Settlements around pumping wells: analysis of influential factors and a simple calculation procedure.

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Abstract

Estimated and measured settlements caused by pumping rarely agree. Several reasons could explain this mismatch, including the influence of layering, the mechanical parameters used in the predictions, or the relationship between settlements and drawdown. We analyze the influence of the above issues by investigating the mechanical response of pumped elastic porous media under different conditions. A radially symmetric conceptual model is considered and several hydro-mechanical simulations are performed varying the boundary conditions, the size of the modeled domain and the presence or not of an overlying layer. The simplicity of the considered problem allows us to compare our results with existing analytical solutions, to identify the role of each variable on pumping settlements and to generalize the results. The most relevant results are as follows: (1) Settlements are proportional to drawdown only outside a circle of radius equal to 0.7 times the thickness of the pumped porous medium; inside, they are virtually constant, which leads to two easy procedures for computing pumping settlements. (2) Poorly conductive layers located above (or below) a pumped porous medium (with higher hydraulic conductivity) reduce and smooth settlements. (3) Boundary constraints affect the local specific storage and the displacements that occur in pumped porous media. (4) The storage coefficient evaluated by interpreting pumping tests with the Cooper and Jacob method (1946) leads to overestimation of the actual Young's Modulus of the soil. The main conclusion is that settlements are less differential than expected near pumping wells. Still, they must always be evaluated acknowledging the nature of layering and carefully selecting the mechanical parameters of the soil.

Keywords: Pumping Settlement, Specific Storage Coefficient, Young's Modulus, Soil Displacements, Groundwater.

1. Introduction

Pumping groundwater from porous media promotes soil contraction that produces subsidence (settlements) (Pacheco-Martínez et al., 2013; Conway, 2016). The issue is of great concern and numerous authors have reported large settlements produced by pumping (Hsi and Small, 1992; Gue and Tan, 2004; Roy and Robinson, 2009; Xu et al., 2012) and associated impacts (e.g., cracks) on buildings (Forth 2004). Famous cases where continuous pumping has induced large subsidence include Mexico City (Carrillo, 1947; Ortega-Guerrero et al., 1993) or Venice (Gambolati and Freeze, 1973; Carbognin et al., 1995). As a result, some engineers tend to avoid pumping in urban environments. Yet, pumping is sometimes mandatory, e.g. when underground constructions must be undertaken (Wu et al., 2015a and 2015b). In these situations, settlements have to be predicted in advance (Zhang et al., 2013) and alternative techniques may be required to minimize risks (Wu et al., 2015c). However, predicted settlements are sometimes higher than observed. Differences between measurements and theoretical predictions could be attributed to the plastic behavior of weakly consolidated soils. However, differences have also been observed in overconsolidated sediments that behave elastically (Tessitore et al., 2016; Faunt et al., 2016). This type of sediments is relatively common (Zhang et al., 2014; Ye et al., 2016) because overconsolidation may result not only from geologic unloading, but also from aquifer overexploitation, which was frequent in numerous cities during early stages of industrialization (Vazquéz-Suñé et al., 2005).

This research arose during the construction of the tunnel for the High Speed Train tunnel in Barcelona (Pujades et al., 2015) where pumping was mandatory to excavate some shafts. Pumping settlements were a source of concern, although shallow multilayered aquifers in Barcelona (see Culí et al. (2016) for a detailed description) are overconsolidated and the preconsolidation stress would not be exceeded.

Overconsolidation in Barcelona (Spain) resulted from groundwater extraction for industrial purposes during the early 20th century, which produced drawdowns up to 15 m. Afterwards, during the late 1970's, pumping was reduced and the piezometric head recovered (Vazquez-Suñé et al., 2005). To avoid unexpected problems during the High Speed Train tunnel construction, land subsidence was estimated in advance and soil displacements were monitored. It was observed that measurements rarely agreed with the theoretical predictions. This frequent mismatch motivates this study, which is exclusively based on pumping settlements occurred in overconsolidated porous media. Thus, the theory of elasticity, which is useful under restrictive conditions (linear elasticity, isotropy, homogeneity), is the only considered. This theory is not applicable in the most of real soils, especially in compressible, normally consolidated and saturated clays, which cause great deformations (e.g. Mexico City, San Joaquin Valley, etc.). In these cases, the most of the deformation is concentrated in low conductivity layers that are normally consolidated. These layers are slowly drained and after long drainage periods cumulative settlements can be very large (order of meters). Contrary, settlements occurred in elastic media are much smaller and generally their magnitude is inconsequential. However, low deformations in urban environments are a source of concern because small differential settlements can damage surface or underground infrastructures. The results of this work are neither useful if the preconsolidation stress was largely exceeded during the pumping because the soil would not behave elastically (see Loáiciga, 2013).

Potential reasons for discrepancies between predicted and observed settlements include assumptions regarding (1) the direction of soil displacements, (2) the influence of surrounding layers, and/or (3), the effective stresses distribution around pumping wells and their relationship with drawdown. Heterogeneity of the soil deformation properties

could also produce discrepancies between predicted and measured settlements, but this is not considered in this study.

Over the last 60-70 years, the standard assumption to compute settlements has been that pumping provokes displacements only in the vertical direction (Gambolati and Freeze, 1973; Helm, 1975; Leake and Prudic, 1991; Burbey, 2001), which immediately leads to:

$$\Delta z = S_{se} \Delta p b , \quad (1)$$

where Δz [L] is the settlement, S_{se} [L^{-1}] is the portion of the specific storage coefficient associated to soil deformation, b is the thickness of the porous medium [L] and Δp is the drop in pressure head [L]. Although pumping can produce horizontal displacements, it is generally accepted that their magnitudes are negligible (Burbey, 2001). However, field observations (Poland and Davis, 1969; Yerkes and Castle, 1969; Wolff, 1970; Lofgren, 1978a and 1978b) and analytical studies (Bear and Corapcioglu, 1981a) have demonstrated that horizontal displacements can be comparable to settlements. The discussion by Hsieh and Cooley (1995) on the work by Helm (1994) demonstrated that computed (horizontal and vertical) displacements can be very sensitive to the mechanical boundary conditions (BCs) and to the surrounding formations. Ironically, they are rarely considered in predictions. In fact, no studies have compared the behavior of pumped porous media under all combinations of realistic mechanical BCs. For example, Hsieh and Cooley (1995) compared only two different scenarios by varying the mechanical BCs, one of which was not realistic because it was purposefully based on Helm's hypothesis (Helm, 1994). Similarly, Burbey and Helm (1999) only compared two scenarios that considered vertical or three-dimensional displacements. Helm (1994) assumed that pumping causes only horizontal displacements to explain situations where vertical displacements are not observed when pumping. However, other mechanisms can

explain this behavior. For example, upper confining layers could alter pumping settlements towards the surface causing differences between estimated and measured settlements, but they are not considered when pumping settlements are predicted.

Inaccurate predictions can also be a consequence of the assumptions underlying analytical solutions. For example, Eq. (1) implicitly assumes that the change in effective stresses induced by pumping is isotropic and equal to Δp . However, Davis et al. (1969), Burbey (1999), Burbey and Helm (1999) and Ye et al., (2016) noticed that the distribution of settlements near a well displays a concave shape, which does not match the drawdown curve. Davis et al. (1969) attributed the unexpected shape to heterogeneities, whereas Burbey (1999), Burbey and Helm (1999) and Ye et al., (2016) did not place much emphasis on this behavior.

The contraction of pumped elastic porous media depends on their specific storage coefficient (S_s) given by (Jacob, 1950):

$$S_s = \rho_w g (\alpha + \theta \beta), \quad (2)$$

where ρ_w is the density of water [ML^{-3}], g is the gravitational constant [LT^{-2}], θ is the porosity [-], α is the soil compressibility coefficient [LT^2M^{-1}], and β is the water compressibility coefficient [LT^2M^{-1}]. S_s expresses the deformability of elastic porous media (overconsolidated) and can be used to derive soil compressibility, which is necessary for predicting pumping settlements. Obviously, soil compressibility can be obtained from laboratory tests, but tested samples may not be representative and/or can be altered during the extraction process. Field pressurometer tests can also be used, but the obtained values often suggest a material that is softer than the real one (Pujades et al., 2014a and 2014b). However, S_s , which can be easily computed from pumping tests applying the Cooper and Jacob method (1946), is representative and is not altered by

extraction processes. Therefore, mechanical parameters derived from S_s should be suitable for settlements quantification in elastic porous media. Nevertheless, boundary constraints are not considered in the interpretation of pumping tests and besides, Gambolati (1977) affirmed that pumping tests overestimate S_s . Consequently, the suitability of mechanical parameters obtained from pumping tests is investigated in this work.

The summary of this introductory discussion is that, whereas the basic concepts controlling soil displacements caused by pumping were well established by the 1990's (Prince and Leake, 1997), actual practice has remained relatively unchanged, which may reflect both the complexity of fully coupled poroelastic simulations and the numerous factors that must be considered to evaluate pumping deformations. The objective of this work is to examine the influence of boundary constraints and the presence of confining layers on (1) S_s , (2) the settlements and (3) the direction of the displacements.

2. Problem statement

2.1 Governing equations

The mechanical and the hydraulic problems are coupled and must be solved together to analyze the displacements generated by pumping. To solve the mechanical problem, the momentum balance must be satisfied. If inertial terms are neglected, momentum balance reduces to an equilibrium statement:

$$\nabla \cdot \boldsymbol{\sigma} + \mathbf{f} = 0, \quad (3)$$

where $\boldsymbol{\sigma}$ represents the total stress tensor, and $\mathbf{f}$ represents the body force vector.

Under the assumption of small deformations in a homogeneous and isotropic medium, the constitutive equations result in

$$\boldsymbol{\sigma}' = 2G\boldsymbol{\varepsilon} + \lambda\varepsilon_{vol}\mathbf{I}, \quad (4)$$

where $\boldsymbol{\sigma}'$ is the effective stress tensor ($\boldsymbol{\sigma}' = \boldsymbol{\sigma} - p\mathbf{I}$, where we implicitly assume the Biot's coefficient to be one, and $\varepsilon_{vol} = \varepsilon_x + \varepsilon_y + \varepsilon_z = \nabla \cdot \mathbf{u}$ is the volumetric strain, $\boldsymbol{\varepsilon} = (\nabla \mathbf{u} + (\nabla \mathbf{u})^T)/2$ is the strain tensor, $\mathbf{u}$ is the displacements vector, $G = E/(2(1+\nu))$ is the shear modulus, $\lambda = E\nu/((1+\nu)(1-2\nu))$ is Lamé's first coefficient, and p is the fluid pressure. The substitution of Eq. (4) into Eq. (3) leads to the Biot theory of poroelasticity (1941), which can be written as (Verruijt, 1969):

$$G\nabla^2 \mathbf{u} + (G + \lambda)\nabla(\nabla \cdot \mathbf{u}) = \nabla p, \quad (5)$$

where $\nabla^2 = \partial^2/\partial x^2 + \partial^2/\partial y^2 + \partial^2/\partial z^2$. Eq. (5) is related to the fluid mass balance equation through the volumetric strain. Assuming saturated conditions and neglecting the compressibility of the individual grains, the fluid mass balance equation (Eq. 6) can be written as

$$\frac{\phi}{K_f} \frac{\partial p}{\partial t} + \frac{\partial \varepsilon_{vol}}{\partial t} + \frac{1}{\rho} \nabla \cdot (\rho \mathbf{q}) = f_w, \quad (6)$$

where ϕ is the porosity, $1/K_f$ is the water compressibility, t is the time, $\mathbf{q}$ is the water flux, and f_w is an external supply of water. The water flux is given by Darcy's Law:

$$\mathbf{q} = -\frac{k}{\mu} (\nabla p + \rho g \nabla z), \quad (7)$$

where k is the intrinsic permeability, z is the vertical coordinate, and μ and ρ are the fluid viscosity and density, respectively.

2.2 The flat case

Bear and Corapcioglu (1981a) wrote the above equations in cylindrical coordinates and performed a vertical integration under the assumption of plane stress ($\Delta \boldsymbol{\sigma}'_{zz} = \Delta p$) and axial symmetry ($\tau_{r\theta} = \tau_{z\theta} = \tau_{rz} = 0$) to obtain

$$2G \frac{\partial}{\partial r} \left(\frac{\partial \bar{u}_r}{\partial r} + \frac{\bar{u}_r}{r} \right) + \lambda \frac{\partial \bar{\varepsilon}_{vol}}{\partial r} = \frac{\partial \bar{p}}{\partial r}, \quad (8)$$

where u_r are the displacements in the radial direction, and $\bar{u}_r$, $\bar{\varepsilon}_{vol}$ and $\bar{p}$ are the vertically averaged values of u_r , ε_{vol} and p , respectively. Integration of Eq. (8) with a constant G and λ yields

$$2G \left(\frac{\partial \bar{u}_r}{\partial r} + \frac{\bar{u}_r}{r} \right) + \lambda \bar{\varepsilon}_{vol} - \bar{p} = g(t), \quad (9)$$

where $g(t)$ is an arbitrary function. Following Bear and Corapcioglu (1981a), the integrated solutions under steady state conditions and under the stated assumptions result in

$$\begin{aligned} \varepsilon_z &= \frac{\Delta z}{b} = \frac{1}{2} \frac{1}{\lambda + G} \Delta p \\ u_r &= \frac{1}{4} \frac{1}{\lambda + G} \Delta p \cdot r \end{aligned} \quad (10),$$

where Δp can be replaced (in Eq. 10) by the Thiem equation under a steady state:

$$\varepsilon_z = \frac{\Delta z}{b} = \frac{\rho g}{2(\lambda + G)} \frac{Q}{2\pi T} \ln \left(\frac{R}{r} \right), \quad (11)$$

where Q is the total pumping rate [L^3T^{-1}], R is the influence radius of the pumping [L], r is the distance between the well and the site where the strain is computed [L], and T is the transmissivity of the porous medium [L^2T^{-1}] ($T = Kb$, where K is the hydraulic conductivity of the medium [LT^{-1}]). S_s can be introduced in Eq. (11) if it is assumed that the porous medium behaves elastically. Considering the characteristics of the problem (flat geometry and axial symmetry) and if pumping displacements occur in the vertical and horizontal directions, then $\alpha = 1/(\lambda + G)$ (Bear and Corapcioglu, 1981a) that can be substituted into Eq. (2), which yields

$$S_s = \frac{\rho g}{(\lambda + G)} + \rho g \phi \beta, \quad (12)$$

where the first term on the right hand side is the portion of the specific storage coefficient associated to soil deformation ($S_{se} = \rho g / (\lambda + G)$), and the second term on the left hand side is the portion of the specific storage coefficient associated to water ($S_{sw} = \rho g \phi \beta$). For practical purposes, S_{sw} can be neglected because shallow draining soils are typically two or three orders of magnitude more compressible than water (Scott, 1963 cited by Helm, 1994). Thus,

$$S_s \approx S_{se} = \frac{\rho g}{(\lambda + G)}, \quad (13),$$

which can be substituted into Eq. (11), yielding

$$\varepsilon_z = \frac{\Delta z}{b} = \frac{S_{se} Q}{4\pi T} \ln\left(\frac{R}{r}\right). \quad (14)$$

If the pumping displacements only occur in the vertical direction, $\alpha = 1/(\lambda + 2G)$ (Bear and Corapcioglu, 1981b). Thus,

$$S_s \approx S_{se} = \frac{\rho g}{(\lambda + 2G)}. \quad (15)$$

Note that the definition of α provided by Bear and Corapcioglu (1981a) for a flat porous medium with axial symmetry is somewhat inconsistent with the traditional definition of compressibility, which should yield $\alpha = 1/(\lambda + 2G/3)$. Therefore, in general

$$S_s \approx S_{se} = \frac{\rho g}{(\lambda + 2G/3)} \quad (16)$$

2.3. Characteristics of the problems

The conceptual model is identical to that of Bear and Corapcioglu (1981a) but acknowledging that deformation may not be constant in the vertical direction, so as to be able to generalize the results, identify the main factors controlling pumping settlements and to define their role. Two reference cases are analyzed: (1) the “completely confined

problem” and (2) the “semiconfined problem”. Additional scenarios are derived from the reference cases to study the influence of each factor separately.

2.3.1. Completely confined problem

Figure 1a displays a vertical section of the geometry. A constant flow rate Q is pumped through the well, which is screened along the entire porous medium thickness b . The initial head is enough to ensure that the medium is saturated during the simulated time. Head is prescribed at the outer boundary. The top and bottom boundaries are assumed impervious. Under those conditions, flow obeys initially the Theis solution ($t \ll t_c$, where $t_c = S_s R^2 / K$ is a characteristic time) and the Thiem equation under a steady state ($t \gg t_c$). This problem is used to examine the relationships among (1) the mechanical BCs, the displacements and S_s , (2) the mechanical parameters and S_s , and (3) the drawdown and the pumping settlements.

2.3.2 Semiconfined problem

This problem is similar to the previous one, with the addition of a poorly conductive layer of thickness b_{pcl} , located at the top of the pumped porous medium (vertical section is displayed in Figure 1b). The only difference between the two layers is the hydraulic conductivity. The other parameters such as S_s , E or the Poisson’s ratio are the same. The pumping well is screened only in the lower (and more conductive) layer and pumps a constant flow rate Q . The poorly conductive layer is also homogeneous and isotropic. Head is prescribed at the outer boundary, and the top and bottom boundaries are assumed impervious. This problem allows an understanding of the effects of upper (and lower) poorly conductive layers on pumping settlements. The drainage of the confining layer

introduces a new characteristic time ($t_{Cpcl} = S_{Spcl} b_{pcl}^2 / K_{pcl}$, where S_{Spcl} and K_{pcl} are the specific storage coefficient and the hydraulic conductivity of the poorly conductive layer, respectively).

3. Methods

The stated objectives are achieved by applying analytical and numerical procedures. The Bear and Corapcioglu (1981a) analytical solution (see Section 2) is used to calculate the pumping settlements. Numerical simulations are performed by means of the finite element numerical code CODE_BRIGHT (Olivella et al., 1994; Olivella et al., 1996), which solves coupled thermo-hydro-mechanical processes in porous media. The characteristics of this code are explained in detail in CODE_BRIGHT user's guide (ETCG-UPC, 2016). The problems are studied in steady and transient states. The considered values of the hydrogeological and mechanical parameters (Table 1) are typical of overconsolidated sedimentary soils (Pujades et al., 2014a and 2014b). Notice however that, given the linearity of the problem, the actual values of the parameters are not relevant. Specifically, as can be deduced from Eq. 5 or Eq. 14, settlements are inversely proportional to the Young's modulus. Given the axial symmetry of the problem, a rectangular mesh is adopted, which allows reducing the number of elements and the computation times. The mesh for the completely confined problem (Figure 1) consists of 20 and 100 elements in the vertical and horizontal direction, respectively, with a greater refinement towards the pumping well (the elements closer to the pumping well have horizontal sizes of 0.1 m). The same mesh is adopted to model the lower layer of the semiconfined problem (Figure 1), where the mesh that covers the confining layer is discretized into 10 and 100 elements in the vertical and horizontal directions, respectively. The validity of the mesh has been verified by ensuring that results are not sensitive to

mesh refinements. The hydraulic BCs (Figures 1a and 1b) are: (1) prescribed pumping rate at the inner boundary (fully penetrating pumping well), (2) prescribed zero drawdown at the outer boundary, and (3) no flow at the top and bottom boundaries. This last BC assumes that groundwater provided by layers that overlay or underlay the modeled domain is negligible. Thus, no vertical flow occurs in the completely confined problem, whereas in the semiconfined problem, the upper layer provides water vertically to the lower layer. Regarding the mechanical BCs: zero normal displacement is applied at the pumping well (i.e. the inner boundary cannot deform), whereas equilibrium of stress is applied at the top boundary simulating the absence of constraints to displacement (i.e., the top boundary is free to deform). Different combinations of mechanical BCs are applied at the outer and lower boundaries, defining various scenarios for the completely confined problem that are presented in the following. Moreover, additional scenarios consider the variation in the thickness of the porous medium b for the completely confined problem and the variation in the hydraulic conductivity of the confining layer for the semiconfined problem.

The analysis and corresponding scenarios are summarized as follows:

1) The directions of the displacements and the influence of the mechanical boundary constraints are analyzed using the numerical results of four different scenarios (Sce1 to Sce4) of the completely confined problem. These scenarios differ in the application (or not) of a mechanical constraint to the normal displacements at the outer and lower boundaries (Figure 2). All the important and realistic combinations of mechanical BCs are considered. Mechanical BCs simulate the properties of the materials that surround the porous medium. Thus, the influence exerted by a stiff bedrock or layer is simulated constraining the horizontal displacements at the bottom boundary, while if the bedrock is soft the horizontal displacements are not limited. Similarly, if the soil is

laterally surrounded by a stiff formation, the horizontal displacements are constrained at the outer boundary. Sce1, which allows sliding but no normal displacements, is adopted as the reference scenario to evaluate the influence of the other variables.

2) The pumping settlements and vertical strain are first studied by varying the thickness of the porous medium (Sce5 to Sce9) with respect to Sce1 (Table 2) and comparing the steady state results. The same unit pumping rate (pumping rate per meter of thickness) is adopted to ensure that Δp is equal in all the scenarios ($q_u = 5.4 \text{ m}^2/\text{d}$). Second, the numerical vertical strain, ε_z , from Sce1 and Sce2 are compared with those obtained from Eq. (14).

3) The impact of poorly conductive layers on settlements is studied by analyzing the numerical results for the semiconfined aquifer problem. Two scenarios are simulated (Sce10 and Sce11), which consider different values of hydraulic conductivity for the confining layer (Table 2).

4) S_s , its relation to the mechanical parameters of the porous medium and its dependence on the boundary constraints are analyzed using the numerical results from Sce1, Sce2 and Sce3. S_s is computed using three different methods: (1) dividing ε_{VOL} by Δp , thus reflecting the definition of S_s as change in volume of stored water per unit volume of porous medium and unit change in head (steady state results), (2) using the Cooper and Jacob method (1946) (transient results), and (3) using the mechanical parameters of the porous medium. Two hypotheses are considered to apply the latter method: (1) displacements occur in all directions, and (2) displacements only occur in the vertical direction. Procedures 1 and 3 yield S_{se} , whereas procedure 2 yield S_s , which is taken into account when results are compared (although S_{sw} can be usually neglected).

4. Results

4.1. Direction of the displacements and the influence of the mechanical BCs

The steady state distributions of horizontal displacements and settlements versus the logarithm of the distance from the pumping well (r) are shown in Figure 3. Negative values of horizontal displacements indicate that the soil moves toward the pumping well, whereas negative settlement values mean that the soil settles.

The mechanical BCs noticeably affect the resulting displacement field. The most perceptible impact on the horizontal movements is observed when they are limited (or not) at the lower boundary (Figure 3a). This condition depends on the stiffness of the material underlying the pumped porous medium. In fact, the presence of a stiff material limits the horizontal movements, as discussed by Hsieh and Cooley (1995). In such case (Sce3 and Sce4), the horizontal movements are approximately 0, whereas if the underlying material is soft (Sce1 and Sce2), the magnitude of the horizontal movements is comparable to the magnitude of the settlements (i.e., the horizontal movements are not negligible). The mechanical constraints adopted at the lower boundary also influence the settlements. In fact, the magnitude of settlements and the slope of their distribution increase if the horizontal movements are constrained (Figure 3b). This response is reasonable because the volumetric deformation should be similar in all cases. In radial coordinates, $\varepsilon_{\text{vol}} = \nabla \cdot \mathbf{u} = (1/r) \partial \mathbf{u}_r / \partial r + \partial \mathbf{u}_z / \partial z$, a reduction in horizontal displacements is compensated by an increase in vertical deformation. The opposite is also true. A small heave (upwards displacement) is observed near the outer boundary in Sce2 to compensate the compressive horizontal strain, with virtually zero drawdown.

The effect of constraints at the outer boundary is only relevant when horizontal displacements at the bottom boundary are free, that is, by comparison of Sce1 and Sce2. When they are hindered at the bottom boundary, vertical displacements are very similar

(Sce3 and Sce4, but see also Hsieh and Cooley, 1995) because of the same reason as discussed in the previous paragraph (volumetric deformation is not sensitive to the outer B.C. and can only be caused by vertical strain when horizontal displacements are impeded at the bottom boundary). The horizontal displacements of Sce1 and Sce2 are equal near the well and diverge away. In Sce1, the horizontal displacements increase from the well to a distance of 700-800 m, where maximum (in absolute value) displacements towards the pumping well are observed, and decrease further away until the outer boundary, where they are 0. A maximum can also be observed in the horizontal displacements of Sce2, but closer to the outer boundary ($r \approx 1500$ m) than in Sce1. The large displacements of Sce2 are a consequence of the BCs and the radial geometry of the problem, which cause most of the aquifer to contract and even a small soil heave near the edge (Yeh *et al.*, 1996; see also discussion of Helm, 1994, and Hsieh and Cooley, 1995). The mechanical BCs adopted at the outer boundary of Sce2 do not constraint horizontal displacements. As a result, the whole aquifer should contract horizontally. However, this is not possible because the reduction of the water pressure in the edge of the modeled domain is virtually zero. As a result, the soil response to avoid the reduction of the water pressure in the edge consist in 1) a horizontal displacement near the boundary on the opposite direction of the well and 2), an increase of the soil thickness (heave) because of the Poisson effect.

The subsequent scenarios are constructed using the mechanical BCs adopted for Sce1. Scenarios Sce3 and Sce4 do not appear to be representative of the common behavior of pumped porous media because horizontal movements caused by pumping have been reported in the field (Davis *et al.*, 1969). In addition, the Noordbergum effect (Kim and Parizek, 1997) is induced by horizontal movements that occur in aquitards that surround pumped aquifers (Verruijt, 1969, see also Berg *et al.*, 2011). The mechanical BCs of Sce2 are not considered because of the heave observed at the edge of the modeled domain.

However, the common response of pumped porous media is probably a combination of Sce1 and Sce2.

4.2. Distribution of settlements along the porous medium

4.2.1. Numerical results

Pumping settlements are constant near the pumping well (Figure 3b), which indicates that they are not proportional to drawdown. This behavior is analyzed by comparing the computed settlements for six scenarios of the completely confined problem (Sce1 and Sce5 to Sce9), which differ by the thickness of the porous medium (Table 1). Figure 4a displays the steady state settlements versus the logarithm of r . Settlements increase towards the well with a constant slope. However, the slope starts to decrease at a distance that depends on the thickness b of the porous medium, and they are constant (i.e., not differential) near the well. Figure 4b shows the settlements versus a dimensionless distance from the well ($r_d = r/b$). It can be observed that the settlements in all the scenarios cease to increase proportionally to Δp at $r_d \approx 0.7$. This behavior can be attributed to an arch effect (actually, dome effect, given the radial symmetry) that develops around the well, so that the upper portions of the aquifer hold the increase in effective stress caused by the drop on pressure near the well and transfer it to the base of the porous medium at the circular crown between $r \approx 0.12b$ and $r \approx 0.7b$. Obviously, this stress transfer causes differences between the variation in the vertical effective stress ($\Delta \sigma_z'$) and Δp near the well, as can be observed in Figure 5a, which displays the distributions of $\Delta \sigma_z'$ and Δp versus $\log-r$. The values of $\Delta \sigma_z'$ near the well depend on the thickness of the porous medium and differ from Δp for $r < 0.7b$, that is, $r_d < 0.7$.

Figure 5b shows the vertical strains ε_z for the six scenarios versus $\log-r_D$. It is clear that the curve of ε_z is similar to that of settlements. They increase constantly from the outside boundary to $r_D \approx 0.7$. From that distance towards the well, ε_z increases more slowly until becoming constant at approximately $r \approx 0.12b$. Note that, our numerical model does not consider any friction between the soil and the pumping well because the radius of the well is assumed small. If a considerable friction occurs, settlements would decrease just near the well.

4.2.2. Analytical and dimensionless form

The numerical results of ε_z for Sce1 and Sce2 are compared with those computed analytically by applying Eq. (14) (Figure 6a). Recall that Sce1 differs from Sce2 on the outer boundary mechanical BC. Sce3 is not discussed because Eq. (14) considers three-dimensional displacements. The analytical results match the numerical ones only for distances from the well greater than $0.7b$. Thus, another procedure is needed to compute ε_z and the settlements near the pumping wells. ε_z near the pumping well can be calculated from the constant solution of the Bear and Corapcioglu equation (1981a) for $r=0.3b$:

$$\varepsilon_{z(r=r_w \text{ to } r=0.3b)} = \frac{\Delta z}{b} = \frac{S_s Q}{4\pi T} \ln\left(\frac{R}{0.3b}\right). \quad (17)$$

The dashed line in Figure 6a displays ε_z computed using Eq. (17). This solution allows approximating ε_z near the pumping well. However, ε_z between $r = 0.3b$ and $r = 0.7b$ should be interpolated. Therefore, an analytical solution is obtained from the generalized curve of the dimensionless vertical strains (ε_{zd}) to calculate ε_z in the whole aquifer. ε_{zd} can be obtained dividing the numerical ε_z by $S_s Q / 4\pi T$. Imposing that the

dimensionless strain is zero at $r = R$, equals $\ln(0.3b/R)$ for $r < 9b/70$, and displays a smooth quadratic transition in between, we get

$$\varepsilon_{zd} = \frac{4\pi T \varepsilon_z}{S_s Q} = \begin{cases} \ln(0.3b/R) & \text{for } r < 9b/70 \\ \ln(0.3b/R) + \ln^2(70r/9b)/4\ln(7/3) & \text{for } r \geq 9b/70 \text{ and } r < 0.7b \\ \ln(r/R) & \text{for } r \geq 0.7b \end{cases} \quad (18)$$

Vertical strains, ε_z , can be obtained multiplying ε_{zd} by $S_s Q/4\pi T$. The settlement is determined by multiplying ε_z by b . Note that for steady state problems, R can be approximated as $\sqrt{2.25Tt/S}$. Note also that the settlements computed using Equation (18) are a linear function of the pumping rate. Therefore, superposition is applicable and the settlement caused by pumping a number of wells can be obtained by adding up the settlements, computed using this procedure, caused by each well separately. The ε_{zd} calculated by Eq. 18 fits with those computed numerically for Sce1 and Sce2 (see Figure 6b). Therefore, Eq. 18 can be used to predict pumping settlements in overconsolidated porous media if the consolidation is three-dimensional (i.e. the horizontal displacements induced by the pumping are not constrained).

4.3. Semiconfined problem:

The influence that not pumped poorly conductive layers exert on settlements will now be discussed by comparing the numerical results of the two simulations performed on the semiconfined problem, which have different values in the hydraulic conductivity of the confining layer K_{pcl} (Table 2). Figure 7a displays the vertical displacements (u_z) (10 m from the well) versus depth at different times. At early times, the poorly conductive layer mitigates the settlements occurred at the top of the pumped porous medium because the Δp in the confining layer is lower than that in the pumped one. The

attenuation towards the surface decreases with time as the poorly conductive layer is vertically drained (Sce10). The hydraulic parameters of the poorly conductive layer determine the duration of this mitigation effect. If K_{pcl} is very low (Sce11), a long time is necessary to drain this layer. Therefore, the mitigation of the settlements occurs over a long time. Even when steady state is reached (Sce10), the poorly conductive layer slightly mitigates the pumping settlements near the surface. This behavior occurs not only because the poorly conductive layer is not pumped, and Δp is lower, but especially because of the arch (dome) effect discussed earlier. This causes the poorly conductive layer to become compressed radially and axially, but not vertically, so that it has to expand vertically, as can be observed in Figure 7b, which displays ε_z versus depth (10 m away from the well) for several times under Sce10. At early times, ε_z in the poorly conductive layer is positive, which means that the soil heaves. Yet, the radial and axial compression may cause Δp to increase, as extensively discussed by Berg et al. (2011) in the context of hydraulic test interpretation. As the poorly conductive layer becomes drained, the values of ε_z become negative. However, ε_z in this layer is always smaller than that in the pumped one, which reveals that the poorly conductive layer continues to mitigate the settlements.

4.4. Assessment of S_s

S_s is calculated using three methods under Sce1, Sce2 and Sce3, to study the influence of the mechanical properties of the porous medium and the boundary constraints on this parameter. Horizontal displacements (1) are only hindered at the lateral outer boundary in Sce1, (2) occur without any restriction in Sce2, and (3) are impeded at the outer and bottom boundaries in Sce3 (i.e. only vertical displacements occur in Sce3).

Sce4, in which horizontal displacements are only constrained at the bottom boundary, is not considered because its response is identical to that of Sce3. Therefore, the considered scenarios allow the response to a pumping of all the important and realistic combinations of mechanical BCs to be computed.

1) S_{se} is computed according to its definition using the steady state numerical results:

$$S_{se} = \alpha = \frac{\Delta V / V}{\Delta p} = \frac{\varepsilon_{VOL}}{\Delta p} . \quad (19)$$

S_{sw} ($9 \cdot 10^{-7} \text{ m}^{-1}$ with $\theta = 0.2$) is added to the computed S_{se} to obtain S_s . The calculated values of S_s versus the logarithm of the distance are shown in Figure 8. It can be observed that S_s for Sce1 and Sce2 (with different mechanical BCs at the outer boundary) are similar from the well to a distance of $r \approx 100 \text{ m}$. From that distance to the outside boundary, S_s decreases under Sce1 and increases under Sce2. When the horizontal movements are limited at the lower boundary (Sce3), S_s is approximately constant along the porous medium. The values of S_s at distances of $r=r_w$, $r=50 \text{ m}$ and $r=100 \text{ m}$ are summarized in Table 3. The results farther than $r=100 \text{ m}$ are not considered because S_s is considerably influenced by the mechanical BCs on the outer boundary. The values of S_s increase as fewer constraints are applied.

2) S_s is calculated by the Cooper and Jacob method (1946) using the computed drawdowns. The method consists of plotting drawdowns versus the logarithm of the time, drawing a straight line through the late time data points and extending it backward to the point of zero drawdown (time axis intercept), t_0 . S_s is calculated as

$$S_s = \frac{2.25 K t_0}{r^2} . \quad (20),$$

Figures 9a and 9b show the evolution of the drawdown at two observation points located at 50 and 100 m from the well for Sce1 and Sce3. Sce2 is not shown because the drawdown evolution is identical (or very similar) to that of Sce1. The grey straight line is used to find t_0 (time axis intercept). The calculated values of S_s (Table 3) depend on the mechanical BCs and also on the distance where the drawdown is measured. The lowest S_s is obtained by constraining the horizontal displacements at the lower boundary. However, the mechanical BCs at the outer boundary do not affect (or affect slightly) the computed S_s , which decreases as the observation point is moved away from the pumping well.

3) S_s is derived from the mechanical parameters of the porous medium. Two different hypotheses are taken into account. First, displacements occurring in all directions are considered. S_s is then computed with Eq. (2) and applying 1) the definition of α provided by Bear and Corapcioglu (1981a) for a flat geometry with axial symmetry ($\alpha = 1/\lambda + G$), and 2), the traditional definition of α ($\alpha = 1/(\lambda + 2G/3)$). Second, it is assumed that displacements can only occur in the vertical direction. In that case, S_s is obtained from Eq. (2), but with $\alpha = 1/\lambda + 2G$ (Bear and Corapcioglu, 1981b). The resulting values (Table 3) still showed that the storage capacity is greater when horizontal deformations are allowed.

5. Discussion and conclusions

This study investigates the mechanical response of overconsolidated porous media to pumping in order to improve the assessment of settlements and specifically differential settlements. In particular we seek to establish the role of the main variables that influence pumping settlements to improve their quantification and to obtain a fast procedure to

517 predict them. Results are obtained numerically by considering a simplified problem. Still,
518 they can be applied for practical purposes. Geometrical simplifications, such as a constant
519 thickness, are required to generalize the results and allow comparing them with exiting
520 analytical solutions. All logical combinations of mechanical BCs are adopted. Thus,
521 results are representative of real overconsolidated media.

522 The response of the porous medium to pumping is noticeably affected by the
523 stiffness of the surrounding formations, which can constrain the deformation. A stiff
524 bedrock limits the horizontal displacements at the lower boundary. In that case, horizontal
525 displacements caused by pumping are negligible, and settlements increase with respect to
526 the scenarios with soft bedrock because all contraction needs to be vertical. However, the
527 horizontal displacements need not be negligible, as shown by field and analytical studies
528 (Poland and Davis, 1969, Bear and Corapcioglu, 1981a).

529 The stiffness of the lateral surrounding formations affects the horizontal
530 deformation and needs to be addressed (Hsieh and Cooley, 1995). In the case of
531 constrained lateral deformation, the porous medium contracts near the well and expands
532 far from it, whereas when the deformation is not constrained, almost all of the porous
533 medium contracts toward the well. The settlements do not depend too on the mechanical
534 BCs adopted on the outer boundary. They are slightly higher when lateral displacements
535 are not constrained because a small heave that occurs at the edge of the modeled domain.

536 One of the reasons that could explain the mismatch between observed and
537 estimated settlements is that a high hydraulic gradient can occur near the pumping well
538 in layered saturated soils, which would induce a plastic deformation. However, the
539 numerical results show that differences between observations and theoretical predictions
540 near the pumping well also occur in single layer overconsolidated porous media. Contrary
541 to the expected, pumping settlements are not differential from the well to a distance that

depends on the thickness of the pumped porous medium ($r \approx 0.12b$), and they are lower than expected from $r \approx 0.12b$ to $r \approx 0.7b$. Further away (from $r=0.7b$ to the outer boundary), they are differential, but the distribution is as smooth as the drawdown curve. Davis et al. (1969) related this response to heterogeneities. However, our results demonstrate that this behavior is caused by mechanical resistance to deformation (dome effect), so that it also occurs in homogeneous and isotropic porous media. Analytical solutions based on vertical integration do not consider this mechanical response and do not allow settlements to be calculated near the well. Therefore, we propose two procedures to calculate them. The first consists of using the Bear and Corapcioglu (1981a) solution (Eq. 14) for distances greater than $r=0.7b$ and the constant solution of this equation (considering $r=0.3b$) for distances smaller than $r=0.3b$ (Eq. 17). However, the vertical strain from $r=0.3b$ to $r=0.7b$ must be interpolated with this procedure. The second one allows settlements to be computed in the whole porous medium by means an analytical expression of the dimensionless strain. This analytical equation can be used to estimate the pumping settlements for all overconsolidated porous media, regardless of their dimensions, pumped flow rate and material properties. ε_{zd} computed analytically fits with those obtained numerically for Sce1 and Sc2. Differences observed are produced because of the mechanical BCs at the outer boundary that cannot be considered analytically. This procedure is easy to apply and really useful in engineering practice to obtain a preliminary assessment of settlements in real projects. It goes without saying that more sophisticated numerical models should be performed in the final decision making. This is especially relevant in alluvial aquifers where former channels, abandoned meanders or ancient excavation places may have been filled with soft materials.

The most of the pumping settlements in normally consolidated sediments occur in low hydraulic conductivity layers that surround aquifers when their pore pressure is

reduced by vertical drainage. However, the behavior of overconsolidated sediments is different because the system is stiffer, elastic and behaves more as a whole. What causes consolidation and settlement in these sediments is the increase in effective stress, caused by the lowering of water pressure, driven by pressure. Poorly conductive layers are also of paramount importance in the response of overconsolidated soils to pumping because more time is needed to reduce their water pressure. As a result, the horizontal contraction of the conductive layers (whose water pressure is reduced faster) squeezes the poorly conductive layers located above (or below), which causes the pressure to increase and the poorly conductive layers to expand vertically mitigating the settlements. That response, which is one of the causes of the Noordbergum effect (Kim and Parizek, 1997), depends on the hydraulic conductivity of the poorly conductive layer, occurs during early times of pumping and decreases after continuous pumping, when the water pressure in the poorly conductive layer is depleted. This fact must be considered when predicting settlements induced by temporary pumping. Note that the same values of Young's modulus and Poisson's ratio have been considered here for the pumped and confining layers. However, this is not relevant to the objectives of this study. Surface settlements during early times would be probably lower if poorly conductive layers had low Young's modulus and high Poisson's ratio, because the vertical expansion would be higher. Contrary, steady state settlements would be higher as softer is the material that made up the poorly conductive layer.

The relationships among S_s , the mechanical BCs and the mechanical parameters of porous media have been studied by computing S_s using three methods. All methods show that the specific storage of a porous medium is affected by its boundary constraints and, therefore, by the nature of the surrounding formations. The computed values of S_s are greater when displacements are not constrained. If displacements occur in all

directions, the computed values of S_s obtained from the numerical results (method 1) agree with that calculated analytically (method 3) when the definition of α proposed by Bear and Corapcioglu (1981a) is considered, whereas the S_s calculated applying the traditional definition of α is higher. This occurs because the characteristics of our problem match with those considered by Bear and Corapcioglu (1981a) (flat geometry and axial symmetry). When the displacements occur only in the vertical direction the computed S_s using the first and third methods agree. It is observed that, regardless of the deformation constraints, the values of S_s calculated using the Cooper and Jacob method (1946) are smaller. This result contradicts the findings of Gambolati (1977), who stated that pumping tests overestimate the S_s of aquifers. Consequently, if the mechanical parameters are obtained from pumping tests, the pumping settlements would be underestimated. The observed overestimation, which motivated this study, could be explained by the other factors, including the influence of the boundary constraints and confining layers and the fact that settlements are not proportional to the drop of pressure near the well.

Results show that the value of the specific storage coefficient depends on the boundary constraints. However, these are not easy to determine and could be not known in those projects where deep characterization studies are not undertaken. This fact is not unusual, especially in some civil engineering projects where the schedule is too tight. Thus, it would be helpful to determine an area around the pumping well where the value of the specific storage coefficient was stable and not dependent on the outer mechanical BC. If deformation is three-dimensional (i.e. horizontal movements are allowed in the lower boundary), the results show that the outer mechanical BC barely influences the storage coefficient within a dimensionless distance from the pumping well equal to 0.05. This dimensionless distance, which is obtained dividing the real distance by R , is

applicable in all overconsolidated porous media with similar characteristics. In the case of one-dimensional deformation (i.e. horizontal movements are not allowed in the lower boundary), the storage coefficient does not depend on the outer mechanical BC. Still, the most advisable option for the evaluation of the specific storage coefficient consists in deeply characterizing the soil in order to determine the nature of the BCs. Similarly, for modeling purposes and assuming three-dimensional displacements, it is advisable to model a sufficiently extended domain to avoid the impact of the lateral BCs on the horizontal motion close to the well. Alternatively, different scenarios could be modeled covering all the important and realistic boundary combinations. In this case, a range of results based on them would be obtained. A narrow range of results would indicate that boundary conditions do not play an important role, while a wide range would imply that the storage coefficient is highly dependent on the mechanical BCs.

The main contributions of this study are:

1. Demonstrate that the deformation induced by pumping in overconsolidated porous media may be three-dimensional, and therefore, horizontal displacements must be considered when pumping settlements are calculated. The displacements field depends on the mechanical boundary conditions, especially those located below the pumped layer.
2. Propose an analytical solution to calculate pumping settlements in overconsolidated porous media. This solution allows anticipating the domain around the pumping well, in which pumping settlements are not differential.
3. Show that poorly conductive layers can smooth the settlements. This effect is temporal and its duration depends on the hydraulic properties of the poorly conductive layer.

- 642 4. Prove that the specific storage coefficient depends on the mechanical
643 boundary conditions. The storage capacity of saturated soils is greater for
644 weaker displacements constraints.

645
646 **7. Acknowledgements:**

647 E. Pujades and A. Jurado gratefully acknowledge the financial support from the
648 University of Liège and the EU through the Marie Curie BeIPD-COFUND postdoctoral
649 fellowship (2014-2016 and 2015-2017; “FP7-MSCA-COFUND, 600405”). Additional
650 funding was provided by the Spanish Ministry of Science and Innovation (MEPONE
651 project: BIA2010-20244) and the EU project FracRisk (640979).

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Figure caption

Figure 1. Main characteristics of the considered problems. The hydraulic BCs, which do not vary under the different scenarios, are shown.

Figure 2. Mechanical BCs adopted under the four scenarios of the completely confined problem. Roller means fixed normal displacements. Free boundary means that stress remains unchanged (i.e., unconstrained displacement).

Figure 3. a) Horizontal displacement at a horizontal section placed in the center of the porous medium and b) settlement versus the logarithm of r for scenarios Sce1 to Sce4 of the completely confined problem. They differ by their mechanical BCs (Figure 2). Note that settlements and horizontal displacements computed under Sce3 and Sce4 are identical. Note also the slight heave of Sce 2.

Figure 4. a) Settlement versus the logarithm of r for Sce1 and Sce5 to Sce9. The scenarios differ by the thickness of the porous medium (b). b) Settlement versus the logarithm of the dimensionless distance from the well (r_D) for Sce1 and Sce5 to Sce9. r_D is computed by dividing r by b . The results are relative to a horizontal section placed at the top of the porous medium.

Figure 5. a) Effective vertical stress (σ'_z) versus the logarithm of r for Sce1 and Sce5 to Sce9. The variation in the water pressure (Δp) is also displayed. b) Vertical strain (ε_z) versus logarithm of r_D for Sce1 and Sce5 to Sce9. The slopes of the curves change at $r_D = 0.7$. The results (σ'_z , Δp and ε_z) are relative to a horizontal section placed at the center of the porous medium.

Figure 6. a) Vertical strain (ε_z) versus the logarithm of r . ε_z is computed numerically for Sce1 and Sce2 (ε_z is relative to a horizontal section placed in the center of the aquifer) and analytically applying the Bear and Corapcioglu equation (1981a) and the Eq. 17. b)

Dimensionless vertical strain (ε_{zd}) versus the logarithm of r_D . ε_{zd} is calculated analytically applying Eq. 18 and numerically from Sce1 and Sce2.

Figure 7. a) u_z versus the depth measured at different times for Sce10 (lines) and Sce11 (symbols). u_z is relative to a vertical section placed 10 m from the well. K_{pcl} differs between the two scenarios. b) Vertical strain versus depth at different times for Sce10. Strain is relative to a vertical section placed 10 m from the well.

Figure 8. S_s computed from the numerical results and from Eq. (19) versus r . The results are obtained by vertically averaging the values of S_s along the porous medium.

Figure 9. Drawdown evolution for Sce1 and Sce2 (a) and for Sce3 (b). The drawdown evolutions, which are computed at two observation points 50 m and 100 m from the well, are used to calculate S_s using the Cooper and Jacob method (1946).

Figures

Figure 1

Figure 2

Figure 3

Figure 4

Figure 5

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Figure 7

Figure 8

Figure 9

Table captions

Table 1. Parameters and characteristics of Sce1 to Sce9. The unitary pumping rate is always the same ($q_u=5.4\text{m}^3/\text{d}$). S_s can be computed from E and ν using the equation of Jacob (1950) for elastic porous media. The shaded cells show the modified parameters with respect to Sce1.

Table 2. Parameters and characteristics of Sce10 and Sce11. The scenarios belong to the semiconfined problem. S_s can be computed from E and ν using the equation of Jacob (1950) for elastic porous media.

Table 3. Computed S_s for Sce1, Sce2 and Sce3 using the methods explained in section 3.4. The calculated S_s from the volumetric deformations (method 1) is relative to three points, which are located at $r=r_w$, $r=50$ m, and $r=100$. The value at $r=50$ m is given for comparison with the results computed using the second method. The S_s calculated using the Cooper and Jacob method (1946) (method 2) is relative to the two points located 50 and 100 m from the pumping well. Finally, obtained S_s from the mechanical parameters of the porous medium (method 3) is relative to the entire domain.

	<i>b</i> (m)	<i>K</i> (m/d)	<i>Q</i> (m ³ /d)	<i>E</i> (Mpa)	<i>ν</i>	<i>R</i> (m)
Scenario 1	10	0.9	54	1000	0.35	2000
Scenario 2	10	0.9	54	1000	0.35	2000
Scenario 3	10	0.9	54	1000	0.35	2000
Scenario 4	10	0.9	54	1000	0.35	2000
Scenario 5	30	0.9	162	1000	0.35	2000
Scenario 6	50	0.9	270	1000	0.35	2000
Scenario 7	100	0.9	540	1000	0.35	2000
Scenario 8	300	0.9	1620	1000	0.35	2000
Scenario 9	500	0.9	2700	1000	0.35	2000

Table 1

	Q (m ³ /d)	R (m)	Parameters of the pumped porous medium			
			b (m)	K (m/d)	E (Mpa)	ν
Scenario 10	54	2000	100	0.79	1000	0.35
			Parameters of the poorly conductive layer			
			b_{pcl} (m)	K_{pcl} (m/d)	E_{pcl} (Mpa)	ν_{pcl}
			10	7.9E-05	1000	0.35
Scenario 11	Q (m ³ /d)	R (m)	Parameters of the pumped porous medium			
			b (m)	K (m/d)	E (Mpa)	ν
			100	0.79	1000	0.35
			Parameters of the poorly conductive layer			
Scenario 11	54	2000	b_{pcl} (m)	K_{pcl} (m/d)	E_{pcl} (Mpa)	ν_{pcl}
			10	7.9E-07	1000	0.35

Table 2

	Method 1		Method 2		Method 3
	Distance	S_s (m ⁻¹)	Distance	S_s (m ⁻¹)	S_s (m ⁻¹)
Scenario 1	$r=r_w$	8.2E-06	$r=50$ m	6.8E-06	8.8E-06 with $\alpha = 1/\lambda + G$
	$r=50$ m	8.7E-06			
	$r=100$ m	8.6E-06			
Scenario 2	$r=r_w$	8.4E-06	$r=100$ m	6.5E-06	9.7E-06 with $\alpha = 1/(\lambda + 2G/3)$
	$r=50$ m	9.0E-06			
	$r=100$ m	9.0E-06			
Scenario 3	$r=r_w$	7.3E-06	$r=50$ m	5.1E-06	7.0E-06 with $\alpha = 1/\lambda + 2G$
	$r=50$ m	7.0E-06			
	$r=100$ m	7.0E-06			

Table 3