Numerical studies of soil–structure interaction using a simple interface model

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The effect of interface roughness in problems of soil–structure interaction is demonstrated using a simple finite element interface model. Three examples of geotechnical interest are presented to demonstrate the approach, and comparisons are made with closed-form solutions where available. Both rough and smooth extremes of interface behaviour are analysed. The smooth interface modelling is performed without the use of specialized elements, and involves uncoupling and rotation of freedoms parallel to the proposed interface direction. It is suggested that, in view of the uncertainties often associated with interface properties, a rational approach for engineering purposes is to obtain solutions for the perfectly rough and perfectly smooth cases leading to upper and lower bounds on the full range of interface behaviour.

Key words: finite elements, interfaces, soil–structure interaction, foundations, lateral loads, culverts, piles.

Au moyen d'un modèle simple d'interface en éléments finis, l'on démontre l'effet de la rugosité de l'interface dans les problèmes d'interaction sol–structure. Trois exemples d'application géotechnique sont présentés pour démontrer la procédure, et des comparaisons sont faites avec des solutions rapprochées lorsque disponibles. Les deux extrêmes de comportement d'interface rugueuse et lisse sont présentés. Le modèle d'interface lisse est réalisé sans faire appel à des éléments spéciaux, et implique le découplage et la rotation des libertés parallèles à la direction proposée de l'interface. Compte tenu des incertitudes souvent associées aux propriétés de l'interface, il est suggéré comme procédure rationnelle pour les finis de l'ingéniéur d'obtenir les solutions pour les cas de rugosité parfaite et de lissage parfait fournissant les limites inférieure et supérieure du domaine complet de comportement de l'interface.

Mots clés : éléments finis, interfaces, interaction sol–structure, fondations, charges latérales, ponceaux, pieux.

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1. Introduction

Numerical analysis of soil–structure interaction frequently includes interface effects, which must be adequately modelled. These effects are characterized by concentrations of shear displacements once certain shear stress levels are reached, and can be described by a range of constitutive assumptions.

Most problems of soil–structure interaction involve compressive contact stresses at the interface, and in such cases, relatively simple numerical models may be adopted, as will be shown. For certain problems, it may be possible to model interface behaviour by simply refining a conventional finite element mesh in the vicinity of the interface. The elements near the interface would then be given suitable properties and a nonlinear analysis performed, possibly using plasticity theory. This approach suffers from the disadvantages that the mesh must remain continuous and that occasional numerical difficulties occur when adjacent elements are assigned greatly different strength properties. For more advanced applications however, especially in the area of jointed rock masses, specialized interface elements should be used (Goodman *et al.* 1968). For example, in general applications where the compressive ambient stress referred to above cannot always be relied upon, the solution processes should allow for the possibility of separation and rebonding at the interface (e.g., Ghaboussi *et al.* 1973). In the area of soil–structure interaction involving anchors, an extensive study of the effects of tensile separation and different interface assumptions has been made by Rowe and Davis (1982).

The present note is confined to problems in which no separation can occur and it is proposed that in the first instance only the perfectly rough and perfectly smooth cases should be considered. Once the full range of behaviour has been covered, it can then be decided whether the use of more specialized interface elements is justified.

2. Method of simple interface modelling

Conventional finite element analysis presents a natural way of modelling *rough* conditions provided the mesh is sufficiently refined. In an analysis such as this, *failure* or *slip* would occur not at the interface itself but at the nearest stress point in the weaker of the two materials sandwiching the interface.

Perfectly smooth conditions can be simulated by allowing unrestricted movement in the slip direction. A smooth rigid footing at the surface of a soil layer, for example, is easily modelled by prescribing vertical displacements at the interface, but placing no resistriction on the horizontal movements (e.g., Griffiths 1982). In general, however, a smooth interface orientated at a certain angle to the horizontal may be required. Situations such as this are dealt with using a two-step approach.

The first step involves the introduction of an extra freedom at nodes that lie on the interface. The second step requires that the freedom directions at nodes that lie on the interface be transformed such that they lie parallel and perpendicular to the proposed interface direction. These modifications result in three freedoms per node along the interface, as shown in Fig. 1. The freedoms parallel to the interface are uncoupled on each side, but the freedom in the normal direction is common to both sides.

For cases where the required smooth interface direction does not lie in the Cartesian x- or y-directions, a transformation involving freedom rotation must be performed. This is a technique well known to structural analysts for dealing with *skew* boundary conditions.

The remainder of this paper describes application of this method to three boundary value problems of geotechnical interest. The particular problems are chosen because the interface properties in each have a significant influence on the computed response. Where possible, comparisons are made with available closed-form solutions.

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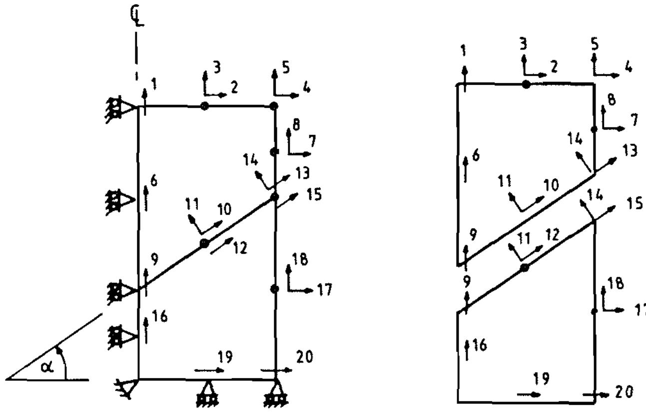

FIG. 1. Freedom numbering for inclined interface.

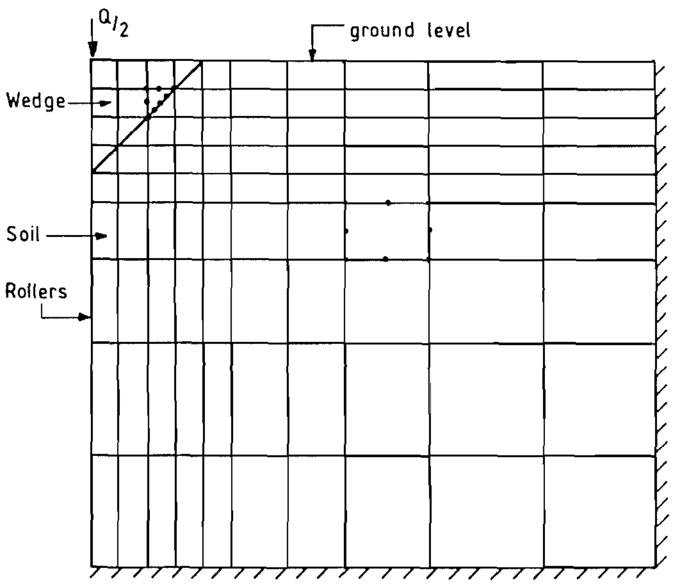

FIG. 2. Mesh for smooth wedge analysis.

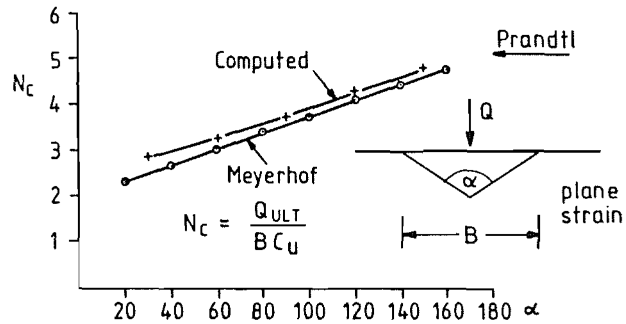

FIG. 3. Bearing capacity for a smooth wedge.

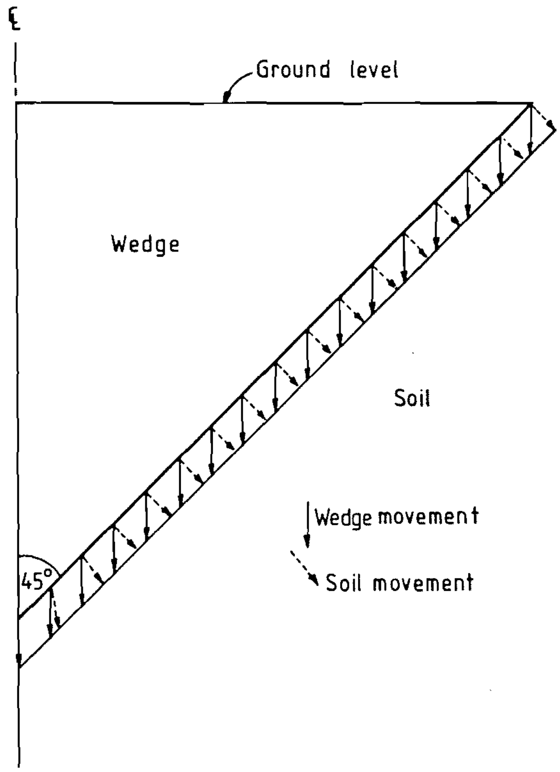

FIG. 4. Relative movements between soil and smooth wedge.

3. A smooth wedge pushed into a cohesive soil

The mesh shown in Fig. 2 was used to analyse the behaviour of a plane strain wedge pushed into a layer of cohesive soil. The soil was assumed to behave as an elastic-perfectly plastic material obeying a Tresca failure criterion. The wedge itself was given a relatively high strength and stiffness. The program data and mesh geometry were arranged so that the apex angle of the wedge could be easily varied.

Along the smooth interface between the wedge and soil, the freedoms on each side were uncoupled and their directions rotated to be parallel and perpendicular to the interface direction. A viscoplastic algorithm was used with iterations to achieve convergence to the failure criterion. In order to use the conventional finite element strain–displacement relationships, the element displacements at the interface nodes had to be transformed back into Cartesian directions before computing the strains. After calculation of stresses and strains at the Gauss points, any body forces in Cartesian directions that were generated at the interface nodes during the stress redistribution process had to be transformed into the relevent directions before solution of the equilibrium equations.

Vertical prescribed displacements were applied to the surface of the wedge and the reactions back-figured from the resulting converged stresses. The soil resistance as it was mobilized was nondimensionalized in the form of a bearing capacity factor N_c defined:

$$[1] \quad N_c = \frac{Q}{Bc_u}$$

where Q = axial force on wedge (or lateral force on disc), B = full width of wedge (or disc diameter), and c_u = undrained shear strength of soil.

In Fig. 3, the computed failure values for a range of apex angles are compared with those due to Meyerhof (1961) and are seen to be in close agreement.

Figure 4 shows how the displacement vectors of the wedge and soil differ along the interface, owing to the additional freedom. The wedge moves vertically downwards, whereas the soil experiences little movement parallel to the interface direction. Further results for this particular problem in the axisymmetric case have been presented by Lane (1986).

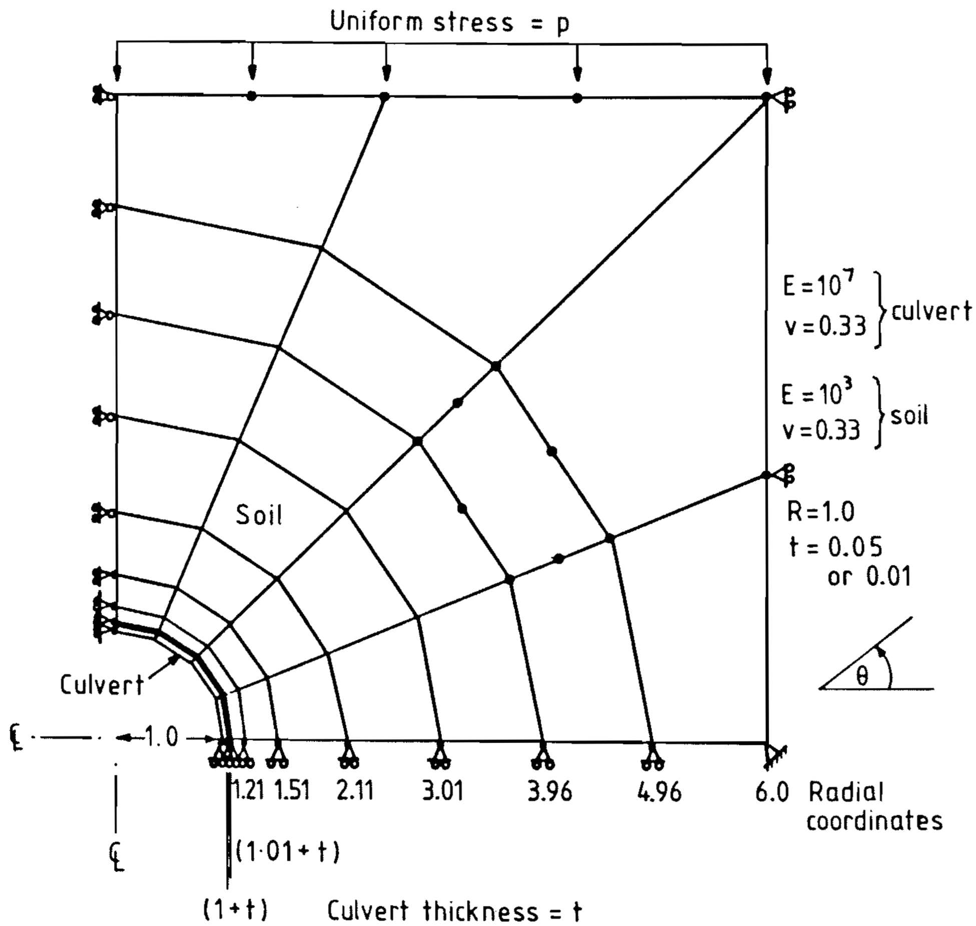

FIG. 5. Mesh for culvert analysis.

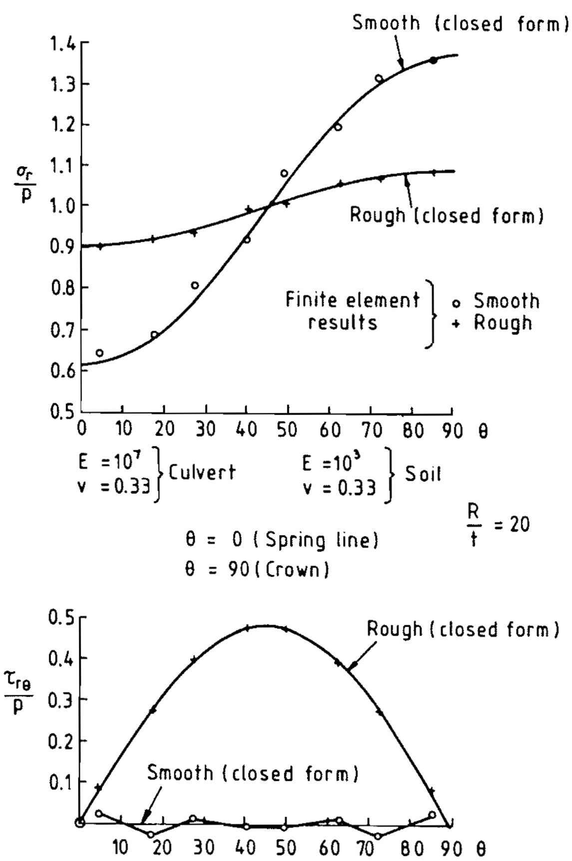

FIG. 6. Computed and closed-form solutions for a "thick" culvert.

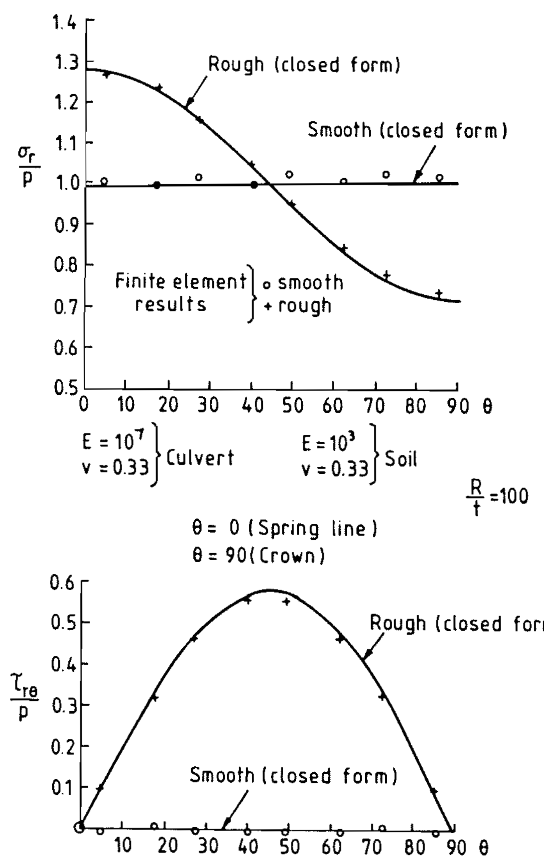

FIG. 7. Computed and closed-form solutions for a "thin" culvert.

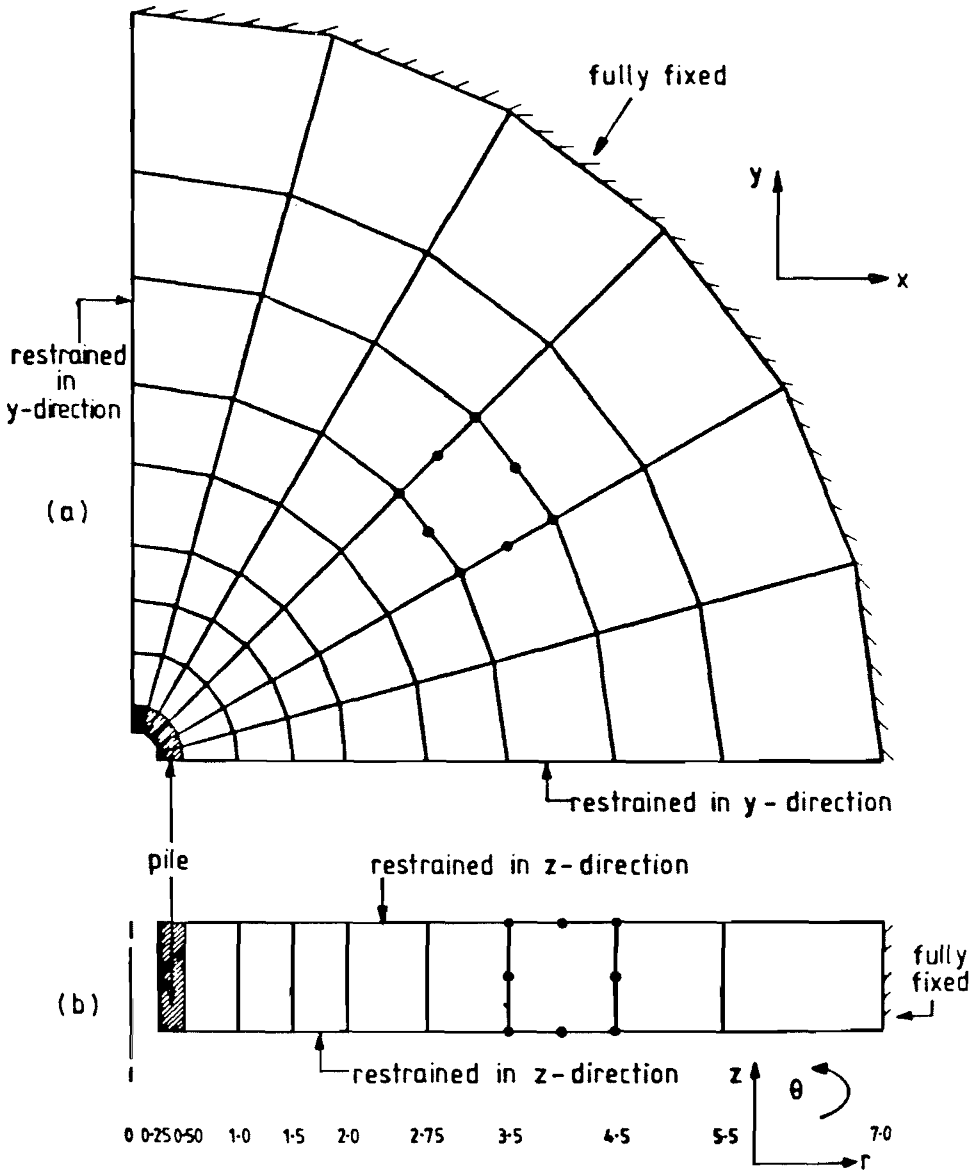

FIG. 8. Meshes used for (*a*) plane strain and (*b*) nonaxisymmetric strain analyses.

4. Stresses around buried culverts

Interest in the analysis of this type of structure has grown in recent years with the realization that it represents a form of low-cost tunneling (e.g., Duncan 1979; Katona 1983). One of the main design problems that must be faced is to obtain the minimum depth of cover that can safely support given loading conditions at the ground surface.

Both rough and smooth interface conditions between the culvert and soil can be modelled numerically using the methods described. For comparison, a closed-form solution giving the stresses exerted by the soil on the culvert is available (Burns and Richard 1964). The solution is limited to the case of a circular culvert embedded at great depth within an elastic soil, and subjected to a uniform applied stress at ground level.

The mesh used for this analysis is given in Fig. 5 and was constructed using eight-noded quadrilateral elements throughout, with reduced integration to form the stiffness matrix. It may be noted that, owing to symmetry, only one-quarter of the problem was analysed and the surface loads were applied at a distance 6*R* (Leonards *et al.* 1982) above the crown, where *R* was the radius of the culvert.

The mesh was designed on the basis of circular arcs of elements concentric with the culvert itself. Although the mesh shown in Fig. 5 is drawn with straight-sided elements, it was found that more stable stresses were computed when curved elements were used with the mid-side nodes also placed on the circular arcs.

One ring of elements was used to model the culvert, and stresses were sampled at the eight Gauss points in the adjacent thin ring of soil elements. Because of the form of the analytical solution, the Cartesian stresses were converted into polar coordinates using the usual transformation for rotation angle θ:

$$[2] \quad \begin{aligned} \sigma_r &= \sigma_x \cos^2\theta + \sigma_y \sin^2\theta + \tau_{xy} \sin 2\theta \\ \sigma_\theta &= \sigma_y \cos^2\theta + \sigma_x \sin^2\theta - \tau_{xy} \sin 2\theta \\ \tau_{r\theta} &= \frac{1}{2}(\sigma_y - \sigma_x)\sin 2\theta + \tau_{xy}\cos 2\theta \end{aligned}$$

The computed results are compared with the closed-form solutions in Figs. 6 and 7 for the cases R/t equal to 20 and 100 respectively, where t was the thickness of the culvert. The finite element results confirm that the method accurately repro-

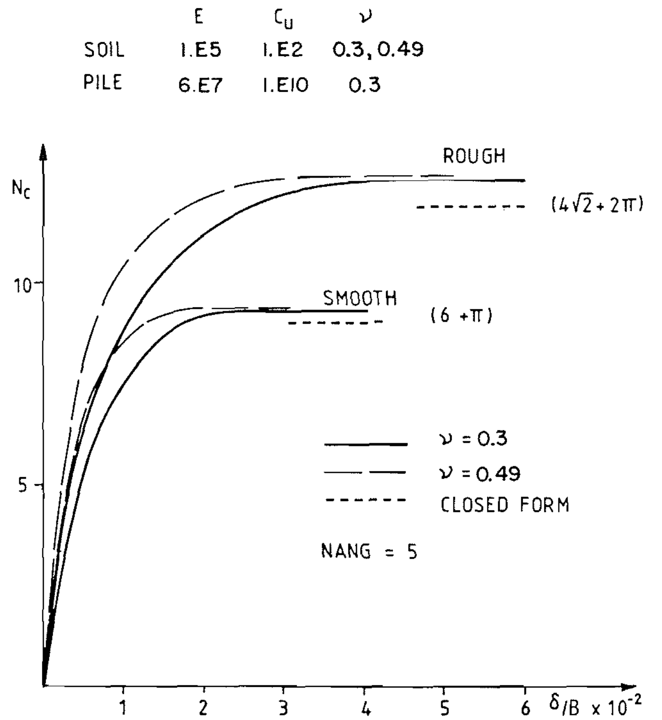

FIG. 9. Load–displacement response for rough and smooth cases.

duces the different behaviour exhibited by the rough and smooth cases (Mokrani 1986). Of particular interest is the change in the distribution of radial and shear stresses around the culvert as its relative rigidity is varied.

5. Laterally loaded disc in cohesive soil (two-dimensional)

The final example considered in this paper involves the behaviour of a laterally loaded disc in a cohesive soil. Such a configuration approximates the situation occurring within a laterally loaded circular pile at sufficient depth below ground level such that plane strain conditions apply. Clearly, such a two-dimensional analysis could by tackled using the plane strain mesh given in Fig. 8*a*. In this case however, a *nonaxisymmetric* strain analysis has been used employing the mesh in Fig. 8*b*. This latter approach has applications for problems involving nonaxisymmetric loading of axisymmetric bodies, but is also used in this case because it produces a particularly simple conversion from rough to smooth conditions at the soil–pile interface. This is because the nodal freedoms include a tangential component, which enables unrestricted slip to be modelled without any transformations.

Computed results obtained by Griffiths and Lane (1986) are compared with analytical solutions of Broms (1964) for the smooth case, and of Randolph and Houlsby (1984) for the rough case, in Fig. 9 and are in good agreement. The lateral displacement of the disc has been nondimensionalized by the diameter B, and the factor N_c is again defined by [1]. It may be noted that the ultimate load in the rough case is only about 30% higher than that in the smooth case, suggesting that the use of specialized interface elements in this instance would be unnecessary for engineering purposes.

6. Conclusions

A simple method has been described for finite element modelling of interface effects in problems of soil–structure interaction. It is proposed that the perfectly smooth and perfectly rough cases should be analysed in the first instance to see whether the use of specialized interface elements is justified. The method of implementation of smooth conditions involved the introduction of an extra freedom at nodes along the interface. This was followed by a transformation of the freedom directions such that they were oriented parallel and perpendicular to the proposed interface direction. Three examples of geotechnical interest were presented to demonstrate the method, and the numerical results obtained compared favourably with available closed-form solutions. It was noted that in the third example involving a laterally loaded pile, no transformations were necessary to achieve smooth conditions because components of the freedoms in the *nonaxisymmetric* analysis were already oriented in the tangential directions.

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