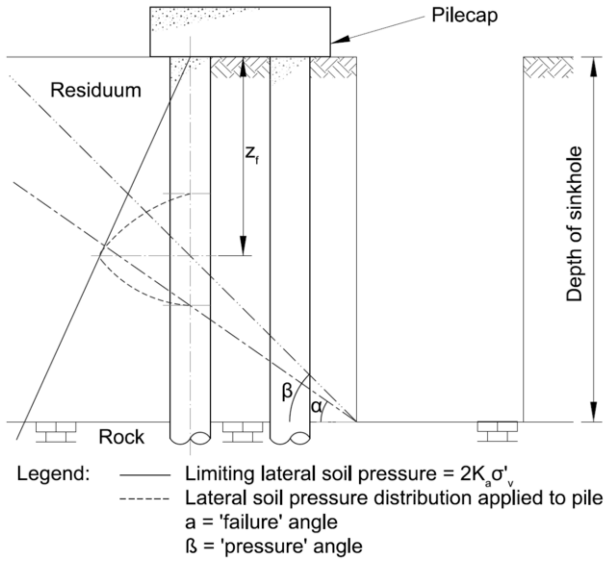

Figure 4. Basis of interpretation of results of finite element modelling – pressure distribution shown on back pile only

Figure 5 shows how these estimates of soil pressure compare against the observations from the FE results for various 'wedge' angles. The prediction slightly overestimates the pressure, particularly as the depth of sinkhole increases.

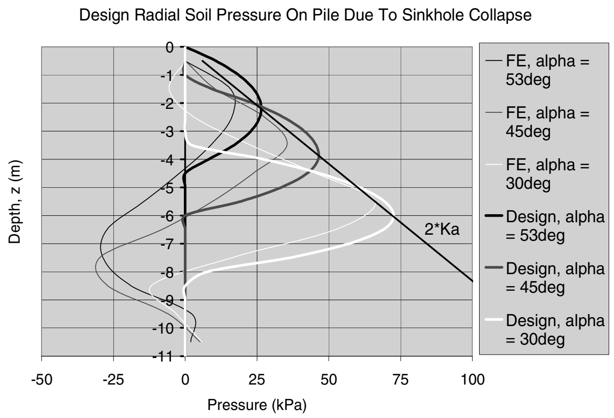

Figure 5. Comparison between finite element (FE) calculated and 'design' pressures on piles

In addition to the lateral forces discussed above, the soil also exerted significant downward force on the foundation as it flowed into the sinkhole. Unlike the lateral forces, they peaked early in the run and then reduced. This is because initially the 'wedge' angle was high and then it opened out, so initially the soil was principally moving down but then becomes progressively more horizontal.

The vertical soil force applied to a pile is a function of the vertical effective stress and the relative movement of soil around the pile. Thus the vertical force on the pile increased within the wedge initially, but then fell as the rate of movement decreased. Relatively simple, conservative rules for design were developed from the results of the various FE models undertaken, similarly to the lateral pressures.

5.3 *The use of design lateral soil pressures*

The design lateral soil pressures presented above were not perfect, but provided they were used by engineers who understood the modelling from which they were derived, they could be used to calculate the force on any pile for the conditions assumed as part of a beam-spring foundation model. The force on each node of such a model should be determined by multiplying the pressure by the height of the element by the cross-sectional area of the pile – this was found to be a reasonable approximation in this study.

The diameter of the sinkhole was not a relevant variable in the determination of lateral force pressures, but would have been had the hole been allowed to choke.

Sinkholes can occur at any location with respect to the foundation. The design lateral soil pressures presented above were used to model all locations of sinkhole in relation to the foundation to determine the most onerous design scenario.

6 CONCLUSIONS

This paper presents some results from a wider ranging FE study in to the effect of sinkholes forming adjacent to deep foundations. It has been demonstrated how this problem can be analysed using powerful FE techniques, from the formation of the sinkhole to the effect the collapsing sinkhole has on the foundations.

Appropriate parametric studies using carefully designed FE analyses were used to develop simplified design methods.

Peak lateral soil pressures on deep foundations resulting from sinkhole collapse were significantly lower than would be predicted if the soil-pile interaction was considered to be similar to that of a laterally loaded pile moving into 'undisturbed' soil. This is because of the change in lateral stresses within the soil caused by the development of the sinkhole, and because only a relatively small part of the pile was ever loaded.

With the growth of computing power, the analysis of 3D, non-linear, time-dependent problems such as these will become more commonplace. It is vital that the ability to analyse grows alongside the ability to understand the physical processes involved and to interpret the results of the analysis. This study demonstrated one way in which this can be done, and which resulted in significant savings in design from a relatively small number of FE analyses.

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