1-1-2012

Radial consolidation modelling incorporating downdrag effect for a multi-layer soil

Cholachat Rujikiatkamjorn
University of Wollongong, cholacha@uow.edu.au

Buddhima Indraratna
University of Wollongong, indra@uow.edu.au

This article has been published under teh title “Radial consolidation modelling incorporating downdrag effect for a multi-layer soil”

RADIAL CONSOLIDATION MODELLING INCORPORATING THE EFFECT OF SMEAR ZONE FOR A MULTI-LAYER SOIL WITH DOWNDRAG OF CAUSED BY MANDREL ACTION

Cholachat Rujikiatkamjorn

BEng (Hons), MEng (AIT), PhD (Wollongong)

Lecturer, School of Civil Engineering, Faculty of Engineering,
University of Wollongong, Wollongong City, NSW 2522, Australia
E-mail: cholacha@uow.edu.au

Buddhima Indraratna

PhD, DIC, MSc (Lond.), FIEAust., FASCE

Professor of Civil Engineering, Faculty of Engineering,
University of Wollongong, Wollongong City, NSW 2522, Australia
Tel: +61-2-4221-3046; Fax: +61-2-4221-3238
E-mail: indra@uow.edu.au

Submitted to: Canadian Geotechnical Journal 09-220

RADIAL CONSOLIDATION MODELLING INCORPORATING EFFECT OF THE SMEAR ZONE FOR A MULTI-LAYER SOIL WITH DOWNDRAG OF SOIL CAUSED BY MANDREL ACTION

Cholachat Rujikiatkamjorn and Buddhima Indraratna

Abstract

A system of prefabricated vertical drains with surcharge preloading is an effective method for promoting radial drainage and accelerated soil consolidation. A piecewise technique is employed to analyse the radial consolidation in the multi-layer soil system to include (a) the effect of soil downdrag and (b) smear zone having soil permeability varying linearly. The effect of soil dragged down from the upper soil layer into the lower layer has been analysed in terms of required time for consolidation. It can be seen that the consolidation of the multi layer soil depends on smear zone charactersitics, permeability ratio between upper and lower soil layers, penetration depth, and drain spacing. Design procedures are described with a worked out example.

Key words: Consolidation, Design charts, Multi-layer soil, Smear zone, Vertical drains.

Introduction

Due to rapid coastal development in Australia, most infrastructure is forced to be built on marine or estuarine soft clays that are highly compressible and of low shear strength, unless substantially consolidated prior to infrastructure construction. The coastal areas contain thick layers of compressible clay which is highly compressible and low permeable. Ground improvement technique is required before the commencement of the permanent structures. Preloading of soft clay with prefabricated vertical drains (PVDs) is one of the most common methods for improving the shear strength of soil and to reduce its post-construction settlement. Using PVDs, the drainage path length is reduced significantly from the soil layer thickness to half of the drain spacing. Moreover, the horizontal hydraulic conductivity is usually larger than the vertical hydraulic conductivity, hence, the consolidation process can occur much faster (Jamiolkowski et al. 1983). This system has been used successfully to improve foundation soils for airports, rail and road embankments (Indraratna and Redana 2000; Li and Rowe 2002).

PVDs are normally installed using steel mandrel pushing through soft ground creating smear zone surrounding PVDs. The soil in the smear zone is disturbed with an associated permeability reduction, which is detrimental to radial consolidation. The extent of smearing depends on the mandrel size, installation procedure and soil type (Eriksson et al., 1999; Lo, 1998; Bo et al. 2003). Highly sensitivity clays with prominent macro fabric generally exhibit the greatest smear effects. A number of studies have noted that the disturbance in the smear zone increases towards the drain (Chai and Miura, 1999; Hawlader et al., 2002; Sharma and Xiao, 2000; Indraratna and Redana, 2000; Madhav et al., 1993; Bergado et al., 1991). Laboratory studies on circular sand drains and rectangular PVD illustrate a linear decrease in horizontal permeability towards the drain. The permeability close to the drain can be decreased by one order of magnitude (Bo et al., 2003) and is usually assumed to be the same as the vertical permeability (Hansbo, 1981; Indraratna and Redana, 2000). The vertical permeability remains relatively unchanged. In layered soil, the disturbance of the lower layer is not only due to the displacement of soil surrounding PVDs but also the down-dragged disturbed soil in the upper layer forming an additional smear zone (Casagrande and Poulos, 1969; Hird and Moseley, 2000).

Barron (1948) introduced the radial consolidation theory including the effects of smear (for equal strain) and well resistance (for equal and free strain). These solutions involved Bessel functions that were time consuming to perform. Consolidation solutions for radial drainage were further developed including: rigorous solutions for vertical and horizontal drainage for equal strain with well resistance (Yoshikuni and Nakanodo, 1974, Onoue et al., 1988). Hansbo (1981) proposed a much simpler solution incorporating both smear effect and well resistance into Barron's formulation. Indraratna et al (2005a, b) incorporated the effect of vacuum pressure and non-linear soil compressibility and permeability for radial consolidation. In the above solutions, a reduced permeability is simply assumed as a constant. Subsequently, Walker and Indraratna (2007) and Basu and Prezzi (2007) employed a linear decay in horizontal permeability towards the drain representing a more realistic variation of soil permeability within the smear zone.

A number of solutions exist for layered soils of which Zhu and Yin (2005) presented design charts for vertical drainage with two layers. Xie et al. (1999) solved the same problem with partially drained boundaries, while Xie et al. (2002) incorporated the small strain theory and nonlinear soil properties where the decrease in permeability is assumed proportional to the decrease in compressibility (i.e. coefficient of consolidation is constant). Double layered ground with radial and vertical drainage was studied by Tang and Onitsuka (2001) and Wang and Jiao (2004). However, none of them considered the effect of soil dragged down from the upper layer to create additional smear in the underlying layer.

A piecewise-linear technique is a method that divides the soil properties into small segments based on material coordinates so that the soil properties can be considered as a constant for a particular segment. This method offers a more general solution compared to the exact solutions that can be applicable for any particular conditions (Walker 2006). Fox et al. (2003) has applied this method for radial consolidation accounting for vertical strain and flow, soil self-weight, and variable hydraulic conductivity and compressibility during the consolidation process.

In this study, a radial consolidation model using a piecewise technique will be proposed to study the smear effect in layered soil. The effects of permeability of the penetrating upper soil layer on the underlying soil layer are discussed in terms of the degree of consolidation. The intrusion of the upper soil layer into the underlying soil layer creates an additional zone where the remoulded permeability of this zone can be increased or decreased depending on the initial permeability of the upper soil layer. In the intrusion zone located in the lower soil layer, the permeability variation can be divided into 3 zones including (a) smear zone due to the dragdown of the upper soil layer, (b) smear zone due to the remoulding of the underlying soil, and (c) the undisturbed lower soil layer. So far there is no technique to take this effect into consideration. The existing theory can only be used for the unit cell with a smear zone and the undisturbed zone. The piecewise-linear technique has been considered as an appropriate approach to determine the effect of soil dragdown on the overall consolidation. An array of design charts with a worked-out example is illustrating the role of down-dragged soil.

Theoretical Development

The main assumptions for radial consolidation considering equal vertical strain condition are assumed to be applicable to a unit cell (Hansbo 1981), and they are:

(1) Laminar flow through the saturated and homogeneous soil (Darcy's law) is valid. Flow is not allowed to occur at the outer boundary (Fig. 1), and for relatively long vertical drains (drain length is usually more than 10 times the drain spacing), only the lateral flow is permitted to occur (i.e. no vertical flow) (Rujikiatkamjorn and Indraratna, 2007).

(2) Soil deformation is uniform at the upper boundary of the unit cell and the small strain theory is valid.

(3) Based on the equal strain concept (Barron 1948, Indraratna et al. 2005a, Walker and Indraratna 2007) that all vertical strains at any given depth z are assumed to be equal, as the radial consolidation prevails.

To enable the use of index notation the inner radius r_w and outer radius r_e have been replaced with r_0 and r_m respectively. Figure 1 shows a unit cell with an external radius r_e, drain radius r_w, and an initial drainage path length l. Any radial distribution of lateral permeability (k_h) can be approximated by dividing the soil into m segments as in Fig. 2. k_h is constant within each segment.

Considering flow in segment ith (Fig. 1), the pore pressure gradient in the i^{th} segment is given by:

[1] $$\frac{\partial u_i}{\partial r} = \frac{\gamma_w r_m^2}{2\bar{k}_h}\frac{\partial \varepsilon}{\partial t}\kappa_i\left[\frac{1}{r} - \frac{r}{n^2 r_0^2}\right]$$

where,

[2] $$\kappa_i = \frac{\bar{k}_h}{k_{hi}}$$

[3] $$n = \frac{r_m}{r_0}$$

In the above, ε = average vertical strain at depth z, c_h = horizontal coefficient of consolidation u = excess pore pressure

The permeability ratio, κ_i, is calculated with respect to a convenient reference value, $\bar{k}_h$ (usually that of the undisturbed soil).

Using the boundary conditions $u(r_0) = w$ and $u_i(r_i) = u_{i+1}(r_i)$, Equation (1) is integrated in the r direction for each segment to give the pore pressure in the i^{th} segment as:

[4] $$u_i = -\frac{r_m^2}{2c_h}\frac{\partial \bar{u}}{\partial t}\left[\kappa_i\left(\ln\left(\frac{r}{r_{i-1}}\right) - \frac{1}{2}\left(\frac{r^2}{n^2 r_0^2} - \frac{s_{i-1}^2}{n^2}\right) + \psi_i\right) + \frac{k_{hi}}{q_w}\pi z(2l - z)\left(1 - \frac{1}{n^2}\right)\right]$$

where,

[5] $$s_i = \frac{r_i}{r_0}$$

and, [6] $$\psi_i = \frac{1}{\kappa_i}\sum_{j=1}^{i-1}\kappa_j\left[\ln\left(\frac{s_j}{s_{j-1}}\right) - \frac{1}{2}\left(\frac{s_j^2}{n^2} - \frac{s_{j-1}^2}{n^2}\right)\right]$$

The pore water pressure in the drain at depth z is designated as $w(z)$. For vertical flow in the drain, the change in flow from the entrance to the exit face of the slice with thickness dz (Fig. 1) is given by:

[7] $$dQ_1 = -\frac{\pi r_w^2 k_w}{\gamma_w}\left(\frac{\partial^2 w(z)}{\partial z^2}\right) dzdt$$

where, k_w = drain permeability.

The radial flow into the slice is then determined from:

[8] $$dQ_2 = \pi\left(r_e^2 - r_w^2\right)\frac{\partial \varepsilon}{\partial t} dzdt$$

Assuming no sudden drop in pore pressure at the drain-soil boundary (that is, $u = w$ at $r = r_w$), then for continuity,

[9] $$dQ_1 = dQ_2$$

Substituting Equations (7) and (8) into Equation (9) and rearranging gives:

[10] $$\frac{\partial^2 w(z)}{\partial z^2} = \frac{\gamma_w}{q_w}\pi\left(r_e^2 - r_w^2\right)\frac{\partial \varepsilon}{\partial t}$$

where, q_w is the discharge capacity of the drain given by:

[11] $$q_w = k_w \pi r_w^2$$

Integrating Equation (10) in the z direction with the boundary conditions $w(0) = 0$ and $w(2l) = 0$, reveals the pore water pressure in the drain to be:

[12] $$w(z) = \frac{r_e^2 \gamma_w}{2q_w}\pi z(2l - z)\left(1 - \frac{1}{n^2}\right)\frac{\partial \varepsilon}{\partial t}$$

The average pore water pressure $\overline{u}$, and the pore pressure distribution with radius are related by the algebraic expression:

[13] $$\pi\left(r_e^2 - r_w^2\right)\overline{u} = \int_{r_w}^{r_e} 2\pi \sum_{i=1}^{m}\left(\frac{r_i u_i}{m}\right) dr$$

Performing the integration in Equation (13), the resulting expression for $\bar{u}$ can be rearranged in the following form:

$$\bar{u} = \frac{\gamma_w r_e^2}{2\bar{k}_h} \frac{\partial \varepsilon}{\partial t} (\mu + \mu_w) \tag{14}$$

the μ parameter is revealed as:

$$\mu = \frac{n^2}{n^2 - 1} \sum_{i=1}^{m} \kappa_i \left[\frac{s_i^2}{n^2} \ln\left(\frac{s_i}{s_{i-1}} \right) - \frac{1}{2}\left(\frac{s_i^2}{n^2} - \frac{s_{i-1}^2}{n^2} \right) - \frac{\left(s_i^2 - s_{i-1}^2\right)^2}{4n^4} + \psi_i \left(\frac{s_i^2}{n^2} - \frac{s_{i-1}^2}{n^2} \right) \right] \tag{15}$$

where, μ_w is the contribution of the well resistance given by:

$$\mu_w = \frac{\bar{k}_h}{q_w} \pi z (2l - z) \left(1 - \frac{1}{n^2} \right) \tag{16}$$

If well resistance is not included ($q_w \to \infty$) then μ_w is omitted. To give an approximate indication as to how the entire soil layer is affected by well resistance Equation (16) can be averaged over length l to give:

$$\mu_w = \frac{\bar{k}_h}{q_w} \frac{2\pi l^3}{3} \left(1 - \frac{1}{n^2} \right) \tag{17}$$

The μ parameter lies at the heart of the equal strain approach. μ is a non-dimensional parameter depending only on the geometry and material property ratios of the soil/drain system.

The average degree of consolidation due to PVDs (U_h) can be determined from:

$$U_h = 1 - \exp\left(\frac{-8T_h}{\mu}\right) \tag{18}$$

$$T_h = c_h t / d_e^2 \tag{19}$$

where T_h = time factor, t = time, d_e = drain influence zone,

Hansbo's (1981) formulation for a smear zone with constant reduced permeability and an undisturbed zone is a special case of the multiple segment solution presented above.

For a single smear zone $m = 2$, $\kappa_1 = k_h / k'_h = \kappa$, $\kappa_2 = 1$, $s_0 = 1$, $s_1 = r_s / r_w = s$ and $s_2 = r_e / r_w = n$ where r_s is the smear zone radius.

The μ parameter is then given by:

[20]
$$\mu_s = \frac{n^2}{n^2-1}\left(\ln\frac{n}{s} + \kappa \ln s - \frac{3}{4}\right) + \frac{s^2}{n^2-1}\left(1 - \frac{s^2}{4n^2}\right) + \\ + \frac{\kappa}{n^2-1}\left(\frac{s^4-1}{4n^2} - s^2 + 1\right)$$

Ignoring higher order (insignificant) terms Equation (20) reduces to:

[21]
$$\mu_s = \ln\frac{n}{s} - \frac{3}{4} + \kappa \ln s$$

Problem Definition for Multi-layer Soil

During the vertical drain installation using a steel mandrel, the soil surrounding the mandrel is dragged down by friction at the soil-mandrel interface. It is well known that mandrel action remoulds the soft clay causing a reduction in permeability. The high effective vertical stresses caused by the mandrel further decreases the lateral permeability. In addition, the mandrel action that forces soil to be dragged down from upper layers to the underlying regions distinctly divides the clay seam into multi-layer permeability zones (Fig. 3). The consolidation process of the underneath soil layer can be retarded, if the soil permeability in the upper layer is lower. As the dragdown of soil influences the consolidation of lower soil layer, a two-layer soil system can be used to determine the consolidation response of the lower layer (Fig. 4). The dragdown effect influences the variation pattern of soil permeability of underlying layer. In the analysis, the permeability of lower soil layer with the intrusion zone has been divided into 3 zones, the undisturbed zone, the smear zone due to the remoulding of lower soil layer and the smear zone due to the intrusion of the upper soil layer. Indraratna and Redana (2000) show that the lateral permeability can vary in the order of 10 within the multilayer soil in which the soil compressibility remains relatively constant, and the variation of the extent of the smear zone (d_s) is insignificant. On the basis of the above, the assumptions can be made:

(a) Horizontal permeabilities of soil in layers 1 (upper) and 2 (lower) are k_{h1} and k_{h2}, respectively. Based on linear decay in horizontal permeability towards the drain proposed by Indraratna and Walker (2007), the smear zone characteristics (i.e. s and κ) in both layers remain the same.

(b) Both soil layers have the same compressibility and overlapping of smear zone does not occur.

Figure 4c presents the variation of soil permeability after mandrel installation at Sections 1-1, 2-2 and 3-3. Sections 1-1 and 3-3 represent the normal situation when the soil is homogeneous. In Section 2-2 where the intrusion of the soil layer (radius $d_i/2$) occurs (infiltration zone), the permeability variation can be expressed as:

[22] $$k_h = k_{h,upper}\left[\frac{A_1}{r_w}r + B_1\right]; \qquad d_w/2 < r < d_i/2$$

[23] $$k_h = k_{h,lower}\left[\frac{A_2}{r_w}r + B_2\right]; \qquad d_i/2 < r < d_s/2$$

[24] $$A_1 = \frac{\kappa_{upper} - 1}{s-1}$$

[25] $$A_2 = \frac{\kappa_{lower} - 1}{s-1}$$

[26] $$B_1 = \frac{s - \kappa_{upper}}{s-1}$$

[27] $$B_2 = \frac{s - \kappa_{lower}}{s-1}$$

As observed from Casagrande and Poulos (1969) and Hird and Moseley (2000), the diameter of intrusion as a function of depth (z) in layer 2 can be expressed as:

[28] $$\alpha = \frac{(d_s + d_w)}{2}\left[1 - z/H_2\right] \qquad 0 < z < H_2$$

The normalised penetration depth (D) can be defined as a function of the penetration length due to upper soil layer (d_2) and the lower clay thickness (H_2) :

[29] $$D = d_2 / H_2$$

The normalised penetration depth represents how deep the upper soil layer can be dragged down into the immediately underlying soil layer. When D is more than 1 (the lower soil layer is quite thin compared to penetration length due to upper soil layer), the entire underlying soil is affected by the down-dragging. When D is less than unity, the down-dragged soil from upper layer partially penetrates the underlying soil layer. The value of D has to be obtained by thorough field investigations as it depends on the interaction between the soil and the mandrel. In order to analyse this problem, a piecewise-linear technique (Equation 15) will be used in conjunction with Equations (22)-(29) to obtain the parameter μ. For modern drains which have a very high discharge capacity (>150m^3/year), the well resistance is negligible.

Influence of downdrag of upper soil layer to the underlying soil layer

To investigate the downdrag influence of upper soil layer, the soil layer 2 in Fig. 4b is divided horizontally into n small layers having thickness of $\frac{H_2}{n}$. In each small horizontal layer ith, The μ_i parameter can be determined using the proposed piecewise linear technique (Equations 15) and the variation of the permeability in the intrusion zone can be calculated based on Equations (22)-(29). The average degree of consolidation for soil layer 2 can be determined from:

[30] $$U_h = \frac{1}{n}\sum_{i=1}^{n}\left[1-\exp\left(-\frac{8T_h}{\mu_i}\right)\right]$$

It can be seen that μ_i depends on smear zone characteristics (κ and s), permeability ratio between upper and lower soil layers ($k_{h,upper}/k_{h,lower}$), penetration depth (D), and drain spacing (n).

The overall time factor at 90% degree of consolidation (T_{h90}) of soil layer 2 can be determined from:

[31] $$0.9 = \frac{1}{n}\sum_{i=1}^{n}\left[1-\exp\left(-\frac{8T_{h90}}{\mu_i}\right)\right]$$

Figure 5 presents the variation of T_{h90} with normalised penetration depth (D) where n and s are assumed to be 30 and 5, respectively. When the horizontal permeability in the upper soil layer ($k_{h,upper}$) is less than that in the underlying soil layer ($k_{h,lower}$) (i.e. $k_{h,upper}$/ $k_{h,lower}$<1.0), T_{h90} significantly increases with an increase in penetration depth, retarding the consolidation in the second soil layer. However, when $k_{h,upper}$/ $k_{h,lower}$ >1.0, T_{h90} slightly decreases and becomes constant after D is less than 2. This implies that the horizontal permeability of the overlying soil layer can accelerate or retard the consolidation of the underlying soil layer.

Design Procedures for Multi-layer Soil

Several vertical drains design procedures applicable for a single layer soil use horizontal time factor (T_h) vs. degree of consolidation curves (U_h) to determine the drain spacing (S) (Hansbo 1981; Zhu and Yin 2005; Rujikiatkamjorn and Indraratna 2007). The determination of the drain spacing can become a cumbersome task when the downdrag effect of upper soil layer combines with an iteration process. In this Section, various time factors at 90% degree of consolidation (T_{h90}) are generated based on a linear smear zone concept, where $2 \le \kappa \le 5$, $0.1 \le k_{h,upper} / k_{h,lower} \le 0.5$, $0.1 \le D \le 10$, $3 \le n \le 60$ and $2 \le s \le 17$ (Figs. 6-11). The required drain spacing (S) for a given soil layer considering the downdrag effect of upper soil layer to achieve 90% degree of consolidation can be determined by:

(a) Assume a drain influence zone of d_e;

(b) Determine d_w from d_w=2(b+d)/π (Hansbo 1981);

(c) Determine n_1 from n= d_e /d_w;

(d) Based on n_1, D, κ; $k_{h,upper} / k_{h,lower}$, and s, determine T_{h90} from Figs. 6-17. Calculate d_e from $d_{e2} = \sqrt{\dfrac{c_h t}{T_{h90}}}$ where c_h = radial coefficient of consolidation, t = required time. Determine n_2 from n = d_{e2} /d_w If the above parameters are the same as those given in the Figures, extrapolation and interpolation can be used to find n_2.;

(e) If $n_1 \neq n_2$, select n_3 from $n_3=(n_2 + n_1)/2$. Repeat procedures *(d)* and *(e)* until n_1= n_2

(f) Determine drain influence zone (d_e) from d_e=1.13S for PVDs with square pattern installation and d_e=1.05S for PVDs with triangular pattern installation;

(g) The overall degree of consolidation of n-layer soil (U) can be determined by

[32] $$U_h = \frac{\sum_i^n U_{hi} \Delta h_i}{\sum_i^n \Delta h_i}$$

where, U_{hi} is the degree of consolidation of soil layer i^{th} and Δh_i is the thickness of soil layer i^{th}.

For the upper soil layer, the degree of consolidation based on linear smear zone can be determined from Eq. (18) where μ can be expressed by (Walker and Indraratna 2007):

[33] $$\mu_L = \ln\left(\frac{n}{s}\right) - \frac{3}{4} + \frac{\kappa\,(s_L - 1)}{s - \kappa}\ln\left(\frac{s}{\kappa}\right)$$

For the case when $s_L = \kappa_L$ the μ parameter is:

[34] $$\mu = \ln\left(\frac{n}{s}\right) - 1.75 + s$$

Worked-out Example

The above methodology is illustrated by the following example. The foundation soils are divided into 2 layers, whereby t = 1.2 years and well resistance is ignored. The required soil parameters for normally consolidated clay for each soil layer are assumed to be:

Layer 1 (Upper layer):

c_{h1} = 0.5 m^2/year, κ= 5, s_1 = 5, $k_{h,\ upper}$=1×10^{-10} m/s, H_1=7m

Layer 2 (Lower layer):

c_{h2} = 1.0 m^2/year, κ= 5, s_2 = 5, $k_{h,\ lower}$=2×10^{-10} m/s, H_2=3m, D= 0.5

PVDs with d_w = 0.06 m are expected to be installed in square pattern

Determine the drain spacing (S) when degree of consolidation of layer 2 is 90% and overall degree of consolidation for both upper and lower soil layer.

Solution:

Considering second soil layer

a) Assume d_e =0.9m;

b) n_1 =0.9/0.06 = 15, $k_{h,upper}/k_{h,lower}$ = 0.5;

c) Based on n_1, D, κ; $k_{h,upper}/k_{h,lower}$, and s, determine T_{h90} = 1.46 from Fig. 11.

d) $n_2 = \sqrt{\frac{1\times1.2}{1.46}} \Big/ 0.06 = 15.1$, $n_1 \approx n_2$, therefore n= 15.0

e) Based on n_3, D, κ; $k_{h,upper}/k_{h,lower}$, and s, determine T_{h90} = 1.48 from Fig. 14.

f) $n_4 = \sqrt{\frac{1\times1.2}{1.48}} \Big/ 0.06 = 15.0$, therefore n= 15 and d_e =0.9m

g) For the upper soil layer, the degree of consolidation determined from Eqs. (20) and (34) is 75%.

h) The average degree of consolidation for both soil layers based on Equation (32) is 80%. For comparison, the overall degree of consolidation is 92% if the downdrag effect of soil is ignored in the lower layer.

Conclusions

Prefabricated vertical drains increase the rate of soil consolidation by providing a short horizontal drainage path, and are now used worldwide in many soft soil improvement projects. A radial consolidation model using a piecewise technique is proposed to study the smear effect in a layered soil with the downdrag of soil caused by mandrel action. The consolidation of the multi-layer soil depends on smear zone charactersitics, permeability ratio between upper and lower soil layers, penetration depth, and drain spacing. When the horizontal permeability in the upper soil layer is less than that of the underlying soil layer, consolidation time of the underlying soil layer can be retarded depending on the upper soil penetration depth. On the other hand, the consolidation process can be accelerated further when the downdrag upper soil layer has a higher permeability. The design charts have been provided to include this downdrag effect of the upper soil layer into immediate lower soil layer. The proposed design method is most beneficial to the practitioner as a preliminary tool for the design of embankments stabilized by prefabricated vertical drains in a multi-layered soil affected by the downdrag.

References

Barron, R.A. 1948. Consolidation of fine-grained soils by drain wells. Transactions, ASCE, 113, 718-742.

Basu, D. and Prezzi, M. 2007. Effect of the smear and transition zones around prefabricated vertical drains installed in a triangular pattern on the rate of soil consolidation. International Journal of Geomechanics, ASCE, Vol. 7, No. 1, pp. 34-43.

Bo, M. W., Chu, J., Low, B. K., and Choa, V. 2003. Soil improvement; prefabricated vertical drain techniques, Thomson Learning, Singapore.

Bergado, D.T., Asakami, H., Alfaro, M.C., and Balasubramaniam, A.S. 1991. Smear effects of vertical drains on soft Bangkok clay. Journal of Geotechnical Engineering, ASCE, 117(10), 1509-1530.

Casagrande, A. and Poulos, S. 1969. On the effectiveness of sand drains. Can. Geotech Journal., 6 pp. 287-326.

Chai, J. C. and Miura, N. 1999. Investigation of factors affecting vertical drain behaviour. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 125(3), 216-226.

Eriksson, U., Hansbo, S. and Torstensson, B.A. 1999. Soil improvement at Stockholm-Arlanda Airport. Ground Improvement, 4(2), 73-80.

Fox, P.J., Di Nicola, M., and Quigley, D.W. 2003. Piecewise-linear model for large strain radial consolidation. Journal of Geotechnical and Geoenvironmental Engineering, **129**(10): 940–950.

Hansbo, S. 1981. Consolidation of fine-grained soils by prefabricated drains. "Proc. 10th International Conference Soil Mechanics and Foundation Engineering, Stockholm, 677-682.

Hawlader, B., Imai, G. and Muhunthan, B. 2002. Numerical study of the factors affecting the consolidation of clay with vertical drains. Geotextiles and Geomembranes, 20, 213-239.

Hird, C.C. and V.J. Moseley 2000. Model study of seepage in smear zones around vertical drains in layered soil. Geotechnique, 50(1), 89-97.

Indraratna B., and Redana I.W. 2000. Numerical modeling of vertical drains with smear and well resistance installed in soft clay. Canadian Geotechnical Journal, **37**(1):133-145.

Indraratna, B., Rujikiatkamjorn, C. and Sathananthan, I. 2005a. Radial Consolidation of Clay using Compressibility Indices and Varying Horizontal Permeability. Canadian Geotechnical Journal , Vol. 42, pp. 1330-1341.

Indraratna, B., Rujikiatkamjorn, C. and Sathananthan, I. 2005b. Analytical And Numerical Solutions for a Single Vertical Drain including the Effects of Vacuum Preloading. Canadian Geotechnical Journal, Vol. 42, Issue 4, pp. 994-1014.

Jamiolkowski, M., Lancellotta, R., and Wolski, W. 1983. Precompression and speeding up consolidation. *In* Proceedings of 8th European Conference on Soil Mechanics and Foundations, Helsinki, Finland, **3**, pp.1201-1206.

Li, A.L., and Rowe, R.K. 2002. Combined effect of reinforcement and prefabricated vertical drains on embankment performance. Canadian Geotechnical Journal, **38**:1266-1282.

Lo, D. 1998. Vertical drain performance: myths and facts. Transactions, Hong Kong Inst. Eng, 5(1), 34-50.

Madhav, M. R., Park, Y.-M. and Miura, N. 1993. Modelling and study of smear zones around band shaped drains." Soils and Foundations, 33(4), 135-147.

Onoue, A. 1988. Consolidation of multi-layered anisotropic soils by vertical drains with well resistance. Soil and Foundations, Japanese Geotechnical Society, 28(3), 75-90.

Rujikiatkamjorn, C. and Indraratna, B. 2007. Analytical solutions and design curves for vacuum-assisted consolidation with both vertical and horizontal drainage. Canadian Geotechnical Journal, 44 (2), 188-200.

Sharma, J.S. and Xiao, D. 2000. Characterization of a smear zone around vertical drains by large-scale laboratory tests. Canadian Geotechnical Journal, 37(6), 1265-1271.

Tang, X.W. and K. Onitsuka 2001. Consolidation of double-layered ground with vertical drains. International Journal for Numerical and Analytical Methods in Geomechanics, 25(14), 1449-1465.

Wang, X.-S. and J.J. Jiao 2004. Analysis of soil consolidation by vertical drains with double porosity model. International Journal for Numerical and Analytical Methods in Geomechanics, 28, 1385-1400.

Walker, R 2006, Analytical solutions for modeling soft soil consolidation by vertical drains, PhD thesis, School of Civil, Mining and Environmental Engineering, University of Wollongong.

Walker, R. and Indraratna, B. 2007. Vertical drain consolidation with overlapping smear zones. Geotechnique, Institution of Civil Engineers, UK, 57(5). pp. 463-467.

Xie, K.-H., Xie, X.-Y. and Gao, X. 1999. Theory of one-dimensional consolidation of two-layered soil with partially drained boundaries. Computers and Geotechnics, 24, 265-278.

Xie, K.-H., Xie, X.-Y. and Jiang, W. 2002. A study on one-dimensional nonlinear consolidation of double-layered soil. *Computers and Geotechnics*, 29, 151-168.

Yoshikuni, H. and Nakanodo, H. 1974. Consolidation of Fine-Grained Soils by Drain Wells with Finite Permeability. Soil and Foundations, Japanese Geotechnical Society, 14(2), 35-46.

Zhu, G. and Yin, J.-H. 2005. Solution charts for the consolidation of double soil layers. Canadian Geotechnical Journal, 42, 949-956.

List of Figures

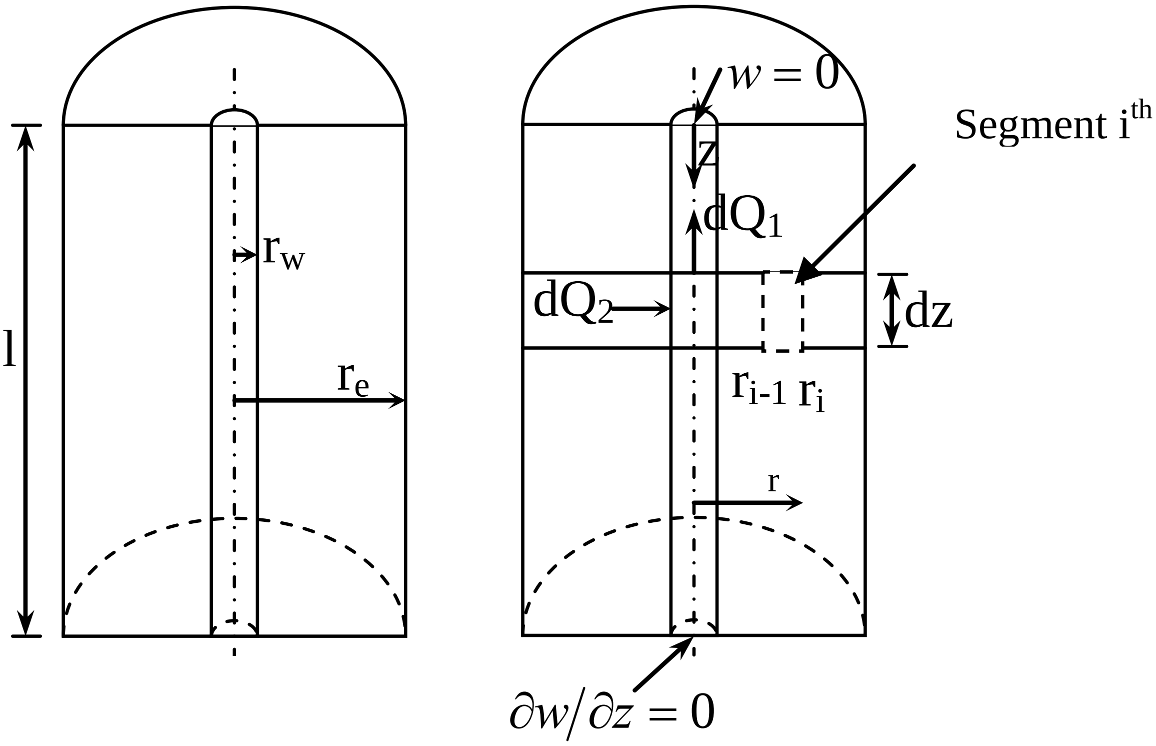

Figure 1 Axisymmetric unit cell

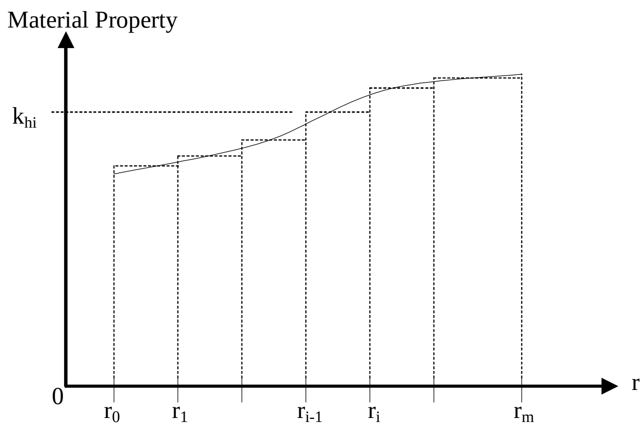

Figure 2 Discretised radial permeability

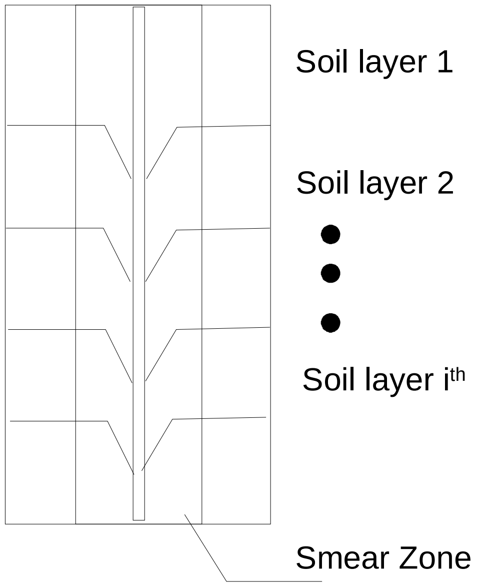

Figure 3 Downdrag effects due to mandrel installation in layered soil

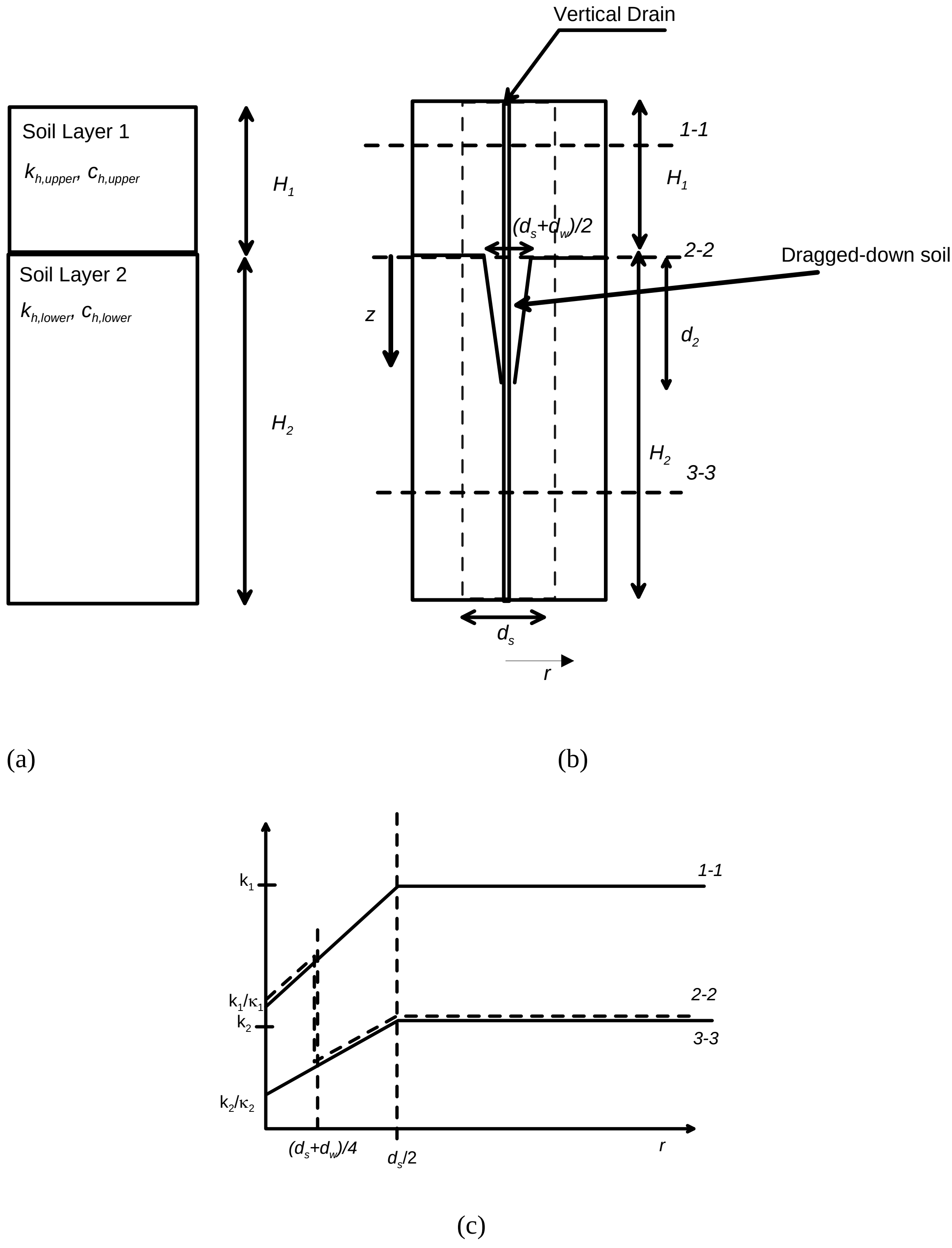

Figure 4 (a) Soil system with upper and lower soil layer, (b) Downdrag and smear effect due to mandrel installation in two-layer soil, (c) permeability variations along the radial direction at various horizontal cross sections

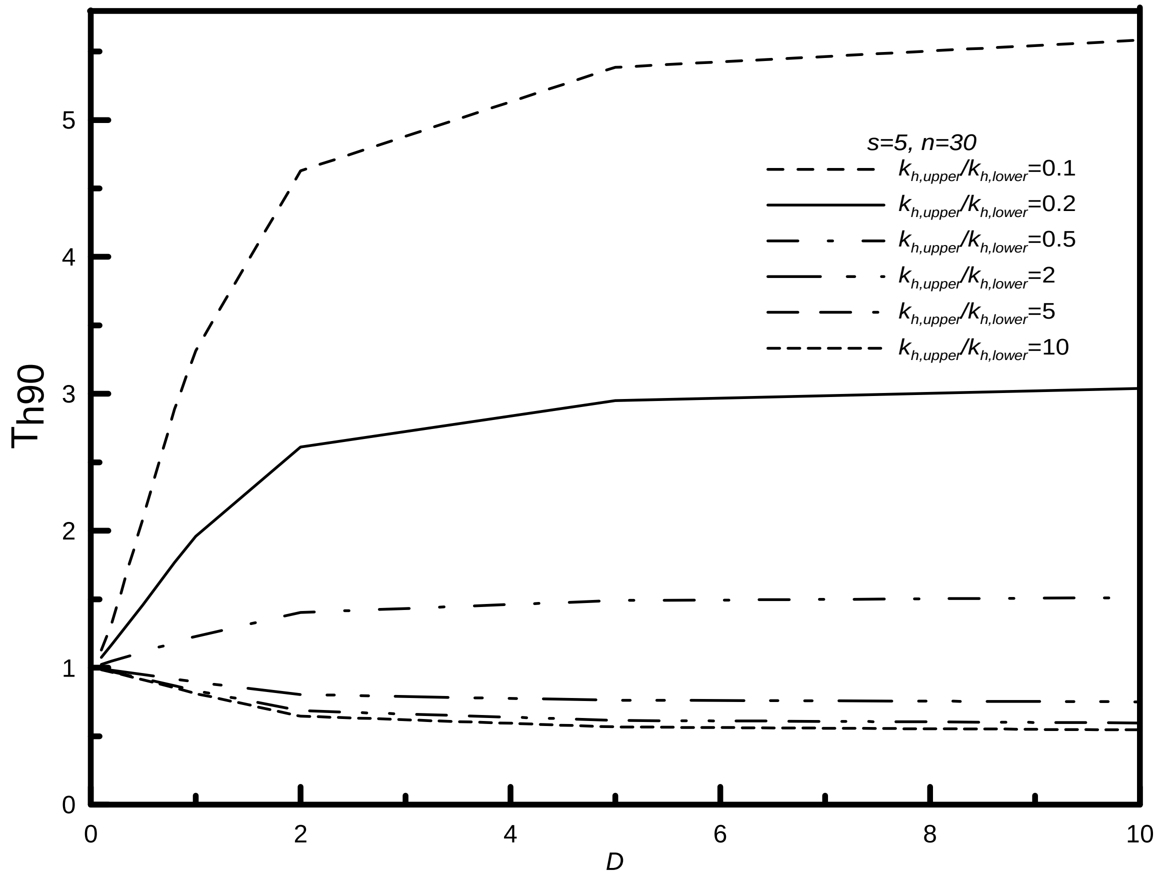

Figure 5 Time factor at 90% degree of consolidation with normalised penetration depth

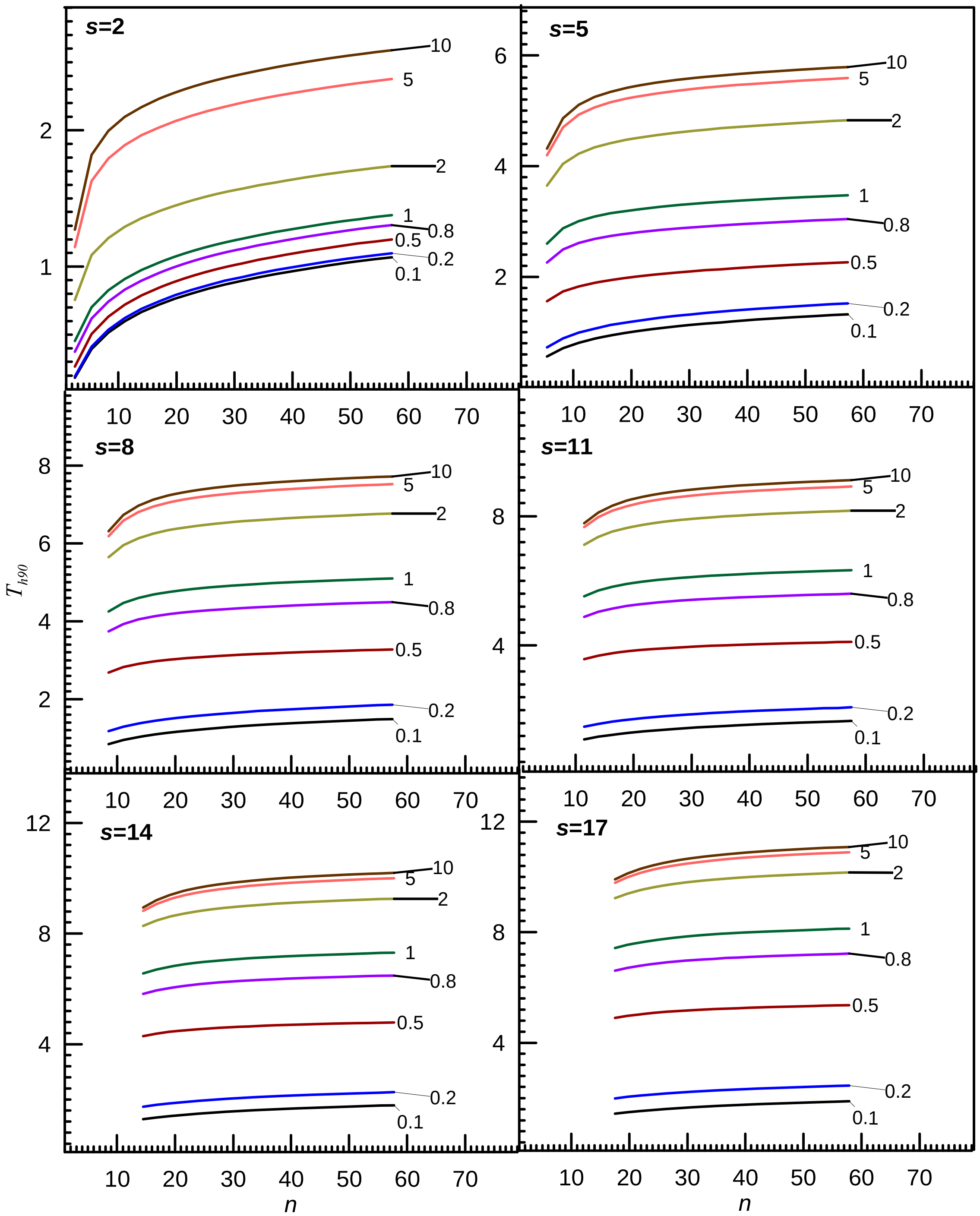

Figure 6 Time factor at 90% degree of consolidation when $\kappa = 2$, $k_{h,upper} / k_{h,lower} = 0.1$

(Number at the end of each curve represents the normalised penetration depth, *D*)

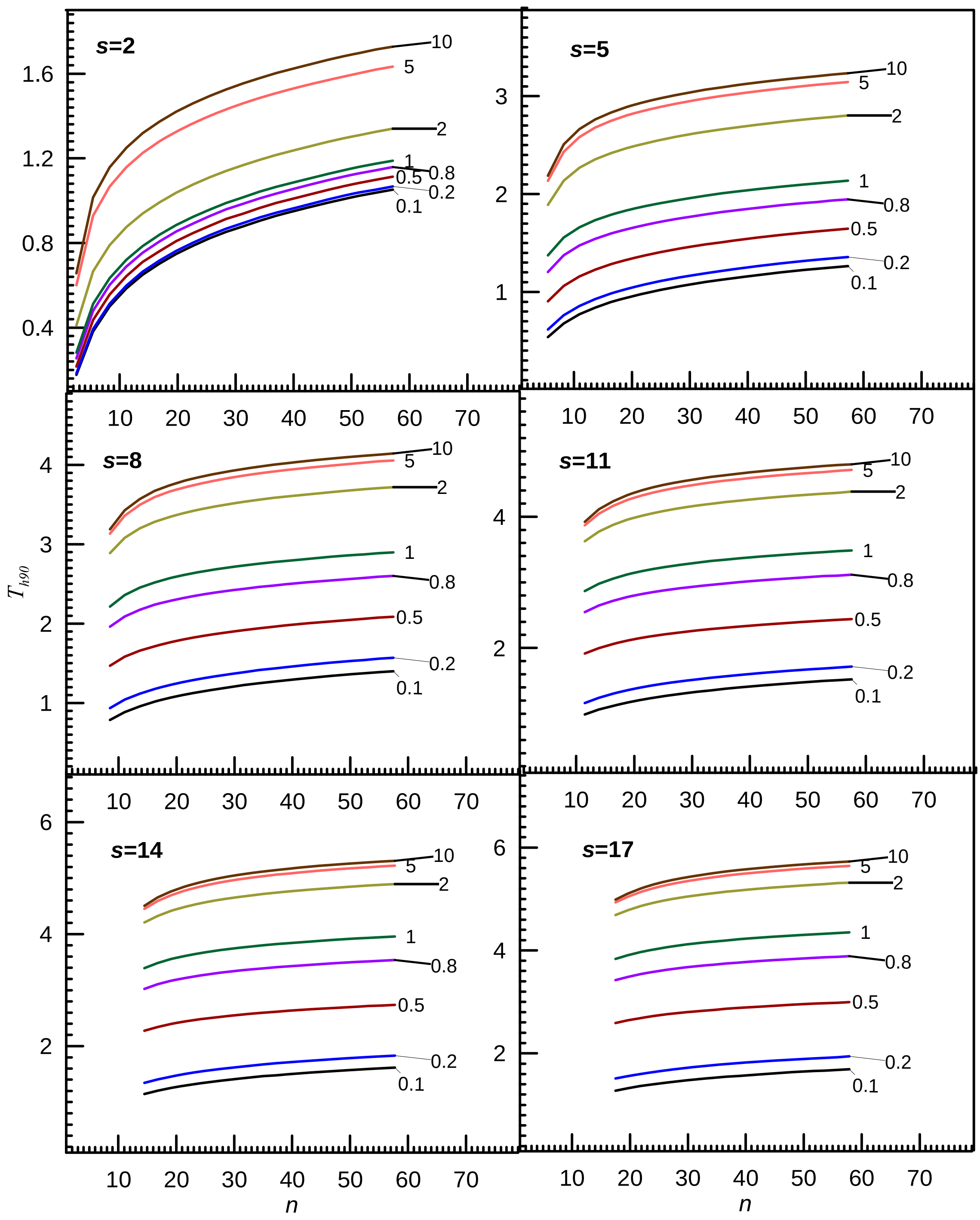

Figure 7 Time factor at 90% degree of consolidation when $\kappa = 2$, $k_{h,upper} / k_{h,lower} = 0.2$

(Number at the end of each curve represents the normalised penetration depth, *D*)

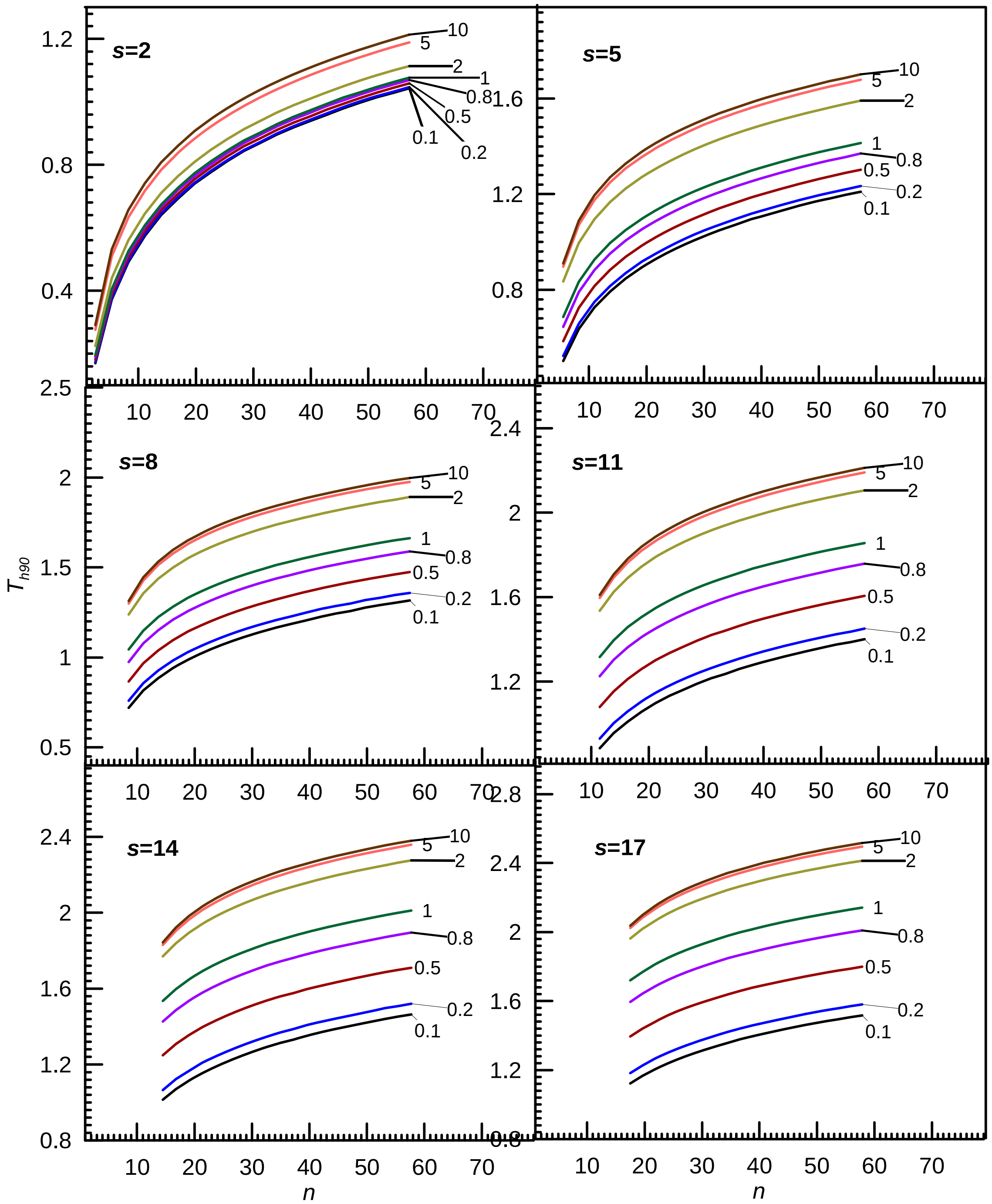

Figure 8 Time factor at 90% degree of consolidation when $\kappa = 2$, $k_{h,upper} / k_{h,lower} = 0.5$

(Number at the end of each curve represents the normalised penetration depth, *D*)

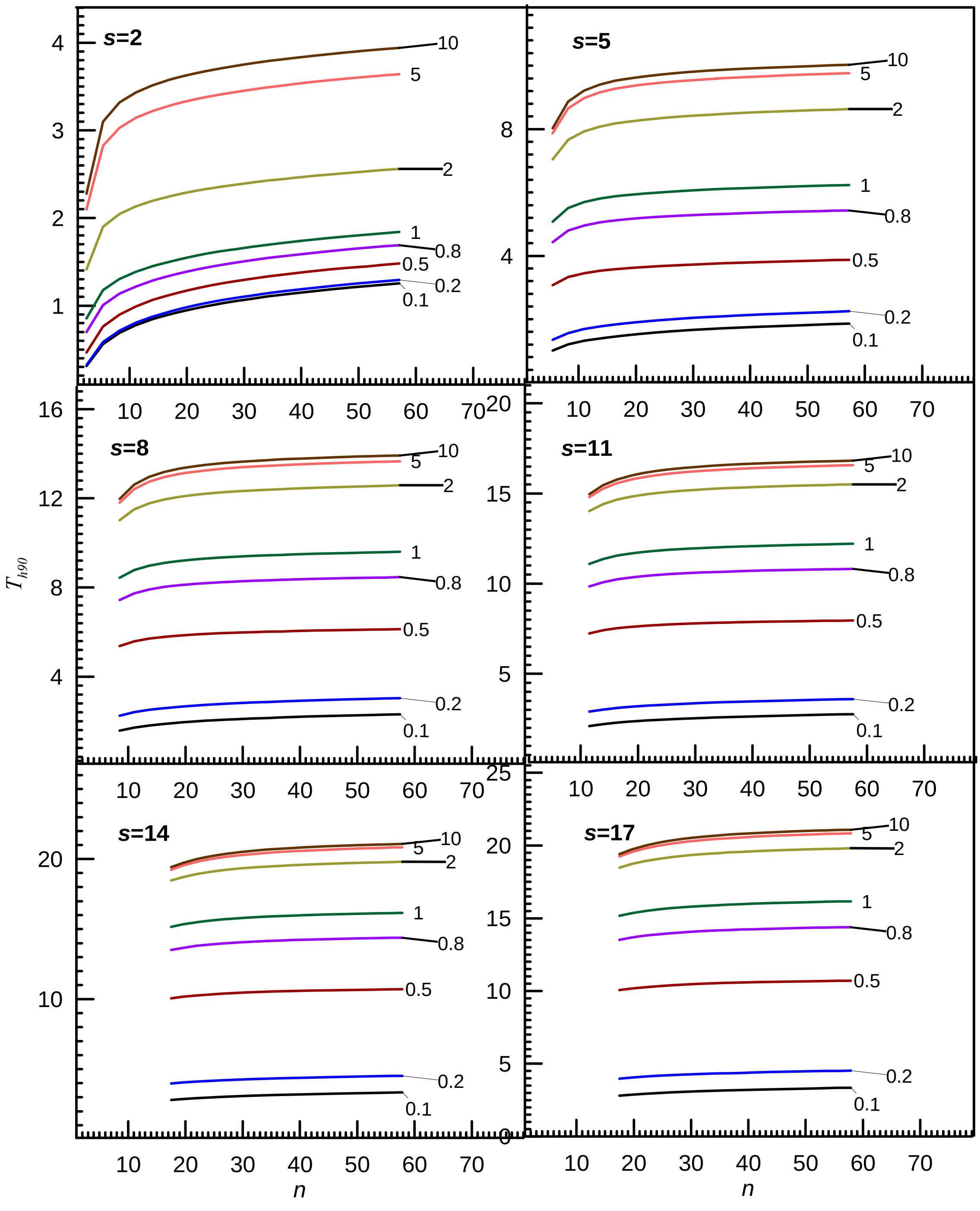

Figure 9 Time factor at 90% degree of consolidation when $\kappa = 5$, $k_{h,upper} / k_{h,lower} = 0.1$

(Number at the end of each curve represents the normalised penetration depth, D)

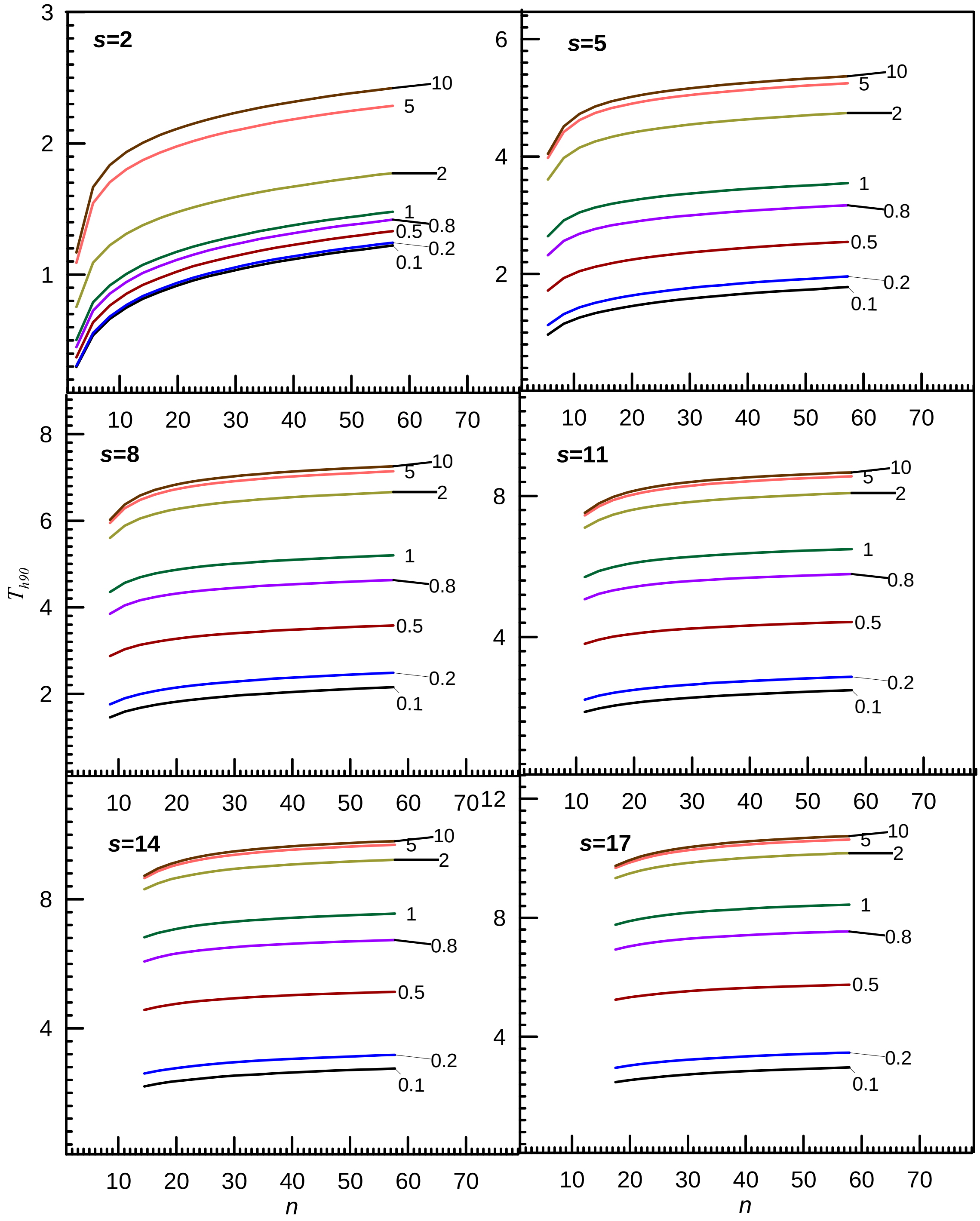

Figure 10 Time factor at 90% degree of consolidation when $\kappa = 5$, $k_{h,upper} / k_{h,lower} = 0.2$

(Number at the end of each curve represents the normalised penetration depth, D)

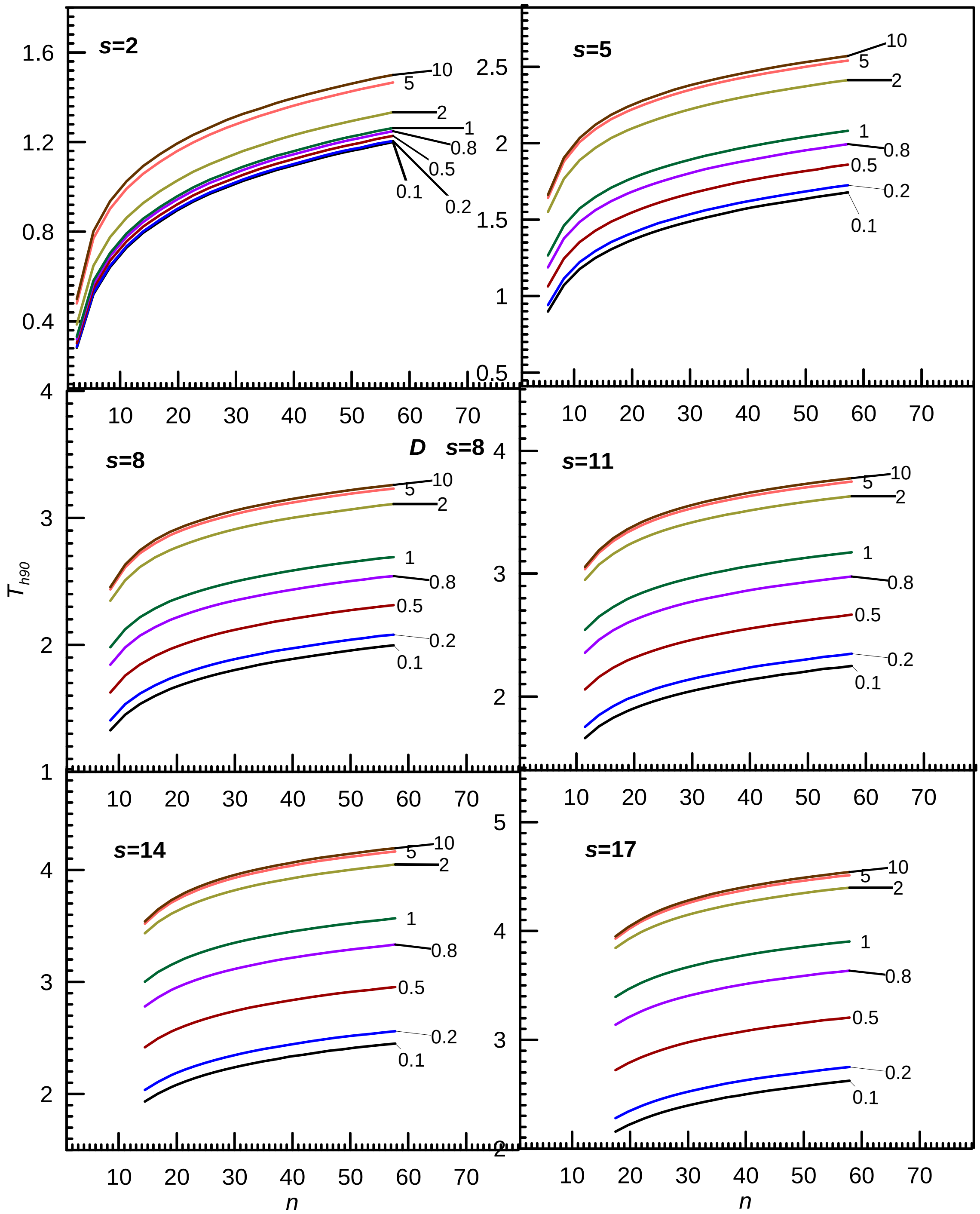

Figure 11 Time factor at 90% degree of consolidation when $\kappa = 5$, $k_{h,upper} / k_{h,lower} = 0.5$

(Number at the end of each curve represents the normalised penetration depth, D)