GEOCRISP — A Version of CRISP93 for Geosynthetic-Soil Applications

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1. Introduction

The concept of soil reinforcement is an ancient one. For thousands of years, nonsoil materials have been used for the purpose of improving the engineering properties of soils. The modern form of earth reinforcement (called "reinforced earth") was first introduced by Vidal in the late 1960s (Jones, 1985). The use of earth reinforcement has increased exponentially ever since. Steel bars and metal strips used in earlier applications of reinforced soil have been replaced by modem geosynthetics such as polyester woven geotextiles or polypropylene geogrids. In addition to the reinforcement applications, geosynthetics are also being increasingly used in drainage, filtration and separation applications. Now-a-days, prefabricated wick drains are usually installed instead of conventional sand drains for ground improvement and a geosynthetic filter is mostly used in between a concrete retaining wall and the backfill instead of a graded gravel filter.

The mechanical behaviour of a geosynthetic differs significantly from other conventional materials such as steel or aluminium. It should be modelled with reasonable accuracy if successful design of geosynthetic-soil structures is to be achieved. Also, the interaction between a geosynthetic and the adjacent soil should also be taken into account in the design method. Presently, there are no simple design methods available which can take into consideration both the mechanical behaviour of the geosynthetic and its interaction with the adjacent soil. The limitations of one of the most widely used design methods — the Limit Equilibrium Method (LEM) — in the design of reinforced soil structures is discussed by Sharma (1994).

The above-mentioned requirements can easily be fulfilled by the Finite Element Method (FEM) which has several other advantages such as the ability to model construction or excavation sequence and difficult geometry. Not so long ago, its use as a design tool was considered prohibitively expensive. However, with the continuing advent of faster and cheaper personal computers or workstations now-a-days, it is gaining popularity amongst geotechnical design engineers. In geotechnical engineering, one of the widely used finite element program is the CRISP (CRItical State Program) (Britto & Guru-t, 1987). Its usefulness as a design tool can be seen from numerous examples of its use in the analysis and design of various soil-structures (Mair et al., 1981; Seneviratne & Gunn, 1985; Bolton et al., 1989; Sanchez & Sagseta, 1990; Hird & Pyrah, 1991; Woods & Clayton, 1992; Bolton et al., 1993; Maheetharan and Lau, 1993). However, the present version of CRISP (called CRISP93) is not particularly well-suited to geosynthetic-soil structures. This report describes several modifications to CRISP93 carried out by the author in order to make it more apropos to geosynthetic-soil applications. The new version is christened GEOCRISP. A 1-D drainage element similar to the one described by Hird et al. (1992) was added. The formulation, implementation and benchmark testing of this element is described in section 2. The formulation of 1-D bar element was also modified so that it could model a geosynthetic reinforcement. These modifications are described in section 3. In addition, the formulation of Critical State soil models (Cam-clay, Modified Cam-clay and Schofield) was also altered. These alterations are outlined in section 4. This report is intended to supplement the user's manuals (Vols. 1 & 3) for CRISP93 (Britto & Gunn, 1990) insofar as the mput specifications to the Geometric Program and the Main Program are concerned- These input specifications are presented in section 5. Section 6 describes the modifications done to the output of CRISP93. Post-processing features of GEOCRISP are outlined in section 7.

2. The Drainage Element

The use of vertical drains in soft clay prior to the construction of an embankment has gained widespread acceptance since the invention of **prefabricated band drains by Kjellman (1948) and** their subsequent improvement (van Zanten, 1986). The vertical drains contribute towards the stability of the embankment by hastening the process of consolidation and can be used in combination with other ground improvement techniques such as installation a geotextile or a geogrid reinforcement at the base of the embankment. Due to the uncertainty linked with the determination of permeability parameters, the design of vertical drains is usually carried out using simple analytical methods (e.g. Barron, 1948; Hansbo, 1981). Several simplifying assumptions are usually involved in these methods, e.g. a uniform column of soil with linear compressibility characteristics and the absence of lateral movements. The FEM is capable of a more rigorous analysis of vertical drains. If needed, it can also model the effects of reinforcement and staged construction of the embankment. However, the difficulty in using the FEM for vertical drain analysis is that most FEM analyses are carried out in plane strain conditions whereas the consolidation of soil around an individual vertical drain is essentially axisymmetric. A logical basis for carrying out a plane strain FE analysis using a matching procedure for the axisymmetric and plane strain analyses was first described by Hird et al. (1992). They also described the incorporation and validation of a 1-D drainage element in CRISP84 (older version of CRISP93). The obvious advantage of such an element is that each vertical drain can be individually represented in the finite element mesh without introducing any additional degrees of freedom. The author has attempted to incorporate and validate this drainage element in CRISP93. In this section, the formulation and benchmark testing of the drainage element is described. A modified matching procedure for plane strain analysis is also presented. In near future, the drainage element is intended to be used in the back-analysis of a centrifuge test on reinforced embankments on soft clay installed with wick drains (Sharma, 1994).

2.1 The Formulation

The formulation of the drainage element is described in detail by Russell (1990). Hence, only a brief description is provided here. The drainage element is a three-noded line element (Figure 1) and is designed to be compatible with both the linear strain triangular (LST) and linear strain quadrilateral (LSQ) elements used to represent the adjacent soil. The variation of both strain and pore **pressure along** the drainage element is linear.

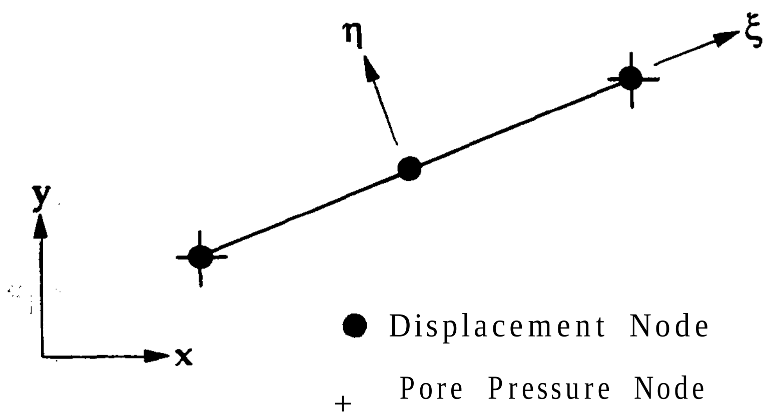


Figure 1 The Drainage Element

Using a fully implicit approximation over time, the discretized forms of the equilibrium and continuity equations (Biot, 1941) are, respectively,

$$K\Delta d + L\Delta u = \Delta F_1 \qquad (1)$$

$$L^T\Delta d - \Delta t\Phi\Delta u = \Delta F_2 \qquad (2)$$

where ΔF_1, ΔF_2, Ad, AU and At are increments of nodal force, nodal flow, nodal displacement, nodal pore pressure and time respectively; K is a material stiffness matrix; L is a link (or coupling) matrix and Φ is a permeability matrix. The element stiffness matrix K_E for displacement and pore pressure variables is

$$K_E = \begin{bmatrix} K & L \\ L^T & -\Delta t\Phi \end{bmatrix} \qquad (3)$$

For a prefabricated linear drain, the relationships between axial force and displacement and between axial flow rate and hydraulic gradient are modelled by the matrices K and Φ respectively. In the present formulation, both these relationships are assumed to be linear. From the consolidation point of view, it is assumed that the drain has a negligible cross-sectional area. Hence, no volume change of the drain results from a change of displacement or no change in force in the drain results from a change of pore pressure. Consequently, the link matrices L and L^T are composed entirely of zero terms.

The reader may wonder at this stage about the possibility of using standard LST or LSQ elements instead of the new drainage element to model the vertical drains. In some situations (e.g. large diameter sand drains), it may be feasible but usually it is likely that the resulting LST or LSQ elements would have unacceptably high aspect ratios. Furthermore, as mentioned above, the 1-D drainage elements have the advantage of modelling the drain without introducing any additional degrees of freedom into the mesh.

2.2 Benchmark Testing

It is mandatory that the incorporation of a new element or a new model in a finite element program be validated against some kind of analytical (closed form) solutions. In this section, the validation of the drainage element against the closed form solution for radial consolidation given by Hansbo (1981) is described. The validation procedure is similar to the one adopted by Hird et al. (1992) for the validation of their drainage element.

Closed form solutions for the consolidation of the soil using vertical drains usually involve the study of an axisymmetric unit cell (i.e. a cylinder of soil around a single drain) under simplified boundary conditions. Figure 2 shows a unit cell of fixed external radius R and initial length 1 containing a vertical drain of radius r_w. The impervious bottom boundary is not permitted to move vertically while the pervious top boundary may be permitted to displace freely under a constant stress (free strain) or may be constrained to displace uniformly (equal strain). In the present study, free strain condition is assumed because it is mathematically more convenient than the equal strain condition. The compression of soil is assumed to take place according to a linear stress-strain law. Drainage within the soil may take place in both the vertical and the radial directions. However, it is often reasonable to assume that the vertical flow is negligible. The permeability of the soil k may reduce to a lower value k_s within a smear zone of radius r_s which is a result of drain installation. Flow in the drain is vertical and towards the top and is governed by the permeability of the drain k_w.

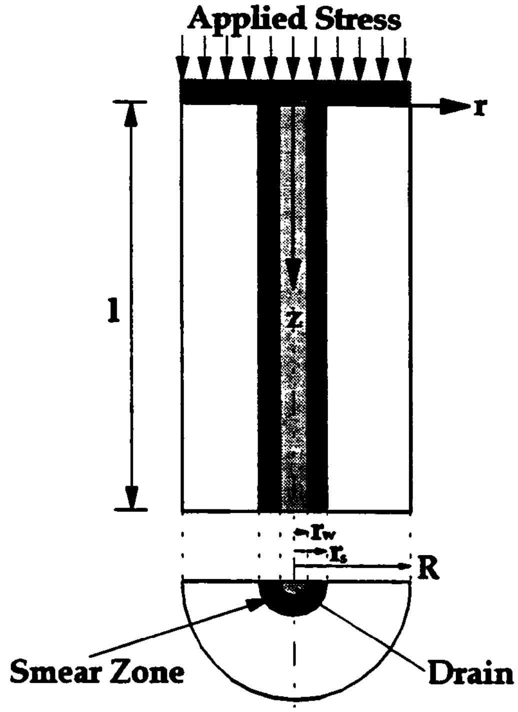


Figure 2 Unit cell adopted for the closed form solution

The available theories vary in their degrees of rigour and complexity. However, following the arguments given by Hird et al. (1992) in its favour, the relatively simple and easy to apply theory proposed by Hansbo (1981) is used in the validation procedure described here. Hansbo's theory allows for both the well resistance and the smear but assumes that there is no vertical flow in the unit cell. Under an instantaneous step loading at the surface, the average degree of consolidation U_h on a horizontal plane at depth z and time t is predicted by Hansbo to be

$$U_h = 1 - \exp(-8T_h / \mu) \tag{4}$$

where T_h , the time factor for radial drainage, and μ are defined as

$$T_h = \frac{C_h t}{4R^2} \tag{5}$$

$$\mu = \ln\left(\frac{n}{s}\right) + \left(\frac{k}{k_s}\right)\ln(s) - \frac{3}{4} + \frac{z(2l - z)^2 k}{k_w r_w^2} \tag{6}$$

In the expression for μ, n and s are defined as

$$n = \frac{R}{r_w} \quad \text{and} \quad s = \frac{r_s}{r_w}$$

Several axisymmetric FE analyses were carried out using GEOCRISP to obtain results for comparison with Hansbo's theory. The mesh and the boundary conditions for these analyses are shown in Figure 3(a).

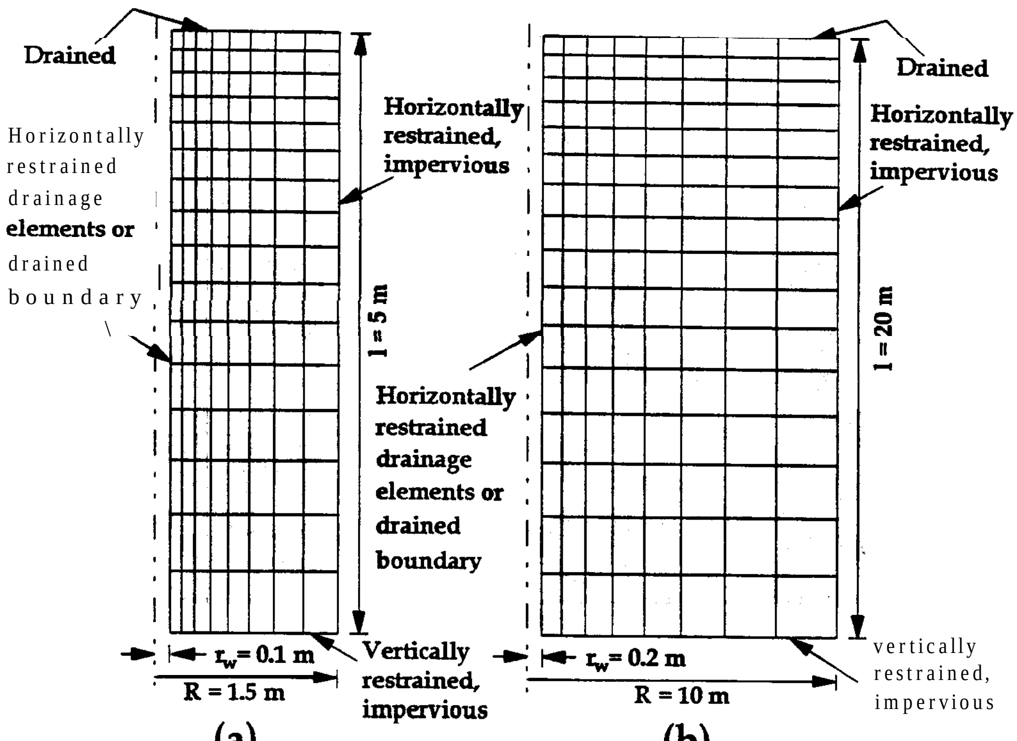


Figure 3 Finite element meshes and boundary conditions for unit cell analyses

Stresses had to be applied to the top boundary under the condition of free strain instead of equal strain. It has been shown by Hird et al. (1992) that the resulting boundary displacements can be expected to be practically uniform. Hansbo's theory incorporates linear stress-strain behaviour of soil by using a constant constrained modulus (E_o) in the calculation of C_h. Therefore, in the FE analyses linear elastic properties were specified with E' $= 10^4$ kPa and $\nu' =$ 0. The neglect of vertical flow in Hansbo's theory was matched in the FE analyses by setting the vertical permeability of the soil to zero. The horizontal permeability was taken to be 10^{-8} m/s. The drain was assumed to possess negligible stiffness (E = 10 kPa) but its finite permeability was varied to obtain different values of a dimensionless parameter L proposed by Yoshikuni and Nakanodo (1974). L is defined as

$$L = \frac{8kl^2}{\pi^2 k_w r_w^2} \tag{7}$$

At any particular stage of the analysis, the average degree of consolidation at the base of the unit cell could be obtained by performing a simple numerical integration using trapezoidal rule. The results are compared with those calculated using Hansbo's theory in Figure 4. It can be seen that the agreement is generally excellent except in the case of no well resistance (L = 0) at low time factors.

Following Hird et al. (1992), two more FE analyses were then carried out with the aim of emulating the parametric studies reported by Jamiolkowski et al. (1983). In the first analysis, the effect of well resistance in the absence of smear was once again explored but using a different mesh shown in Figure 3 (b). The sole purpose of using this mesh was to make the geometry of the unit cell consistent with that adopted in the studies by Jamiolkowski et al. (1983). Assumptions regarding the boundary conditions and material properties remained unchanged. In the second analysis, the effect of wick drain length was investigated for fixed drain permeability and spacing. The variation of the drain length was achieved by stretching or shrinking the mesh of Figure 3(b) in the vertical direction so that R/l varied between 0.2 and 1. The results of these two sets of analyses are presented along with the predictions from Hansbo's theory in Figure 5. Once again, the agreement between the FE and theoretical results is excellent.

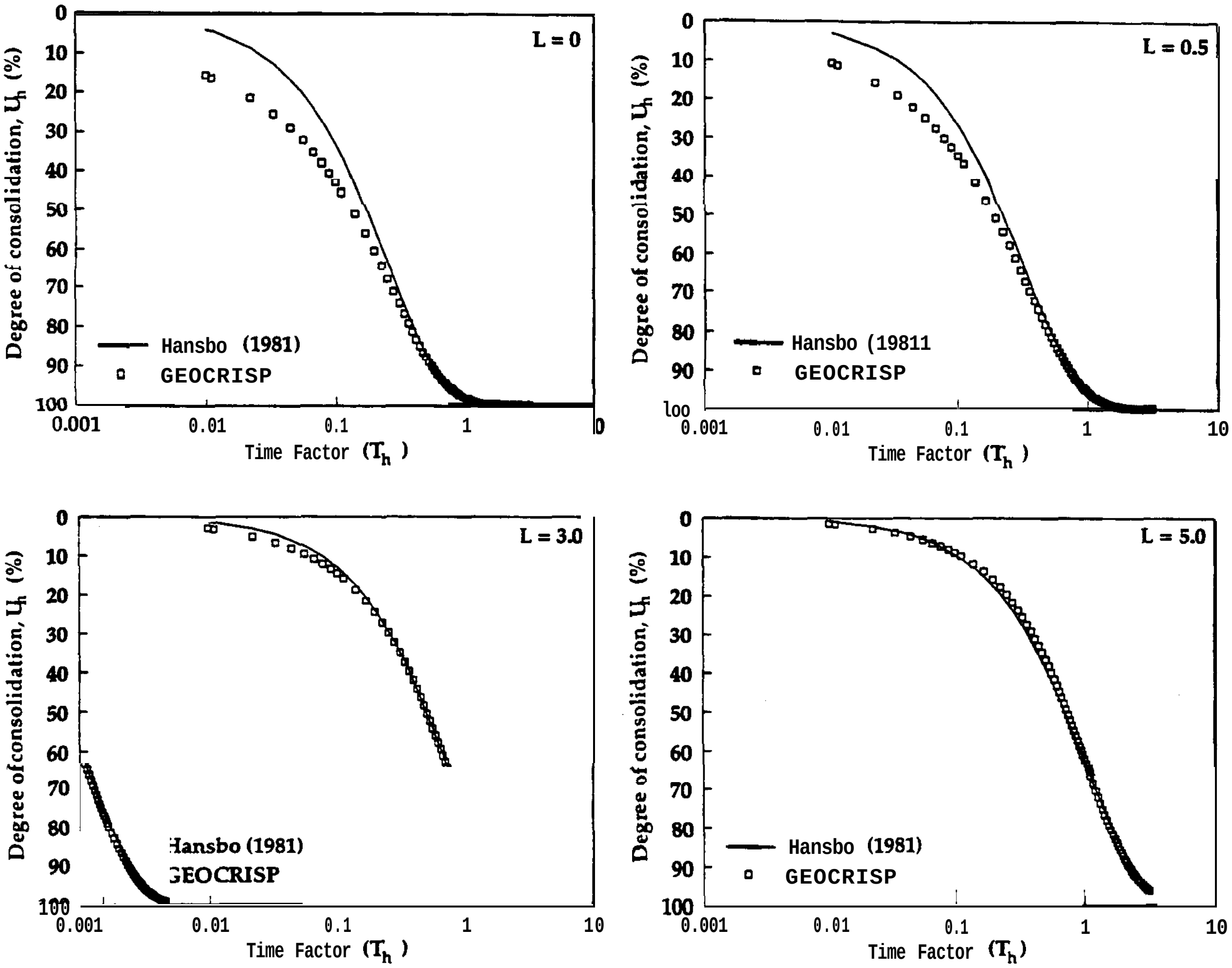
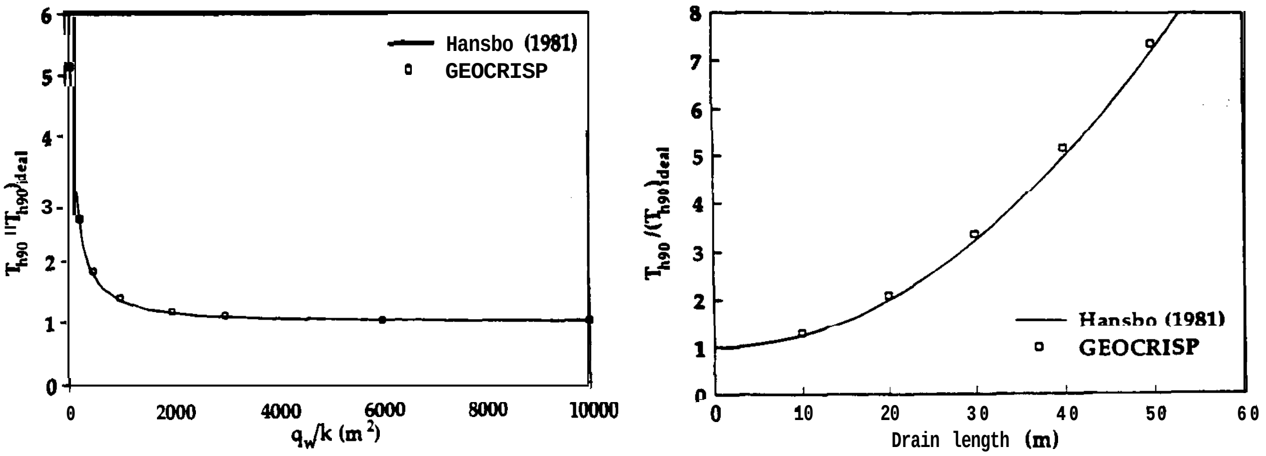


Figure 4 Comparison of finite element and analytical results for consolidation of a unit cell



(Effect of well resistance) (Effect of drain length)

Figure 5 Effect of well resistance and drain length on rate of consolidation

The trends shown in Figure 5 are in close agreement with those illustrated by Jamiolkowski et al. (1983). This is true despite a subtle difference of approach in the latter case. Jamiolkowski et al. (1983) established the time factor for 90% average degree of consolidation by evaluating the average degree of consolidation not just at the mid-depth but over the entire length of the drain.

2.3 *Matching procedure for plane strain analysis*

Prior to the matching procedure suggested by Hird et al. (1992), modelling the effect of wick drains in plane strain analyses involved matching the time taken for a given degree of consolidation to be achieved by horizontal drainage under plane strain and axisymmetric conditions. Thus, neglecting the effect of well resistance,

$$\frac{T_{hpl}B^2}{k_{pl}} = \frac{T_{hax}R^2}{k_{ax}} \qquad (8)$$

where 2B is the dram spacing in plane strain and the subscripts or subscript extensions pl and ax stand for the respective conditions. Either or both the drain spacing and the soil permeability may be manipulated to satisfy equation (8). The catch in using equation (8) is that the resulting values of spacing and permeability do not apply for other than the chosen degree of consolidation. This is due to the fact that the ratio T_{hpl}/T_{hax} is not a constant but varies with degree of consolidation. Moreover, for finite well resistance, the degree of consolidation varies with depth. In view of the generally good agreement between the FE analyses and Hansbo's theory, Hird et al. (1992) proposed a better matching procedure by adapting Hansbo's theory for plane strain. They considered a plane strain unit cell of half width B containing a drain with a discharge capacity of Q_w per unit length in the direction perpendicular to the plane under consideration. The unit cell is shown in Figure 6. For matching purposes, the drain is assumed to possess negligible thickness and there is no smear zone.

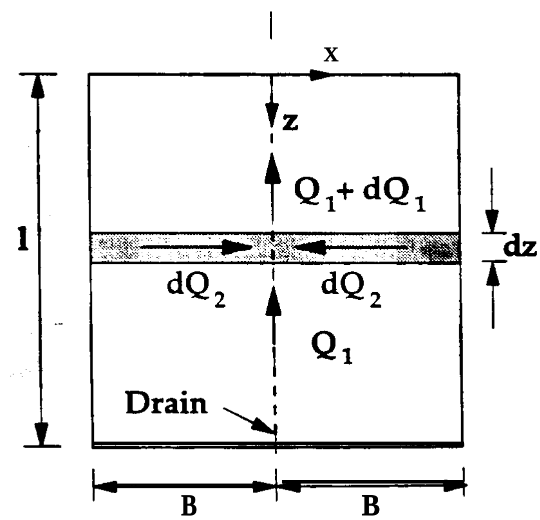


Figure 6 Plane strain unit cell

Based on the derivation presented in Russell (1990), Hird et al. (1992) proposed the following expression for average degree of consolidation of a plane strain unit cell.

$$U_h = 1 - \exp(-8T_h/\mu) \qquad (9)$$

where T_h and μ are defined as

$$T_h = \frac{C_h t}{4B^2} \tag{10}$$

$$\mu = \left[\frac{2}{3} + \frac{2k}{BQ_w}(2lz - z^2)\right] \tag{11}$$

Equation (9) is exactly the same as equation (4) defining degree of consolidation for the axisymmetric unit cell. For the rate of consolidation in a plane strain and an axisymmetric unit cell to be matched, it is necessary that the average degree of consolidations at every tune and every level in the cell are equal. Hence,

$$U_{hpl} = U_{hax} \tag{12}$$

From equations (4) and (9) it follows that

$$\frac{T_{hpl}}{\mu_{hpl}} = \frac{T_{hax}}{\mu_{hax}} \tag{13}$$

or

$$\frac{C_{hpl}}{B^2 \mu_{pl}} = \frac{C_{hax}}{R^2 \mu_{ax}} \tag{14}$$

If the soil parameters are identical in each case,

$$B^2 \mu_{pl} = R^2 \mu_{ax} \tag{15}$$

The relevant expressions for μ_{pl} and μ_{ax} may be substituted in equation (15) and the terms rearranged to give

$$\frac{2}{3}B^2 - R^2\left[\ln\left(\frac{n}{s}\right) + \left(\frac{k}{k_s}\right)\ln(s) - \frac{3}{4}\right] = \left[\left(\frac{\pi R^2 k}{q_w}\right) - \left(\frac{2Bk}{Q_w}\right)\right](2lz - z^2) \tag{16}$$

The condition for geometric matching may be obtained by considering the case of negligible well resistance (q_w and $Q_w \to \infty$). Therefore, geometric matching, including the effect of smear is achieved if

$$B = R\sqrt{\tfrac{3}{2}\left[\ln(n/s) + (k/k_s)\ln(s) - \tfrac{3}{4}\right]} \tag{17}$$

The effect of well resistance is matched independently if

$$\left(\pi R^2 k/q_w\right) - (2Bk/Q_w) = 0 \tag{18}$$

or

$$Q_w = (2B/\pi R^2)q_w \tag{19}$$

The conditions represented by equations (17) and (19) assume that the permeability of the soil is the same in the plane strain and the axisymmetric case. Alternatively, matching can be achieved if the permeability is changed instead of the dram spacing. When k_{pl} differs from k_{ax} , equation (15) can be replaced by

$$\left(k_{pl}/B^2\mu_{pl}\right)=\left(k_{ax}/R^2\mu_{ax}\right) \tag{20}$$

Putting B=R and following the same logic as before, the permeability and well resistance matching requirements are

$$k_{pl} = \frac{2k_{ax}}{3\left[\ln(n/s)+(k_{ax}/k_s)\ln(s)-\frac{3}{4}\right]} \tag{21}$$

$$Q_w = (2/\pi R)q_w \tag{22}$$

In order to validate the matching procedure proposed by Hird et al. (1992), FE analyses were carried out for both an axisymmetric unit cell and an equivalent plane strain unit cell, determined using equations (17) & (19) or equations (21) & (22). For axisymmetric analysis, the mesh of Figure 3(b) was used. The mesh and the boundary conditions for the plane strain analyses is shown in Figure 7. This mesh was either shrunk or stretched in the horizontal direction to achieve geometric matching. The average degree of consolidation at the base of the mesh was obtained by numerical integration as described above. The time factor (T_h) was calculated using equations (5) & (10) for the axisymmetric and the plane strain FE analyses respectively .

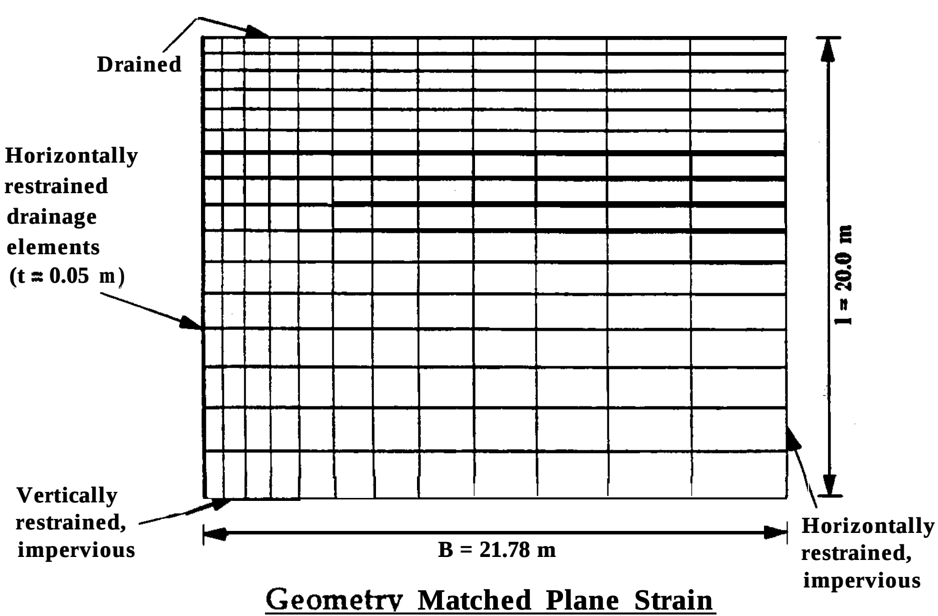


Figure 7 Finite element mesh and boundary conditions for plane strain unit cell analysis

Figure 8 shows the comparison between the axisymmetric and the plane strain FE results in terms of average degree of consolidation. Figure 8 that the geometry and permeability matched plane strain analyses produced almost identical results. Also, the comparison between the axisymmetric and the plane strain FE analyses was generally good except at low time factors. The difference in the degree of consolidation at a given time factor doesn't exceed 5%. The reader should note that the ratio (T_h/μ) is represented on the x-axis rather than the time factor (T_h). It is the ratio (T_h/μ) that is the same at a given degree of consolidation for axisymmetric and matched plane strain unit cells and not the time factor (T_h). A similar comparison presented by Hird et al. (Fig. 9 of the paper) is misleading as it shows a good comparison between the degree of consolidation by plotting it against the time factor (T_h).

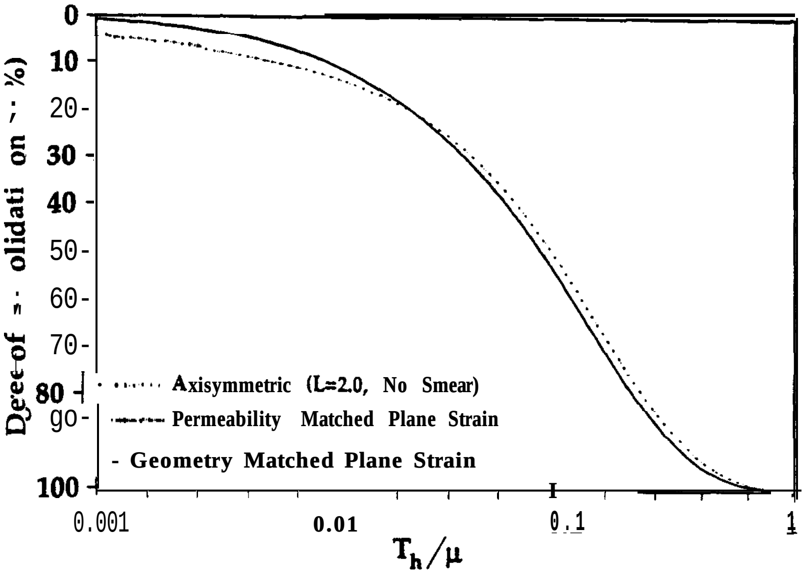


Figure 8 Comparison of average degree of consolidation from axisymmetric and plane strain FE analyses

The reader should note that even if perfect matching is achieved between the axisymmetric and equivalent plane strain unit cell in terms of average degree of consolidation, the excess pore pressures at corresponding points in the unit cell will not be the same. This point can be appreciated by looking at Figure 9 which shows the distribution of excess pore pressures for axisymmetric as well as plane strain FE analyses at 65% average degree of consolidation. The values of excess pore pressures on the periphery of the unit cell (midway between the drains for plane strain unit cell), differ significantly.

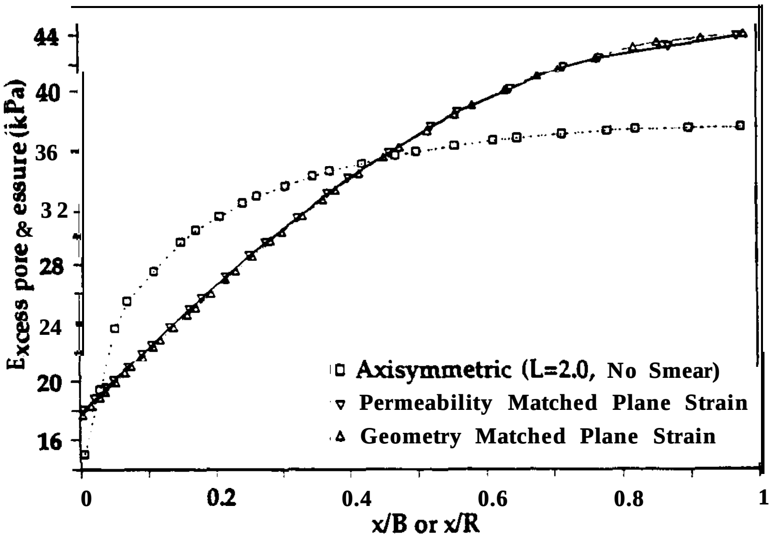


Figure 9 Lateral variation of pore pressure in axisymmetric and plane strain FE analyses (U_h=65%)

3. The Reinforcement Element

For accurate analysis of reinforced soil, it is imperative to correctly represent the reinforcing material in the mesh. This can be done using linear strain bar (LSB) element available in CRISP93. The use of bar elements has added advantage of not introducing any additional degrees of freedom into the mesh. However, the formulation of bar element in CRISP93 is linear elastic whereas the stress-strain curve for modem geosynthetic reinforcements is non-linear. Also, geosynthetic reinforcements are incapable of taking any compressive load and they lack flexural rigidity. Although the latter condition is satisfied by the bar element in CRISP93, it can take significant compressive load provided that the mid-side node is supported. With the objective of eliminating these limitations, it was decided to modify the formulation of the bar element. The bar element in GEOCRISP is now called the reinforcement element. It is formulated as a bilinear elastic material which lacks flexural rigidity and is incapable of taking any compressive load. The constitutive relationship for the reinforcement element is shown in Figure 10.

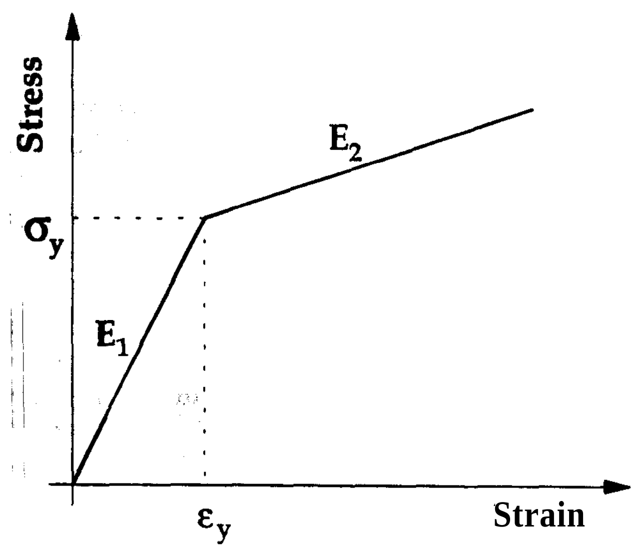


Figure 10 Bilinear elastic constitutive model for the reinforcement element

In addition to the usual input for standard bar element, a value of threshold strain (ε_y) is required. Also, in order to make the reinforcement element extremely weak in compression, a small value of Young's modulus in compression is necessary. It should be noted that the constitutivemodel for the reinforcement element is not elastic-plastic, i.e. it will not produce any residual plastic strains if unloading occurs beyond the threshold strain but it will follow the same stress-strain path during unloading. The reader should also note that the axial stiffness of a geosynthetic reinforcement is usually expressed as force per unit width per unit strain (kN/m). This is commonly referred to as the reinforcement modulus (J) and is equal to the Young's modulus (E) multiplied by the thickness (t) for plane strain conditions. The input specifications for the reinforcement element are described in section 5.

Unlike the drainage element, no benchmark testing was carried out for the reinforcement element. However, it is appropriate to mention that this element was used extensively and reasonably successfully by Sharma (1994) for the FE analyses of reinforced embankments on soft clay.

4. Other Modifications

4.1 Variation of permeability with void ratio

When using Hansbo's (1981) theory to assess the rate of consolidation of a soil, it is assumed that the co-efficient of consolidation C_h is constant. In reality, however, it initially varies with depth and also changes with time as the consolidation progresses under loading. It is defined as

$$C_h = \frac{k}{m_v \gamma_w} \tag{23}$$

where k is the permeability, γ_w is the unit weight of water and m_v is the co-efficient of volume change for one-dimensional consolidation and is defined as

$$m_v = \frac{\lambda}{\sigma'_v(1+e)} \tag{24}$$

The co-efficient of volume change depends on the void ratio as well as the effective vertical stress. Therefore, its value decreases with depth. This implies that the co-efficient of consolidation is significantly smaller at the top of a clay layer as compared to that at the bottom. To certain extent, this effect is nullified by the observation that the permeability decreases with decrease in the void ratio (Tavenas et al., 1983; Al-Tabbaa, 1987).

The decrease in the co-efficient of volume change with depth is automatically modelled in the finite element analyses carried out using CRISP93 when the soil is modelled by elasto-plastic models based on Critical State Soil Mechanics (CSSM), namely Cam-clay, Modified Cam-clay and Schofield. However, the permeability is assumed to be constant. As a result, the upper layers of the finite element mesh consolidate more quickly than those lower down. In general, it is mandatory to accurately model the co-efficient of consolidation throughout the depth of the deposits when carrying out finite element analyses of soil-structure interaction problems involving soft clay. To achieve this using ClUSP93, it is necessary to divide the deposit into several layers and define a different value of permeability in each layer. This procedure can become cumbersome if the subsoil profile is complicated. Therefore, it was decided to change the formulation of CSSM models to introduce a variation of permeability with void ratio.

Al-Tabbaa (1987) has suggested the following empirical relationship between the logarithm of permeability, k and the logarithm of void ratio, e

$$\log(k) = m\log(e) + \log(k_o) \tag{25}$$

This equation is graphically illustrated in Figure 11. In the above equation, k_o is the permeability of soil at void ratio equal to 1. Equation (25) has been implemented in GEOCRISP for all the CSSM models. The variation of horizontal and vertical permeability with void ratio can be modelled independently. The user is asked to input the slope m and the permeability k_o at $e=1$ for both the horizontal and the vertical permeability. The input specifications for this feature are described in section 5. The plot shown in Figure 11 can be obtained for any soil by carrying out several oedometer/permeability tests on intact specimens. The exact experimental procedure can be found either in Tavenas et al. (1983) or in Al-Tabbaa (1987). It is worthwhile to note that for CSSM models, GEOCIUSP calculates the initial void ratios from the in-situ stresses input by the users and then assigns appropriate permeability values at different depths according to equation (25).

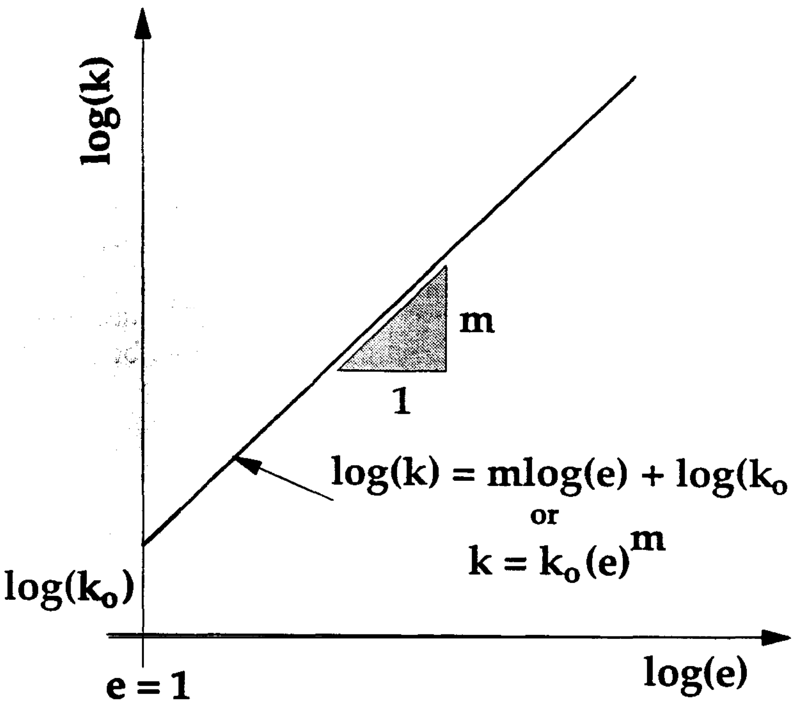


Figure 11 Variation of permeability with void ratio

4.2 Variation of shear modulus with mean effective pressure

The formulation of the CSSM models in GEOCFUSP was modified further so that the shear modulus G could be specified proportional to the mean effective stress p', the constant of proportionality to be given by the user, i.e. G = Np'. Prior to this modification, the user could either specify a constant shear modulus or a value for the Poisson's ratio v for the soil which was used to calculate the shear modulus using the following equation:

$$G = \frac{3(1-2\nu)Vp'}{2(1+\nu)\kappa} \qquad (26)$$

It has been observed by several research workers (e.g. Al-Tabba, 1987) that the constant of proportionality calculated by equation (26) can be significantly different from that obtained experimentally. Now, with the above-mentioned modification, the user can input the experimentally measured value of the constant of proportionality.

The tendency for G to be dependent on p' is a phenomenon observed in large number of soils (Wroth et al., 1979). However, specifying G to be proportional to p' in the elastic region results in a non-conservative model for the elastic behaviour (Zytynski et al., 1978) which may give rise to numerical inaccuracies when cyclic loading is involved.

5. Modifications to Standard CRISP93 input

5.1 Mesh Generation

5.1.1 Drainage elements

As is **the case** for bar, beam and slip **elements,** the Super Mesh Program (SMP) does not incorporate drainage elements. However, all is not lost. The user can generate the mesh without drainage elements using the SMP and view it using mesh plotting program noting down the nodes where he/she wants to put drainage elements. He/she can then edit the *.GPR file generated by the SMP to incorporate the drainage elements. While editing the *.GPR file, ITYP in record J (using the notations used in Vol. 1 of CRISP93 user's manual) should be specified as 21 for drainage element. Only the two end nodes should be specified. The record J should look like the one shown below for drainage element:

KEL	ITYP	IMAT	N1	N2	N3	N4
341	21	3	291	292	0	0

where nodes 291 and 292 are the end nodes of the drainage element no. 341. Since new elements are added, NVTX and NEL in record C of the *.GPR file should also be updated. If there is a gap in **the** numbering of the elements, NUMAX and MUMAX in record D of the *.GPR file should be updated as well. The edited *.GPR file is **then given** as input to the Front Squasher Program (SQ) which produces the data file (*.GPD) for the Geometry Program.

The drainage element is meant for 2-D meshes only. The possibility of its use next to curved elements has not been tried out but it is unlikely to work in combination with curved elements.

5.12 Reinforcement elements

The reinforcement element should be treated as a bar element (ITYP=1) when specifying its geometry in **the** *.GPR file. As with drainage element, only the end nodes should be specified. The record J for the reinforcement element should look like the one shown below:

KEL	ITYP	IMAT	N1	N2	N3	N4
119	1	9	96	110	0	0

As in case of drainage elements, records C and D should also be modified accordingly.

The reinforcement element is meant for 2-D meshes only. The possibility of its use next to curved elements has not been tried but it is unlikely to work in combination with curved elements.

5.2 Main Program Input

5.2.1 Record D

When using the drainage or reinforcement elements, the correct material type should be specified. The material type numbers and the material properties are outlined below:

	Drainage	Reinforcement
NTY	8	8
P(1)	E	E_1
P(2)	t	ν
P(3)	k	A
P(4)	γ_w	E_c
P(5)	0	E_2
P(6)	0	Et
P(7)	0	0
P(8)	0	0
P(9)	0	0
P(10)	0	0
P(11)	0	0
P(12)	0	0

where

- E -Young's modulus for the drainage element
- t — thickness (plane strain) or radius (axisymmetric) of the drainage element
- k -permeability along the length of the drainage element
- γ_w -unit weight of water
- E_1 — Young's modulus in tension for the reinforcement element (axial strain ≤ Et)
- E_2 — Young's modulus in tension for the reinforcement element (axial strain > Et)
- E_c — Young's modulus in compression for the reinforcement element
- ν — Poisson's ratio for the reinforcement element
- Et -Threshold strain (Figure 10)
- A — Area per unit width (thickness) of the reinforcement element

In addition, if the variation of permeability with void ratio and the variation of G with p' is to be modelled, record D for CSSM models (NTY= 3,4, or 6) should be as follows:

NTY	P(5)	P(6)	P(7)	P(8)	P(9)	P(10)	P(11)	P(12)	P(13)	P(14)
3	G or ν'	N	0,K_w,γ_w	γ_{bulk}	k_{xo} or 0	k_{yo} or 0	m_x	m_y	-	-
4	G or ν'	N	0,K_w,γ_w	γ_{bulk}	k_{xo} or 0	k_{yo} or 0	m_x	m_y	-	-
6	G or ν'	N	0,K_w,γ_w	γ_{bulk}	k_{xo} or 0	k_{yo} or 0	H	S	m_x	m_y

where

- N — constant of proportionality between G and p'. P(5) should be zero if N is nonzero.
- k_{xo} -horizontal permeability at e=1
- k_{yo} -vertical permeability at e=1

m_x — slope of permeability-void ratio line (Figure 11) for horizontal permeability
m_y — slope of permeability-void ratio line for vertical permeability

Explanation for other parameters can be found in CRISP93 user's manual (Vol. 1). If the variation of permeability with void ratio is not required, m_x and m_y are set to zero and the constant values of horizontal and vertical permeabilities are input at locations P(9) and P(10) respectively. If the user wants GEOCRISP to compute the variation between G and p', he/she should set N to zero and specify Poissons's ratio at location P(5). However, if a constant G is sought, the value of N should be set to zero and the constant value of G should be specified at location P(5).

6. Modifications to Standard CRISP93 output

The output produced by GEOCRISP is virtually the same as that produced by CRISP93 except that a new output file is created which contains the stress, strain, pore pressure and hydraulic gradient for all drainage elements present in the mesh. This output file has a *.DEO extension. The creation of *.DEO file is controlled by the IOUT variable in record I of the *.MPD file. The IOUT variable is specified as a five digit code, e.g. 00120. It is the fourth digit from left that controls the creation of *.DEO file. If the fourth digit is zero, the *.DEO file is empty. If the fourth digit is 2, the *.DEO file contains the above-mentioned parameters for all the drainage elements (at all integration points). The amount of output in the *.DEO file is controlled in exactly the same way as that for the *.MPO file. The user is advised to refer to pages 20 and 21 of CRISP93 user's manual (Vol. 1).

CRISP93 by default produces a *.MAS file which is used by the Analysis Assessment Program. The information for each element for each increment in an increment block is output to this file. As a result, the *.MAS file swells in size and may exceed the *.NRS file if CSSM models are used. It was thought that with the availability of CRISP93-Lotus 123 interface, the user can derive and assess most of the parameters for elements of his/her choice. Hence, it was decided to suppress the creation of *.MAS file in GEOCRISP.

6.1.1 Post-processing with GEOCRISP

Unfortunately, due to the inclusion of drainage element, CRISP93's own post-processor does not function with the *.NRS file created by GEOCRISP. Modifications to the post-processing program was considered too complicated and hence it was not attempted. However, the user can make use of the interface between GEOCRISP and the FEMVIEW finite element post-processing software. This interface requires a data file (*.FVD) which is of the form shown below:

Record	Variables	Explanation
A	NAME	Model name to be used in FEMVIEW (max. 6 characters)
B	CONS	Y if it is a consolidation analysis else N
C	CAMC	Y if any of the CSSM models is used else N
D	OUTD	Always input TAPE
E	TINC	Total no. of increments in the *.NRS file
F	NINC	No. of increments to be processed
G	INCN(NINC)	List of the increments to be processed

An example of the *.FVD file is given below:

```
RESC8M
N
Y
TAPE
770
3
350 700 770
```

The CRISP-Femview program can be run by typing FV95GEO at the DOS prompt after loading the DBOS run-time system. At the end of its successful run, the program outputs a *.FVI file which contains results for the increments specified in record G of the *.FVD file. The *.FVI file can then be read by the FEMVIEW post-processing package. The user is advised to refer to the FEMVIEW user's manual (Femsys Co., 1993) for further details.

7. Conclusions

The CRISP93 finite element program has been modified in order to make it more suitable for geosynthetic-soil applications. The modified version is called GEOCIUSP. A 1-D drainage element and a 1-D bar element is incorporated into GEOCRISP. The validation of the drainage element was undertaken according to the procedure described by Hird et al. (1992). From the results of the validation procedure, it can be concluded that the drainage element has been successfully incorporated into GEOCRISP. A procedure for matching the average degrees of consolidation for an axisyrnmetric and a plane strain unit cell suggested by Hird et al. (1992) was also tested using GEOCRISP and was found to be satisfactory. Using this matching procedure, it is possible to carry out plane strain FE analysis of soil-structure interaction problems involving soft clay foundation installed with wick drains. In addition, the formulation of the CSSM models was also modified to include the variation of permeability with void ratio and the variation of shear modulus with mean effective pressure. Although no benchmark testing of the reinforcement element and the modifications to CSSM models was undertaken, these features were successfully used by Sharma (1994) in the finite element analyses of reinforced embankments on soft clay.

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