

Original Research Article

A numerical procedure of coupled consolidation in saturated soils

ABSTRACT

Consolidation is a process of decreasing soil volume that occurs in a lapse of time, derives an stresses transfer process that begins with the loading of the foundation and passes to the water phase contained in the soil, producing a reduction in pore pressure and the volume of the soil by the rearrangement of the solid particles. Thus, consolidation is considered a flow-mechanical coupled problem. Models have been developed to simulate this phenomenon properly. Therefore, this paper presents a coupled model of saturated consolidation, it can be extended to the unsaturated consolidation due to the introduction of soil effective stress. This model provides quick and easy results due to its flexibility, allowing infinite combinations in its boundary and loading conditions. The results show an appropriate behavior of the phenomenon of saturated consolidation and are consistent with those reported by other authors and those obtained in the laboratory.

Keywords: coupled model; consolidation theory; saturated soil, dynamic solution, fluid-solid fields

1. BACKGROUND

The soil suffers a volume decrease (consolidation) induced by applied load of the foundation and variations in pore pressure caused due to the increase in water content derived from rain period and water pipe leaks[1,2]. Varying-Variation in the consolidation process depends on the velocity with which the fluid is ejected out of the soil due to the applied load; hence it is a considered as a problem is considered a "flow-mechanical coupled problem"[3]. Coupled models have been developed to simulate the saturated consolidation, such as provided by: Magaña y Romo[1], Bentler[3], Manjolillo et al.[4], Di-Rado et al. [5] y Krishnamoorthy[6]. These models are based on: a) the soil is saturated, b) the water phase is incompressible, c) the solid phase is incompressible, but ~~his~~its arrangement is compressible, d) Darcy's law governs the behavior of flow through the soil. In addition, some authors provided improvements by incorporating some theories: the theory of small deformations used by Bentler[3]. The principle of virtual work was used by Manjolillo et al. [4] and conservation mass law was considered in models of Manjolillo et al. [4] y Di-Rado et al. [5]. These latter employed the effective stress of Terzaghi and the Galerkin's solution method of weighted residuals, respectively. Besides, Krishnamoorthy[6] took in to account the nonlinear behavior of soil and used the hyperbolic relationship of Duncan and Chang (1970).

Comment [PU1]: Please reword the sentence. It's confusing. If possible, break it into two sentence. (minor revision)

This paper presents a coupled model for the saturated consolidation of soils, which can be further extended to the unsaturated consolidation, because its formulation allows the introduction of effective stress of unsaturated soils that include the Terzaghi's effective stress and matric suction stress from the soil-water retention curves (SWRC).

2. GOVERNING EQUATIONS

Due to seasonal changes in weather seasonal changes, soils are subjected to cyclic wetting and drying processes. When the water content is high, the load on the ground causes the water content decreases causing outflow and consequently reduction of the soil volume. Thus, mechanical soil deformation and flow water pressure is a coupled field problem-field. Some assumptions include: a) Full saturation and water incompressibility, b) the solid phase (soil skeleton) is also incompressible, c) Darcy's law governs the behavior of the flow through the soil. Besides, small deformations theory is already in use. So, according to the Continuum mechanics theory the governing equations are as follow:

i. Equilibrium Equations:

According to the Balance Law of Continuum Mechanics, and introducing the Piola-Kirchhoff Stress Tensor $\underline{S}$ in Lagrangean Configuration:

$$\text{Div} \underline{S} + \underline{b}_o = \ddot{\underline{\chi}} \rho_o \quad (1)$$

Whereas in our case body forces commonly are considered $\underline{b}_o = \underline{0}$, also acceleration field $\ddot{\underline{\chi}} = \underline{0}$, since the settlement processes are considered as quasi-static/quasi-static.

ii. Continuity Equations

$$\text{div} \underline{v} - \dot{\varepsilon} = 0 \quad (2)$$

Here $\underline{v}$ is the velocity field and $\dot{\varepsilon}$ is the derivative of the field strain.

iii. Darcy's Law

$$v_i = k_{ij} (\gamma_w h_i)_{,j} / \gamma_w \quad (3)$$

where: $\underline{v}$ is the flow velocity field, k the permeability coefficient of soil, γ is the specific weight, piezometric level h is evaluated according to the reference level z : $h = p_w / \gamma_w + z$.

iv. Boundary Conditions:

These conditions are related to the pore pressure values on the whole boundary soil and pressure load applied on soil surface. One more condition is involved: the surface inflow due to weather conditions. All these conditions will change in time. The whole boundary Ω include Dirichlet boundary as well Newman's ones Ω_N . Then, we can write these conditions in terms of prescribed values:

a) Pore pressure Conditions:

$$u = \hat{u} \quad \text{on} \quad \partial\Omega_D \quad (4)$$

85 b) Pressure Surface loads:

$$86 \quad \underline{S} \underline{n} = \hat{\underline{l}} \text{ on } \partial\Omega_N \quad (5)$$

87
88 c) Surface Inflow:

$$89 \quad Q = \hat{Q} \quad \text{on } \partial\Omega_i \quad (6)$$

90
91 | One more condition is related to the inflow inside soil. This case can be observed as an induced a-flow
92 charge due to some fail on drainpipes.

93
94 d) Inflow inside soil:

$$95 \quad Q_s = \hat{Q}_s \text{ on } \partial\Omega_i \quad (7)$$

96 97 3. CONSTITUTIVE EQUATION

98
99 | The relationship between Stresses and deformation on soil will follow the Hooke's Law. In particular,
100 Stress Tensor S involves stresses on soil ~~squeleton~~skeleton as on the water.

$$101 \quad \underline{S} = \underline{\bar{S}} + p_w \underline{I} \quad (8)$$

102
103 where: $\underline{S}$ is the Tensor of soil stresses, p_w is the water pore pressure (Hydrostatic Stresses), and $\underline{I}$ is
104 the identity Tensor = $\sum_i e_i \otimes e_i$ use proper symbol for multiplication here.

105
106 The procedure for determining coupling equations is based on the virtual work principle. Discrete coupled
107 final equation is written as follow:

$$108 \quad [K] \{U\} + [K_v]^T \{p\} = \{F\} \quad (9)$$

109
110 Here, $[K]$ is the soil stiffness matrix, $[K_v]^T$ coupled matrix transpose, $\{U\}$ nodal displacements vector,
111 $\{p\}$ nodal water pore pressure, $\{F\}$ external forces vector.

$$112 \quad [K_v] \left\{ \dot{U} \right\} + \frac{1}{\gamma_w} [K_h] \{p\} = \{Q\} \quad (10)$$

113
114 where: $[K_v]$ coupled matrix, $[K_h]$ drained matrix, $\{p\}$ nodal water pore pressure, $\{Q\}$ external/internal
115 flow vector, $\left\{ \dot{U} \right\}$ velocity vector of nodal displacements.

116
117 Resulting matrices involve integral equations:

$$118 \quad [K] = \int_V [B_\phi]^T [\bar{D}] [B_u] dV$$

$$[K_v]^T = \int_V [B_\phi]^T \langle 1 \ 1 \ 0 \rangle^T [N_p] dV \quad (11)$$

$$[K_h] = \int_V [B_\theta]^T [k] [B_p] dV$$

Matrices $[B_i]$ contain derivatives of the shape function, $[k]$ is the permeability matrix, and vector $\langle 1 \ 0 \ 0 \rangle^T$ is required to couple the values corresponding to the degree of freedom of the two involved fields.

The resulting matrix equations associate the coupled model (flow-mechanical) consolidation on saturated soils. Final equation (9) and (10) can be written as a coupled equations:

$$\begin{bmatrix} 0 & 0 \\ K_v & 0 \end{bmatrix} \begin{Bmatrix} \dot{U} \\ p \end{Bmatrix} + \begin{bmatrix} K & K_v^T \\ 0 & \frac{1}{\gamma_w} K_h \end{bmatrix} \begin{Bmatrix} U \\ p \end{Bmatrix} = \begin{Bmatrix} F \\ Q \end{Bmatrix} \quad (12)$$

To solve Eq. (12), we need to apply a dynamic solution procedure. The variables are the soil displacements and the pore pressures over time. One of the procedures used in this work is the application of Finite difference method in time together with Galerkin method for Theta values. The approximates quantities are referred to the external Forces F on soil surface; Induced inflow charges Q ; Pore pressure on the soil p ; Displacements of soil mass U , as well rate movement in terms of its velocity.

$$\begin{aligned} \{F\} &= \{F_a\} + \theta \{\{F_b\} - \{F_a\}\} \\ \{Q\} &= \{Q_a\} + \theta \{\{Q_b\} - \{Q_a\}\} \\ \{p\} &= (1 - \theta) \{p_b\} + \theta \{p_a\} \\ \{U\} &= (1 - \theta) \{U_b\} + \theta \{U_a\} \\ \{\dot{U}\} &= \{U_b\} - \{U_a\} / \Delta t \end{aligned} \quad (13)$$

where: $\{F_a\}$, $\{Q_a\}$, $\{p_a\}$ and $\{U_a\}$ are the initial vectors, $\{F_b\}$, $\{Q_b\}$, $\{p_b\}$ and $\{U_b\}$ are the vectors after an interval of time, θ is the value of Galerkin's parameter on the numerical solution of evolution over time, Δt is the pass of time to avoid numerical oscillations according to the next relationship (for triangle finite elements).

$$\Delta t \leq \frac{\alpha}{1 - \theta} \quad \alpha_{\min} = \frac{2A}{9C_v} \quad (14)$$

where: A is the area of the linear triangular finite element, C_v is the coefficient of consolidation for the soil.

In order to obtain a solution, we proceed to build a Fortran code together with some visualizations tools. Fig. 1 shows the flow chart of the applied solution:

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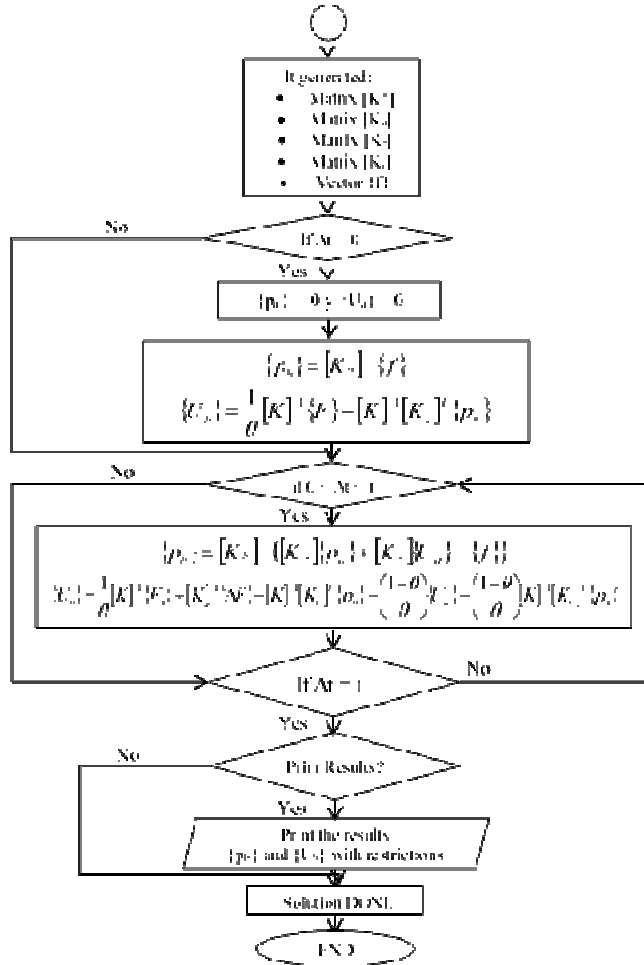


Fig. 1. Flowchart of the dynamic process of saturated consolidation coupled model.

4. NUMERICAL EXAMPLE

We propose a rectangular slab foundation of $L=5.0\text{m}$, having unitary width (plane strain condition) with a distributed surface load $F=4.0\text{ Ton/m}^2$ (40 kPa) on a clay soil of $H=3.0\text{ m}$ thickness (see Fig. 2).

The displacements U_x and U_y are considered restricted in both lateral boundaries and in the lower boundary of the soil. Furthermore, the clay layer is placed above a sand layer which allows outflow of water from the lower boundary. So, on this boundary excess pore pressures are zero. On the other hand, on the lateral boundaries do not allow water outflow. The mesh spacing was taking of 0.25 m (side value of the triangle used mesh).

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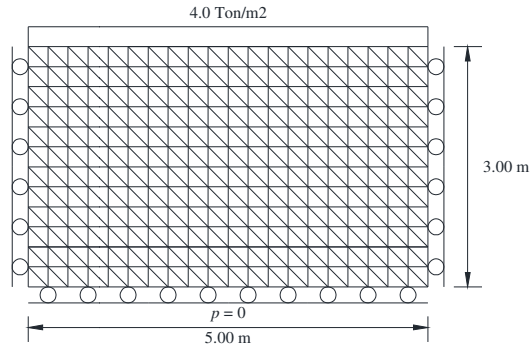


Fig. 2. Soil's domain with slab foundation applied on the surface.

In order to proceed to the dynamic analysis, step time used was of 10 days. The soil properties required for mechanical analysis are shown in the next **TABLE 1**. Although, these values do not necessarily represent an actual soil, and however, the values are suitable for a saturated clayey soil (Any references used to come up with these numbers?).

Table 1. Properties of clayey soil stratum for consolidation analysis.

Soil Properties	Symbol	Magnitude
Elastic Modulus	E	20000kPa
Poisson's ratio	ν	0.35
Coefficient of permeability (x direction)	k_x	0.000117 m/day
Coefficient of permeability (y direction)	k_y	0.000117 m/day

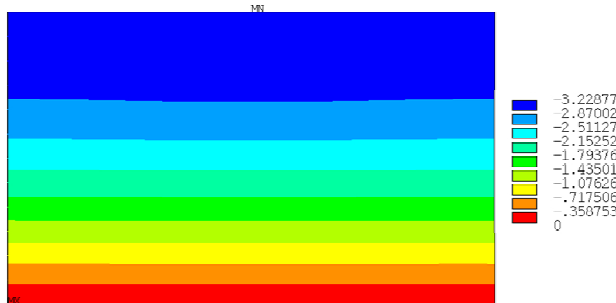


Fig. 3. Pore water pressure distribution at 50 days. Units: T/m²(x10 kPa)

Pore water pressure tend to dissipate when time is increased. This can be seen on

Table 2

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Comment [PU2]: Compulsory revision: The pore water pressure generated due to external loading is "excess pore pressure" developed. Please mention the pore pressure here and everywhere else in text as "excess pore pressure" instead of just pore pressure.

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Table 2, here it shows the distribution of pore water pressure at different times: 50, 450, and 900 days.

Table 2. Pore water pressure distribution.

Pore water pressure distribution Ton/m ² (x 10 kPa)			
Color Scale	50 days	450 days	900 days
	-3.229	-0.463x ⁻⁴	-0.197x ⁻⁹
	-2.870	-0.412x ⁻⁴	-0.175x ⁻⁹
	-2.511	-0.360x ⁻⁴	-0.153x ⁻⁹
	-2.153	-0.309x ⁻⁴	-0.131x ⁻⁹
	-1.794	-0.257x ⁻⁴	-0.109x ⁻⁹
	-1.435	-0.206x ⁻⁴	-0.875x ⁻¹⁰
	-1.076	-0.154x ⁻⁴	-0.656x ⁻¹⁰
	-0.718	-0.103x ⁻⁴	-0.437x ⁻¹⁰
	-0.359	-0.515x ⁻⁵	-0.219x ⁻¹⁰

Fig. 4 shows the evolution of pore water pressure through vertical profiles. Vertical axis refers the depth of soil, horizontal axis shows the pore pressures, all the values tends to zero on below boundary.

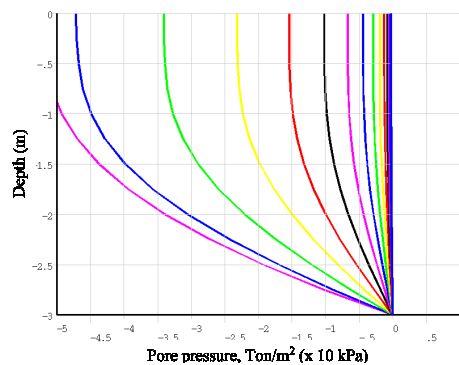


Fig. 4. Evolution of the vertical profiles of pore water pressure.

Another main result is regarding the vertical displacements. These vertical displacements due to applied loads tend to diminish by the applied load as with increasing time goes on. Thus, Fig. 5 shows the vertical displacements in the passage of time at 50 days. Through this figure it can be seen that displacements are about -0.0168 m in the superficial edge, while near the bottom border they are near zero. Here, the negative sign indicates the direction in which evolve the displacements. The behavior of the evolution of vertical displacement is consistent with those obtained in the laboratory and the field. This is evidenced in Fig. 5 and Table 3.

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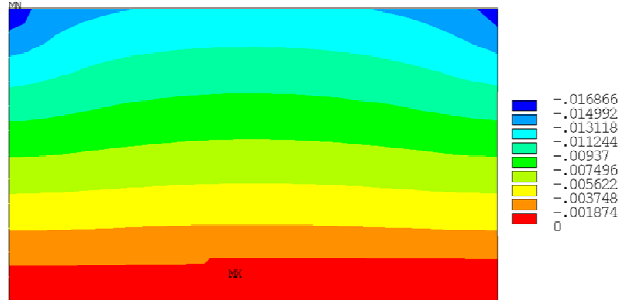


Fig. 5. Distribution of the vertical displacements U_y . Units: meters

In addition, the diagrams of distribution and evolution of the displacements are equal and these values changes with time, only change the values of these as time goes on. Therefore, Fig. 5 shows such distribution, and Table 3 shows the values of the displacements at 50, 450, and 900 days. Furthermore, through Table 3, the maximum displacement observed on surface, is reached at 450 days. In turn, these displacements represent the final values that were caused by the forces transmitted by the slab foundation. This assertion is verified through To verify this assertion, the model also provides a figure (Fig. 6) that shows the evolution of the profile of surface displacements at each passage of time. Thus, the profile of the final displacements of soil due to the applied load is obtained (see Fig. 6).

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Table 3. Distribution of vertical displacements U_y .

Vertical displacements U_y (m)			
Color Scale	50 days	450 days	900 days
	-0.0169	-0.0189	-0.0189
	-0.0149	-0.0168	-0.0168
	-0.0131	-0.0147	-0.0147
	-0.0112	-0.0126	-0.0126
	-0.0094	-0.0105	-0.0105
	-0.0075	-0.0084	-0.0084
	-0.0056	-0.0063	-0.0063
	-0.0037	-0.0042	-0.0042
	-0.0018	-0.0021	-0.0021

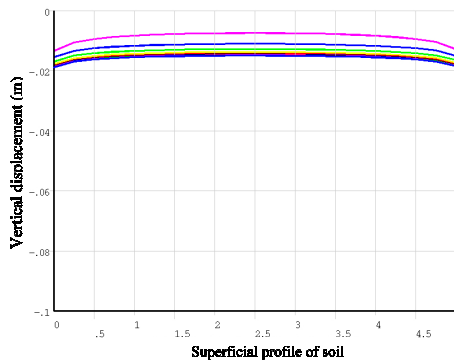


Fig. 6. Superficial profile of soil.

Another case of interest is when an isolated foundation on a saturated clay soil is placed as shown in Fig. 7. For the next exercise the same domain with $L=5.0$ m was used, but only one isolated footing is placed to the center and transfers a distributed load of -40 kPa (-4 Ton/m²) over 2.5 m. With this, it is observed that the free parts of the upper border allow water flow (1.25 m on each side of the foundation). It means that the excess pore pressure developed is zero. Also, the lower boundary also allows water flow. The displacements are restricted in the lateral and lower boundaries (see Fig. 7, Table 4). Table 4 shows the properties of the soil, the passage of time and the time required for dynamic analysis.

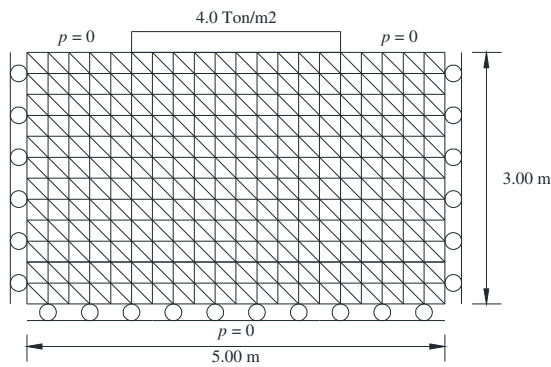


Fig. 7. Soil's domain with isolated footing applied on the surface.

Table 4. Data required for the dynamic analysis.

Data for dynamic analysis	Symbol	Magnitude
Elastic Modulus	E	20000 kPa
Poisson's ratio	ν	0.35
Coefficient of permeability (x direction)	k_x	0.00117 m/day
Coefficient of permeability (y direction)	k_y	0.00117 m/day
Passage of time	Δt	30 days
Time analysis	t	900 days

After the analysis is complete, we note that pore pressure for a time step of 30 days is very close to the value of the pressure applied by forces on the surface. Thus, Fig. 8 shows the distribution of pore pressure for time steps of 30 days. However, but the diagram of the distribution is the same for 450 and 870 days with; only change being in the values of pore water pressures as are shown in.

Table 5

Table-5. Furthermore,

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Table 5

Table-5 shows again that pore water pressure dissipates with time and, it means that it's values, tending to become null-values.

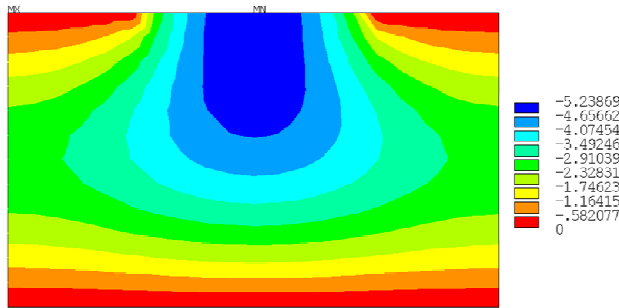


Fig. 8. Distribution of pore water pressure for isolated footing at 30 days.Units: T/m² (x10 kPa)

Table 5. Distribution of pore water pressure for isolated footing.

Pore water pressure distribution Ton/m ² (x 10 kPa)			
Color Scale	30 days	450 days	870 days
	-5.238	-0.441	-0.023
	-4.656	-0.392	-0.021
	-4.074	-0.343	-0.018
	-3.492	-0.294	-0.016
	-2.910	-0.245	-0.013
	-2.328	-0.196	-0.010
	-1.746	-0.147	-0.008
	-1.164	-0.098	-0.005
	-0.582	-0.049	-0.003

Another result of analysis obtained by using the coupled model, are the vertical profiles of the pore water pressure. Fig. 9 shows a vertical profile of pore pressure at a distance of 1.251.25 m, this is in the domain zone that is uncharged, and allows water flow. Thus, it can be seen that the pore pressure is zero at the surface and the lower border, while within the domain it undergoes variations.

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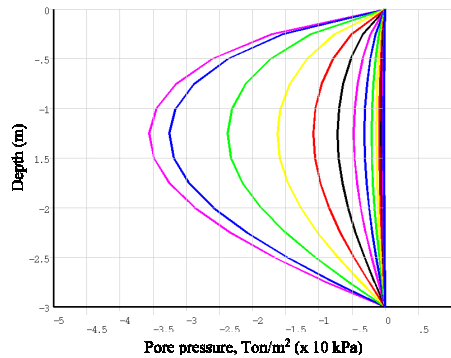


Fig. 9. Evolution of the vertical profiles of pore water pressure at 1.25 m.

Moreover, the [Fig. 10](#) shows the vertical profile of the pore water pressure at 2.50 m (center line of the domain). We can see that the values, start very close to the value of the applied force on surface, and as the depth increases, the value decreases. In both figures, the values of pore water pressures tend to be zero.

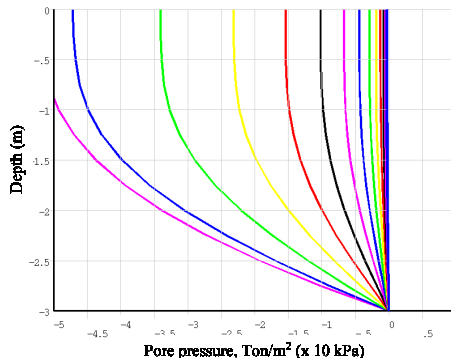


Fig. 10. Evolution of the vertical profiles of pore water pressure at 2.50 m.

As for measuring the displacements, the model provides diagrams of distribution of horizontal U_x and vertical U_y displacement. Thus, [Fig. 11](#) shows the distribution of horizontal displacement and [Fig. 12](#) shows the vertical displacements. Additionally, [Table 6](#) and [Table 7](#) show the distribution of displacement U_x and U_y for each time step (30, 450, and 870 days).

Through [Fig. 11](#) and [Table 6](#), a characteristic phenomenon of symmetrical problems due to loads applied at center is observed. So, from the center line to the left, we have offset values with a negative sign. This is due to the direction in

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which they occur, given a coordinate system in which the y-axis corresponds to the center line. Moreover, the values of the displacements are positive, from the center line to the right.

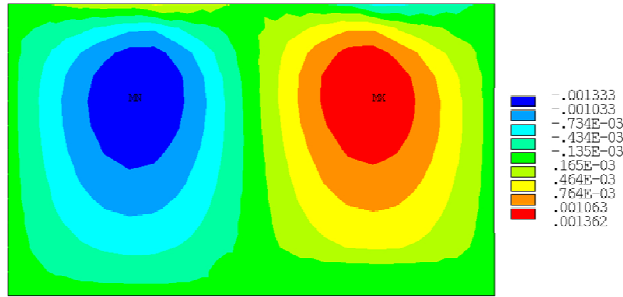
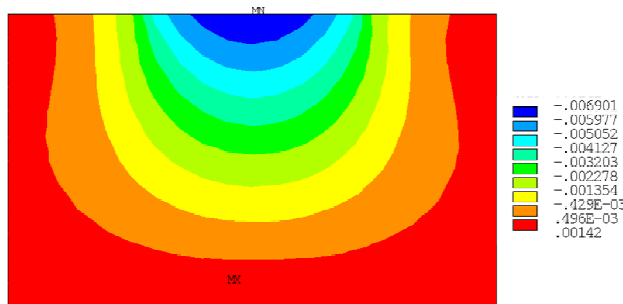


Fig. 11. Horizontal displacements U_x for isolated footing.

Table 6. Distribution of U_x displacements for isolated footing.

Horizontal displacements U_x (m)			
Color Scale	30 days	450 days	870 days
	-1.33×10^{-3}	-1.52×10^{-3}	-1.51×10^{-3}
	-1.03×10^{-4}	-1.18×10^{-4}	-1.17×10^{-4}
	-7.34×10^{-4}	-8.35×10^{-4}	-8.26×10^{-4}
	-4.34×10^{-4}	-4.91×10^{-4}	-4.86×10^{-4}
	-1.35×10^{-4}	-1.48×10^{-4}	-1.46×10^{-4}
	1.65×10^{-4}	1.96×10^{-4}	1.95×10^{-4}
	4.64×10^{-4}	5.40×10^{-4}	5.35×10^{-4}
	7.64×10^{-4}	8.83×10^{-4}	8.75×10^{-4}
	1.06×10^{-4}	1.23×10^{-4}	1.22×10^{-4}

Observing Fig. 12 and Table 7, the evolution of the vertical displacement U_y is appreciated. Furthermore, through this figure it can be seen that displacements are about -0.0069 m in the superficial edge, while near the bottom border are near zero. Here, the negative sign indicates the direction in which evolve the displacements evolve. Vertical displacements are produced by the load applied on the surface and the center of the length of the domain. It is notable that these values also have a symmetrical behavior. They are also appropriate and consistent with the results reported by other researchers.



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Fig. 12. Vertical displacements U_y for isolated footing.

Fig. 13 shows the surface profile of vertical displacements. Again, we can see that these are negative (decrease of volume) and the maximum value (-0.0130 m) occurs in the middle of the domain, i.e., it means, in the part that is subjected to surface forces. Furthermore, in areas (which areas?) uncharged positive offsets values are presented. This shows us that these areas suffer an increase in volume or bulging.

Table 7. U_y displacements for isolated footing.

Vertical displacements U_y (m)			
Color Scale	30 days	450 days	870 days
	-6.90×10^{-3}	-1.30×10^{-2}	-1.30×10^{-2}
	-5.97×10^{-3}	-1.16×10^{-2}	-1.16×10^{-2}
	-5.05×10^{-3}	-1.10×10^{-2}	-1.10×10^{-2}
	-4.13×10^{-3}	-8.68×10^{-3}	-8.68×10^{-3}
	-3.20×10^{-3}	-7.23×10^{-3}	-7.23×10^{-3}
	-2.28×10^{-3}	-5.79×10^{-3}	-5.79×10^{-3}
	-1.35×10^{-3}	-4.34×10^{-3}	-4.34×10^{-3}
	-4.29×10^{-4}	-2.89×10^{-3}	-2.89×10^{-3}
	-4.96×10^{-4}	-1.45×10^{-3}	-1.45×10^{-3}

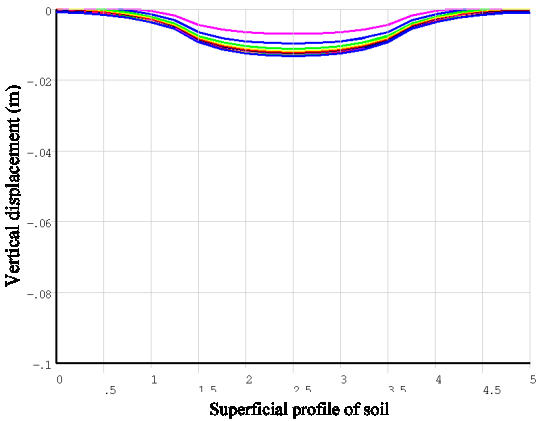


Fig. 13. Superficial profile of soil for isolated footing.

5. CONCLUSIONS

The coupled model algorithm delivers results quickly and easily, just by simply introducing basic soil parameters. It is flexible, allowing infinite combinations that can reproduce any field or laboratory conditions. It also considers any issue in which there are input or extraction water, through the flow vector $\{Q\}$. The above produces changes in pore pressure and the horizontal and vertical displacements. Furthermore, the results show an appropriate behavior of the phenomenon of saturated consolidation. The Even, results from present studies are consistent to those reported by other researchers and to those obtained in the laboratory.

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Another advantage of the model is that it was formulated so that it can be extended to simulate the unsaturated consolidation. This is possible because its structure takes in to account the effective stress of saturated soil which can be replaced by the unsaturated effective stress. Additionally, the permeability coefficient which is in the flow matrix can vary with the water content changes. Furthermore, Also, due to the presence of the flux vector can be considered that these values can be negative or positive. In future Therefore, efforts will be are aimed towards unsaturated consolidation study using this model.

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