

PARAMETRIC ANALYSIS FOR A LOW-VOLUME CONCRETE ROAD WITH REINFORCED SUBGRADE SUBJECTED TO SWELLING PRESSURE

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ABSTRACT:

This paper presents the parametric analysis of a combined system of a low-volume concrete pavement, base layer and reinforced expansive soil, subjected to the stresses due to wheel loading and swelling of the soil. A soil-structure Interaction model is considered for the analysis, in which the pavement and the reinforcement are idealized as a finite beam, with plane-strain conditions, whereas a base layer and the expansive soil are modelled as Winkler Springs of different stiffnesses. During the analysis, it was taken into account that the geosynthetic reinforcement, which could be in the form of a geogrid, geocell, or a combination of both, possesses bending stiffness and is placed at the interface between the base layer and the expansive subgrade. The reinforcement layer is subjected to stresses due to wheel load, self-weight of pavement slab and surcharge load of the base layer at the top and the swelling pressure from the subgrade at its bottom. The governing differential equations for the flexural response of the model are derived and a closed-form solution is presented in a non-dimensional form. The outcomes of the parametric analysis highlight that the flexural response of the pavement is predominantly affected by the relative stiffness of the base layer and the expansive subgrade. In contrast, the relative flexural rigidity of the upper and lower beams has a comparatively minor impact on the model's response. Additionally, parameters like the depth of placement of the lower beam, unit weight of the upper soil layer, and the self-weight of the upper beam also contribute to influencing the response of the pavement model. The study suggests that, while designing for the concrete pavement, opting for an increase in the modulus of subgrade of expansive soil may be a preferable choice over reinforcing the foundation of the pavement.

1. INTRODUCTION

Low-volume roads are those which carry average daily traffic of less than 450 Commercial Vehicles Per Day (CVPD). The Indian design code of low-volume concrete pavements recommends the use of a non-expansive subgrade with a minimum CBR of 4% to 8%, to rest the pavement system (IRC:SP62 2014). However, local soils which may be expansive, with low CBR of 2% to 3% in wet conditions, are sometimes used for economic reasons. These pavements would experience stresses due to loading (wheel load and environmental load) and swelling (from the subgrade). Concerning this, various chemical stabilization methods (with lime, cement, additives, industrial waste etc.) and mechanical or physical stabilization methods (such as compaction, pre-wetting, geosynthetic reinforcement etc.) have evolved.

The inclusion of geosynthetic reinforcement in the base layer or subgrade is found to be successful and cost-effective up to some extent, in solving a dual problem of distributing the stresses from wheel loading and dissipa-

tion of excess swell pressure from expansive soil (Chittoori, Mishra, and Islam 2018; Md Ashrafuzzaman Khan et al. 2020; Tamim 2017; Kief, Ph, and Geotechnical 2016; Vessely and Wu 2002; Al-Omari and Hamodi 1991; Zornberg and Gupta 2009; Zornberg, Gupta, and Ferreira 2010). However, most of these studies focus on flexible pavements or unpaved roads. Maheshwari et al. (P Maheshwari, Basudhar, and Chandra 2004) proposed a double-beam model to represent the structure and geosynthetic reinforcement as a beam. Other studies using this model mainly focused on the analysis of strip footings and rails (Priti Maheshwari 2008; Priti Maheshwari and Viladkar 2009; Priti Maheshwari, Basudhar, and Chandra 2006). Notably, subgrade soil in these models was non-expansive. Musa (Musa 2018) proposed an analytical solution for a long beam subjected to swelling pressure using the superposition technique. This solution has practical implications for the design of a load-bearing masonry wall. Khan and Wang (Md Adnan Khan and Wang 2017) and Khan et al. (Md Adnan Khan, Wang, and Sarker 2020) developed a closed-form solution using Winkler Model for the asphalt concrete pavement resting on expansive soil.

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They adopted a virtual load concept to solve the problem using inverse theory for the analysis of stresses. However, they did not consider any reinforcement mechanism in the pavement foundation system. Wang et al. (Wang, Sarker, and Ikra 2018) developed the model considering reinforced asphalt concrete pavement as a Timoshenko beam with expansive soil as a Pasternak model. Sarker et al. (Sarker, Wang, and Khan 2019) proposed a solution for the pavement with geosynthetic reinforcement by applying the theory of inverse analysis and the virtual load method.

Presently, Lytton's method has gained widespread acceptance for the design of pavement slabs on expansive soil from both the Texas Department of Transportation (DOT) and the Federal Highway Administration (FHWA). Lytton et al. (Robert Lytton, Aubeny, and Bulut 2004; R. Lytton, Aubeny, and Bulut 2006) proposed a suction-based approach for the design of rigid pavement resting on expansive soil, which was the compilation of the results of several studies (McKeen 1978; McKeen and McKeen 1984; Aitchison and Holmes 1965; Covar and Lytton 2001; R. L. Lytton 1978; McDowell, Herner, and Woollorton 1956; McKeen and Hamberg 1981). This approach entails the computation of a suction profile within the moisture active zone, utilizing an equation put forth by Mitchell (1980). Furthermore, the index properties of the soil are utilized to determine the Suction Compression Index (γh), a parameter that establishes a correlation between matric suction and volume changes in the soil (Covar and Lytton 2001). The method also investigates different stabilization techniques for expansive soil and explores mitigation methods for both swell and shrinkage. However, the primary focus of the method is to assess reliability and serviceability indices for analyzing pavement performance. The approach involves the use of specialized software to generate results. Other methods, like the Post-tensioning Institute method (Post-Tensioning 2003), which is based on Lytton's approach, are commonly used for designing slabs on the ground on expansive soil.

The review of the literature shows that a simplified mechanical model for a low-volume concrete pavement with reinforced subgrade, considering swelling pressure as a design parameter has not been studied so far. Concerning this, the objectives of the present study are as follows.

- To develop an analytical solution for a mechanical model considering a pavement- geosynthetics - expansive soil interaction, subjected to loading and swelling stresses for a low-volume traffic road.
- To generalise the model by presenting the analytical solution in the non-dimensional form.
- To explore the influence of various parameters on the performance of the flexural response of pavement and the reinforcement through a parametric analysis.

2. MATERIALS AND METHODS

2.1 Statement of problem

Fig.1 shows a concrete pavement resting on a granular base layer (thickness h) and subjected to a concentrated wheel load in the middle of the pavement. Reinforcement is placed at the interface of the granular base layer and the compacted expansive subgrade. As the geosynthetic reinforcement is subjected to a surcharge load at the top and the swelling pressure at the bottom from the soil, it is assumed that bending

will be the predominant mode for the response of the reinforcement (Priti Maheshwari, Basudhar, and Chandra 2006).

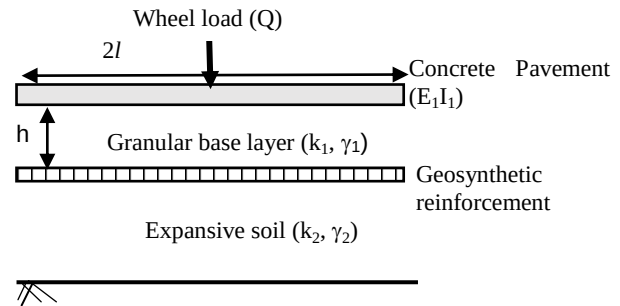


Figure 1. Definition sketch of the problem

2.2 Governing Differential Equations

Fig. 2 shows a mechanical model (plane strain condition) for the problem mentioned in Fig.1. A concrete pavement and geosynthetic reinforcement are idealized as a finite beam (upper and lower beam respectively) of length $2l$ and flexural rigidity E_1I_1 and E_2I_2 respectively. The moduli of subgrade reactions of the base layer and subgrade soil are idealized as Winkler Springs of stiffnesses k_1 and k_2 respectively. A concentrated wheel load Q is acting in the middle of the upper beam, whereas a lower beam is subjected to a surcharge load ($\gamma_1 h + w$) where γ_1 is the unit weight of the base layer soil, h is the depth of placement of the lower beam and w is the self-weight of the pavement beam from the top and swelling pressure P_{sc} from the compacted expansive soil at the bottom.

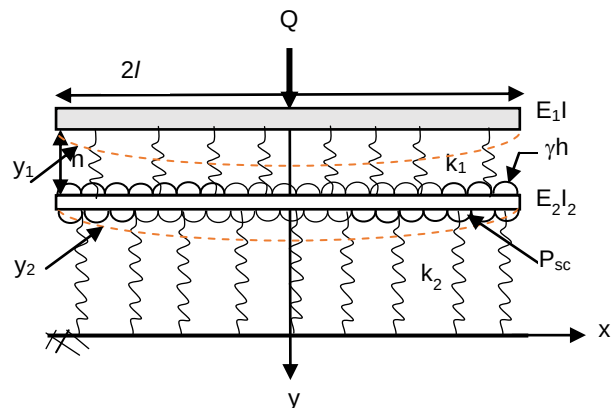


Figure 2. Idealization of the problem into a Mechanical Model

When the width of the pavement is wide enough to model it as a long beam, the swelling pressure at the centre of the beam and the edge of the beam will differ due to the difference in suction at these locations (Perera et al. 2004). However, for smaller widths (half-width is 2m), the change in suction across the length of the lower beam will be uniform. Therefore, swelling pressure at the bottom of the lower beam is assumed to be uniform.

The analysis is to be carried out to evaluate the flexural response of the upper and lower beam considering the wheel loading and the range of swelling pressure. The effect of various parameters such as the normalized depth of placement of reinforcement (h'), the relative flexural rigidity of the upper and

lower beam (R), the relative stiffness of base-layer and subgrade soil (r) and the normalized self-weight of the upper beam (w') on the response of the model are to be studied.

The governing differential equations are derived in this section, for the proposed model and the closed-form solution is presented. Owing to the symmetry, only one-half of the model ($x \geq 0$) is considered for the analysis. The weight of the upper beam w is considered uniformly distributed on the base layer below. The deflection ordinates of the upper and lower beam are denoted as y_1 and y_2 respectively. The reaction force for the upper and lower beams is expressed as $P_1 = k_1(y_1 - y_2)$ and $P_2 = k_2 y_2$ respectively. Sign conventions are taken as positive for downward forces and negative for upward forces.

For the region ($0 \leq x \leq l$)

The governing differential equation for the upper beam:

$$I_1 \frac{d^4 y_1}{dx^4} = -P_1 = -k_1(y_1 - y_2)$$

Solving Eq. (1) for y_2 , we get Eq. (2):

$$y_2 = \frac{E_1 I_1}{k_1} \frac{d^4 y_1}{dx^4} + y_1$$

The differential equation for the deflection of the lower beam is

$$E_2 I_2 \frac{d^4 y_2}{dx^4} = P_1 + \gamma_1 h + w - P_2 - P_{sc} = k_1(y_1 - y_2) + (\gamma_1 h + w) - k_2 y_2 - P_{sc} \quad (3)$$

Simplifying the above equation to Eq. (4)

$$E_2 I_2 \frac{d^4 y_2}{dx^4} = k_1 y_1 - (k_1 + k_2) y_2 + (\gamma_1 h + w) - P_{sc} \quad (4)$$

Substituting y_2 from Eq. (2) to Eq. (4), we get

$$E_2 I_2 \frac{d^4}{dx^4} \left(\frac{E_1 I_1}{k_1} \frac{d^4 y_1}{dx^4} + y_1 \right) = k_1 y_1 - k_1 \left(\frac{E_1 I_1}{k_1} \frac{d^4 y_1}{dx^4} + y_1 \right) - k_2 \left(\frac{E_1 I_1}{k_1} \frac{d^4 y_1}{dx^4} + y_1 \right) + (\gamma_1 h + w) - P_{sc} \quad (5)$$

Arranging Eq. (5)

$$\frac{d^8 y_1}{dx^8} + \left(\frac{k_1}{E_1 I_1} + \frac{k_1}{E_2 I_2} + \frac{k_2}{E_2 I_2} \right) \frac{d^4 y_1}{dx^4} + \frac{k_1 k_2}{E_1 I_1 E_2 I_2} y_1 = \frac{(\gamma_1 h + w) k_1}{E_1 I_1 E_2 I_2} - \frac{P_{sc} k_1}{E_1 I_1 E_2 I_2} \quad (6)$$

Eq. (2) and Eq. (6) are the governing differential equations for the proposed model. These equations are converted into a non-dimensional form using non-dimensional parameters as below. (Matlock and Reese 1960). R_1 and R_2 are the characteristic lengths of the upper and lower beams respectively and are defined as:

$$R_1 = \sqrt[4]{\frac{E_1 I_1}{k_1}} \text{ and } R_2 = \sqrt[4]{\frac{E_2 I_2}{k_2}}$$

$$\text{Let } X = \frac{x}{R_1}, y'_1 = \frac{y_1 E_1 I_1}{Q R_1^3}, y'_2 = \frac{y_2 E_2 I_2}{Q R_2^3}, R = \frac{E_1 I_1}{E_2 I_2}, r = \frac{k_1}{k_2}, \\ \gamma'_1 = \frac{\gamma_1 R_1^2}{Q}, h' = \frac{h}{R_1}, w' = \frac{w R_1}{Q}, P'_s = \frac{P_{sc} R_1}{Q}$$

The governing differential Eq. (2) and Eq. (6) in non-dimensional form can be written as follows:

$$y'_2 = \frac{1}{R} \left(\frac{R}{r} \right)^{3/4} \left(\frac{d^4 y'_1}{dx^4} + y'_1 \right) \quad (7)$$

$$\frac{d^8 y'_1}{dx^8} + \left(R + \frac{R}{r} + 1 \right) \frac{d^4 y'_1}{dx^4} + \left(\frac{R}{r} \right) y'_1 = (\gamma'_1 h' + w' - P'_s) R \quad (8)$$

Eq. (8) can be rewritten as:

$$\frac{d^8 y'_1}{dx^8} + A \frac{d^4 y'_1}{dx^4} + B y'_1 = 0 \quad (9)$$

Where:

$$A = R + \frac{R}{r} + 1 \text{ and } B = \frac{R}{r}$$

Then, eight roots of the auxiliary Eq. (9) will be:

$$m_{1,2,3,4} = (\pm 1 \pm i) \sqrt[4]{\frac{\alpha + \beta}{4}} \text{ and } m_{5,6,7,8} = (\pm 1 \pm i) \sqrt[4]{\frac{\alpha - \beta}{4}}$$

$$\text{Where } \alpha = \frac{A}{2} \text{ and } \beta = \sqrt{\frac{A^2}{4} - B}$$

$$\text{Considering } \sqrt[4]{\frac{\alpha + \beta}{4}} = \lambda_1 \text{ and } \sqrt[4]{\frac{\alpha - \beta}{4}} = \lambda_2$$

The non-dimensional deflection of the upper beam is:

$$y'_1 = e^{\lambda_1 x} (C_1 \cos \lambda_1 x + C_2 \sin \lambda_1 x) + e^{-\lambda_1 x} (C_3 \cos \lambda_1 x + C_4 \sin \lambda_1 x) + e^{\lambda_2 x} (C_5 \cos \lambda_2 x + C_6 \sin \lambda_2 x) + e^{-\lambda_2 x} (C_7 \cos \lambda_2 x + C_8 \sin \lambda_2 x) + (\gamma'_1 h' + w' - P'_s) r \quad (10)$$

The non-dimensional deflection of the lower beam is:

$$y'_2 = \frac{1}{R} \left(\frac{R}{r} \right)^{3/4} (e^{\lambda_1 x} (1 - 4\lambda_1^4) (C_1 \cos \lambda_1 x + C_2 \sin \lambda_1 x) + e^{-\lambda_1 x} (1 - 4\lambda_1^4) (C_3 \cos \lambda_1 x + C_4 \sin \lambda_1 x) + e^{\lambda_2 x} (1 - 4\lambda_2^4) (C_5 \cos \lambda_2 x + C_6 \sin \lambda_2 x) + e^{-\lambda_2 x} (1 - 4\lambda_2^4) (C_7 \cos \lambda_2 x + C_8 \sin \lambda_2 x) + \gamma'_1 h' + w' - P'_s r) \quad (11)$$

The expression for the normalized bending moment of the upper and the lower beam can be obtained by differentiating Eq. (10) and Eq. (11). (5)

2.3 Boundary and continuity conditions

There are eight constants of integration, which need to be evaluated using appropriate boundary and continuity conditions. For the upper beam, at the point of application of load, i.e., at $x = 0$, the slope of the deflected shape of the beam is zero and the shear force is $Q/2$. At the edge of the upper beam, i.e., at $x = l$, the bending moment and shear force are zero for free ends. For the lower beam, which is within the foundation soil, at point $x = 0$, the slope of the deflected shape of the beam and shear force is zero and at $x = l$, the bending moment and shear force are zero. From the above-mentioned boundary conditions, Eq.(12) is presented in the non-dimensional form.

For upper beam:

$$\text{At } x = 0, \frac{dy'_1}{dx} = 0 \text{ and } \frac{d^3 y'_1}{dx^3} = \frac{1}{2}$$

$$\text{At } x = l, \frac{d^2 y'_1}{dx^2} = 0 \text{ and } \frac{d^3 y'_1}{dx^3} = 0$$

For lower beam

$$\text{At } x = 0, \frac{dy'_2}{dx} = 0 \text{ and } \frac{d^3 y'_2}{dx^3} = 0$$

$$\text{At } x = l, \frac{d^2 y'_2}{dx^2} = 0 \text{ and } \frac{d^3 y'_2}{dx^3} = 0 \quad (12)$$

Using the above boundary conditions eight equations are formed. Solving these equations simultaneously using

MATLAB software eight unknown constants (C_1 to C_8) are evaluated. The present mechanical model is based on Hetenyi's solution (Hetényi 1946) of the beam on an elastic foundation and applies the well-established procedure for the development of governing differential equations for the equilibrium of forces and its solution. The only difference is in Hetenyi's model the lower beam is imaginary whereas in the proposed model it is a physical object (Priti Maheshwari, Basudhar, and Chandra, 2006). The validation of the model with Hetenyi's solution is already presented by (Priti Maheshwari, Basudhar, and Chandra, 2006).

To ensure the computational reliability and accuracy of the proposed solution using the MATLAB Program, the algorithm is first compared with the solution presented by Maheshwari et al. (Priti Maheshwari, Basudhar, and Chandra, 2006). This comparison is made without considering the swelling pressure and surcharge load on the lower beam. It is observed that the results show a consistent match as shown in Fig. (3) for the deflection of the beam considering non-dimensional parameters $R=1$, $r=5$ and $x_m=8$. However, to gain confidence in the accuracy of the response of the model, it needs to be verified with laboratory or field experimentation for known swelling pressure and other parameters.

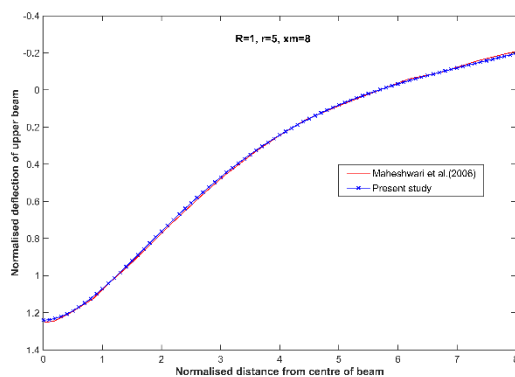


Figure 3. Comparison of deflection of the upper beam with Maheshwari et al. (2006)

2.4 Selection of range of values of design parameters

2.4.1 Soil-suction approach for the computation of swelling pressure Expansive soil tends to swell when it comes in contact with water. Swelling pressure gets exerted on the pavement when the swelling of expansive soil is restricted by it. Reliable prediction of swelling pressure and its impact on the pavement structure is a major challenge to design engineers. Swelling pressure can be computed in the laboratory through different types of oedometer tests or using empirical or semi-empirical models (Mou and Chu 1981; Pedarla et al. 2012; Tu and Vanapalli 2016; Erzin and Erol 2007; Fredlund, Rahardjo, and Fredlund 2012; Singhal, Houston, and Houston 2015). These models have shown that the matric suction (negative pore water pressure) is the most relevant parameter to characterize the swelling behaviour of soil and the maximum potential swelling pressure depends on the initial condition of the soil such as the initial matric suction, dry density, degree of saturation etc. (Mallik, Chandra, and Singh 2006) Tu (2015) recognized that there is a strong relationship between the Soil-Water Characteristic Curve (SWCC) and the variation of swelling

pressure for the values of initial matric suction. Tu and Vanapalli (2016) proposed two different equations to estimate the swelling pressure exerted by natural and compacted expansive soil, based on the SWCC. The equations are helpful to estimate the maximum swell pressure soil can experience, for all possible initial suction ranges and degree of saturation, knowing the conditions of soil at the time of construction of pavement. Though the proposed model can handle any range of swelling pressure, for the present analysis, the range of the swelling pressure of compacted expansive soil (P_{sc}) is taken as 50 to 500 kPa.

2.4.2 Stiffness of upper and lower soil layer soil

The base layer (upper soil layer in the proposed model) is generally compacted granular soil, but any non-swelling local soil can be used to support the pavement. The range of values of stiffness for an upper soil layer (k_1) is considered from 10000 kPa/m to 50000 kPa/m for the present analysis.

The modulus of subgrade reaction or stiffness of soil is an important parameter in pavement design. It is generally determined by carrying out a Plate Load Test or can be correlated using equations based on the soaked CBR value or Dynamic Cone Penetrometer Test of soil (IRC:SP62 2014). However, these correlations are not rational for the expansive soils as the behaviour of expansive soil depends on the amount and type of clay content (montmorillonite, kaolinite, illite etc) in the soil. Payne and Cameron (Payne and Cameron 2014) presented 'The Walsh method of the swell-stiffness' in the form of a bi-linear curve for swelling clay soil. Where swell stiffness is the pressure required to suppress a meter of heave in units kPa/m. For beams in contact with the soil, the swell stiffness can be calculated as $100q$ kPa/m, where q is the average loading pressure on the soil including the self-weight of the structure (Standards Australia 2011). The stiffness of soil (k_2) can also be evaluated by knowing the value of the modulus of elasticity (E_2) of clay from Eq.(12)

$$k_2 = \frac{E_2}{b(1-\vartheta^2)} \quad (13)$$

Where: b is the width of the loaded area in m and ϑ is the Poisson's ratio of clay. Vanapalli and Oh (Vanapalli and Oh 2010) presented an equation for the modulus of elasticity considering the matric suction for the soils with a plasticity index $<16\%$. This model is then extended by Adem and Vanapalli (Adem and Vanapalli 2015) for the modulus of elasticity of unsaturated expansive soil based on SWCC and the modulus of elasticity of soil in a saturated condition as shown in Eq. (14).

$$E_{unsat} = E_{sat} \left[1 + \alpha \frac{(u_a - u_w)}{P_a} S^\beta \right] \quad (14)$$

Where E_{unsat} is the modulus of elasticity under unsaturated conditions, E_{sat} is the modulus of elasticity under saturated conditions, $(u_a - u_w)$ is the matric suction, P_a is the atmospheric pressure (101.3 kPa), and S is the degree of saturation. α and β are the fitting parameters. Based on the literature, the range of parameter (k_2) is taken as 1000 kPa/m to 20000 kPa/m for the analysis of the proposed model. However, the value of k_2 can be further increased by various stabilization methods

2.4.3 Other design parameters. Other design parameters considered for the analysis are mentioned below:

Design wheel load (Q) = 50 kN

Length of the upper beam (L) = 1.875 m (Half-width of pavement)

Width of the beam (b) = 1 m (Plane strain condition)

Modulus of Elasticity of upper beam (E_1) = 30000 MPa

Modulus of Elasticity of lower beam (E_2) = 60 MPa - 200 MPa

Thickness of upper beam (t) = 0.16m, 0.2 m, 0.25m

(The range of thickness of the upper beam 0.16m, 0.2 m and 0.25 m, is based on the design of a low-volume road for daily traffic of 50 CVPD, 150 CVPD and 450 CVPD respectively, considering wheel loading and temperature stresses (IRC:SP62 2014).

Unit weight of base layer (γ) = 14 kN/m³, 16 kN/m³, 18 kN/m³

Depth of placement of lower beam = 0.1m, 0.3m, 0.5m, 1.0 m, 1.5m

Table 1 shows the range of values for various non-dimensional parameters based on the above design parameters, considered for the analysis. An algorithm is developed in MATLAB to perform a parametric study.

Sr. No.	Parameter	Symbol	Range of values
1	Relative flexural rigidity of upper and lower beams	$R = E_1 I_1 / E_2 I_2$	150-500
2	Relative modulus of upper and lower soil layer	$r = k_1 / k_2$	1- 50
3	Normalised unit weight of granular soil	γ'	0.28- 0.36
4	Normalised depth of placement of the geosynthetic layer	h'	0.1-1.5
5	Normalised swelling Pressure	P'_s	1-10
6	Normalised self-weight of the pavement slab	w'	.077-0.12

Table 1. The range of values of non-dimensional parameters considered for the analysis

3. CASE STUDY

In this section, a case study of expansive soil at the village Ravet, Pune, India is discussed, illustrating the impact of initial soil suction on soil swelling pressure. Table 2. presents grain-size distribution data and index properties of soil, which are utilized in the generation of the Soil-Water Characteristic Curve (SWCC), depicted in Fig. 4(a), (Fredlund and Xing 1994). The Swelling pressure Initial Suction Curve is then derived as shown in Fig. 4(b), considering the computed SWCC. (Tu and Vanapalli (2016).

In Figure 4(b), it is observed that the swelling pressure reaches a maximum value of 190 kPa when the soil is initially in its driest state, corresponding to the maximum initial soil suction and then gets fully saturated progressively (until suction approaches zero). Conversely, if the initial soil suction is 1000 kPa, as depicted in Fig. 4(b) for an initial volumetric water content of 38%, referring to Fig 4(a), the maximum value of swelling pressure decreases to 24.5 kPa. Similarly, the swelling pressure

is minimal for saturated soil in its initial state. This concept also supports the pre-wetting of soil approach to mitigate swelling pressure after post-construction.

Sr. No.	Soil Properties	Unit	Value	Reference
1.	Liquid Limit (LL)	%	85	(Chavan and Bhosale 2022)
2.	Plastic Limit (PL)	%	36	
3.	Plasticity Index (PI)		49	
4.	Sand content	%	15	
5.	% clay <0.002mm	%	30.16	
6.	Fines content % passing #No.200 ASTM sieve (IS 75μ sieve)	%	83.26	
7	Void Ratio (e_0)		1.13	

Table 2. Index properties of expansive soil at village Ravet, Pune, India

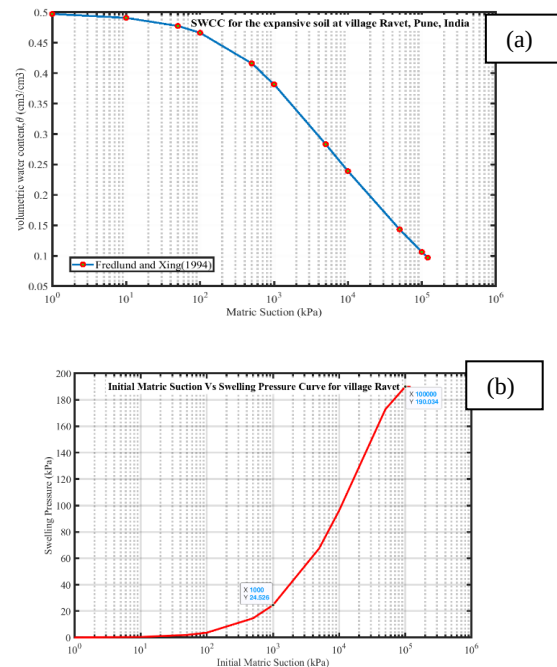


Figure 4 (a) Soil-Water Characteristic Curve and (b) Swelling Pressure Vs Initial suction Curve for expansive soil at village Ravet, India

4. RESULTS AND DISCUSSION

Though the Winkler model shows limitations such as discontinuity in the response of the model at the edge of the beam, it is still used in the design of rigid pavement because of its simplicity. Despite the limitations, the present research is more general and takes care of the practical values of parameters. Some typical results have been presented in this section, to understand the influence of various parameters on the response of the pavement system.

Note that the word 'deflection' used in the discussion below indicates the resultant positive deflection (heave) unless specified.

4.1 Effect of the depth of placement of a lower beam on the deflection of beams

Fig. 5(a) and 5(b), show the influence of the depth of placement of the lower beam h' , on the normalized deformation of the upper and lower beam respectively, for the non-dimensional parameters $P'_s=2$, $R=150$, $r=2$, $\gamma'_1=0.32$, $w'=0.096$.

- As the parameter h' increases from 0.1 to 1.5, the normalised deflection of the upper beam y'_1 , decreases around 29 % at the edge and 32 % at the centre of the beam.
- In the case of the lower beam for the same parameters, when h' increases from 0.1 to 1.5, the maximum normalised deflection for the lower beam y'_2 , is found to decrease around 27 % at the edge and 29 % at the centre.
- It is observed through analysis that when swelling pressure P'_s is equal to 1, the decrease in the deflection due to the increase in h' from 0.1 to 1.5 is 112 % at the centre and 79 % at the edge of the upper beam. Whereas, for the increase in swelling pressure to $P'_s=10$, the decrease in the deflection due to an increase in h' is about 4.7 % at the centre and edge of the upper and lower beam.
- The above result suggests that as the depth of placement of the lower beam increases, deflection decreases due to the increase in the weight of the surcharge.

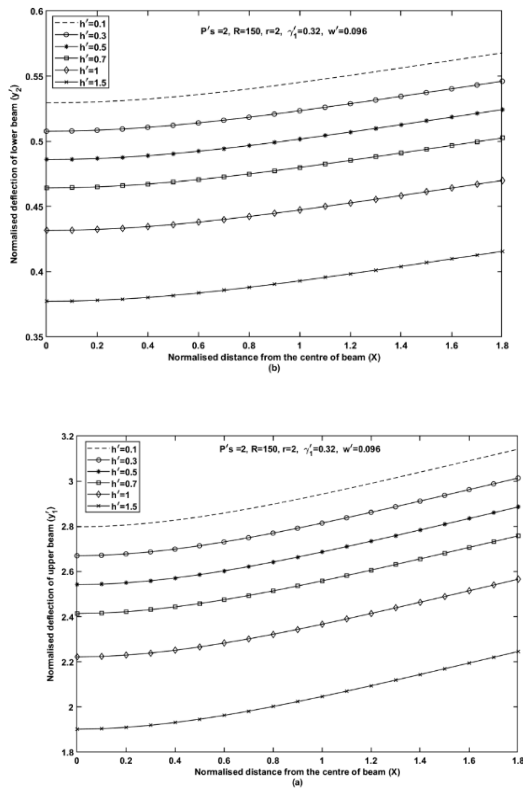


Figure 5. (a) Effect of depth of placement of lower beam on the deflection of upper beam (b) Effect of depth of placement of lower beam on the deflection of lower beam

4.2 Effect of relative flexural rigidity of the upper and lower beam on the deflection of beams

Fig. 6(a) and 6(b), show the influence of the relative flexural rigidity R of the upper and lower beams, on the normalized deformation of the upper and lower beam respectively, for the non-dimensional parameters $P'_s=2$, $h'=0.5$, $r=2$, $\gamma'_1=0.32$, $w'=0.096$.

- It is observed that the decrease in the value of parameter R from 500 to 150, does not affect the normalised deflection of the upper beam. For the lower beam as the parameter R decreases from 150 to 500 deflection increases by 26 % at the centre and edge of the beam. The increase in the swelling pressure $P'_s=1$ to 10, show a similar trend of deflection of the upper and lower beam.
- Results indicate that the decrease in the value of R , which means increasing the rigidity of the lower beam while keeping the rigidity of the upper beam constant, is not found significant to reduce the deflection of the upper beam. This may be due to the presence of a more rigid upper beam, the 'beam effect' of geosynthetic reinforcement does not get mobilised enough under loading and swelling stresses. This means the lower beam just adds the surcharge weight over the swelling pressure without providing the bending stiffness. However, this needs to be verified with experimental studies.

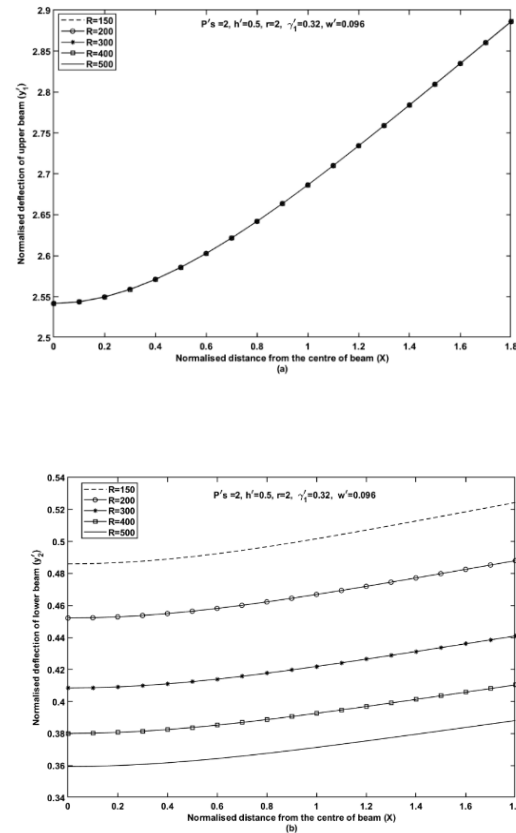


Figure 6. (a) Effect of relative flexural rigidity of beams on the deflection of the upper beam (b) Effect of relative flexural rigidity of beams on the deflection of lower beam

4.3 Effect of the relative stiffness of base layer and subgrade on the deflection and bending moment of the beams

The effect of variation of the relative stiffness of the base layer and subgrade soil r , on the normalized deflection of the upper beam y'_1 and the lower beam y'_2 , is shown in Fig. 7(a) and 7(b) respectively, for non-dimensional parameters $P'_s = 2$, $h' = 0.5$, $R = 150$, $\gamma'_1 = 0.32$, $w' = 0.096$. Fig. 8(a) and 8(b) show the effect of relative stiffness r , on the normalized bending moment of the upper and lower beam respectively.

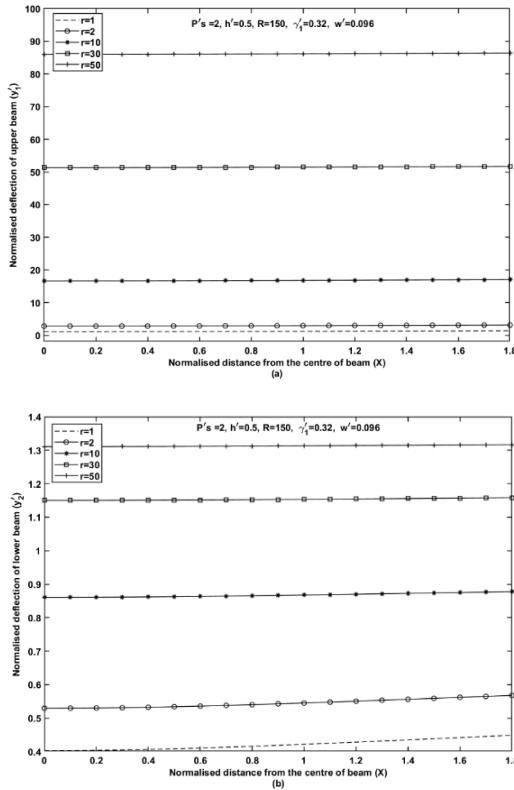


Figure 7. (a) Effect of the relative stiffness of upper and lower soil on the deflection of upper beam (b) Effect of the relative stiffness of upper and lower soil on the deflection of lower beam

- As the value of r decreases from 50 to 1, the normalized deflection of the upper beam y'_1 , decreases by 98 % at the centre and edge of the beam whereas the normalized deflection of the lower beam y'_2 , decreases around 69% at the centre and 66% at the edge of the lower beam. The decrease in percentage remains almost the same with an increase in swelling pressure P'_s from 1 to 10.
- It is also found that the percentage variation in the relative stiffness of the base layer soil and subgrade soil is directly proportional to the percentage variation in the normalized deflection of the upper beam with the constant of proportionality approximately equal to 1. This means that if the value of parameter r increases by 30%, the deflection of the upper beam will increase by around 30 %.

- For the upper beam, the maximum positive bending moment is seen at the centre for $r = 50$ and gradually decreases to zero at the edge of the beam. As the ratio r increases from 1 to 50, the maximum normalized bending moment at the centre of the upper beam increases by 11 %.
- For the Lower beam, the maximum normalized bending moment is observed at the centre for $r = 2$ and gradually decreases and becomes zero at the edge of the beams. As the ratio r increases from 2 to 50, the maximum normalized bending moment at the centre of the lower beam decreases by 78 %.
- The results show that the relative stiffness of the base layer and subgrade soil significantly affects the response of upper and lower beams.

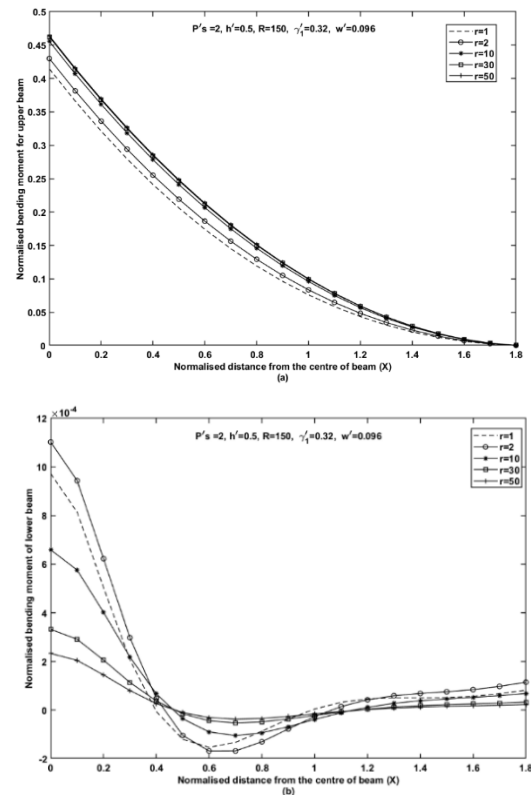


Figure 8. (a) Effect of the relative stiffness of upper and lower soil on the bending moment of the upper beam (b) Effect of the relative stiffness of upper and lower soil on the bending moment of the lower beam

4.4 Effect of self-weight of the upper beam on the deflection of beams

The increase in the thickness of the upper beam (pavement slab) is related to an increase in the daily CVPD and temperature differentials. The change in the self-weight of the upper beam can be achieved by changing the thickness of the upper beam from 0.16 m to 0.25 m (IRC:SP62 2014), which also changes the rigidity and characteristic length of the upper beam which affects all the other non-dimensional parameters. Therefore the analysis is performed while keeping the characteristic length R_1 equal to 1. Fig. 9(a) and 9(b) show the influence of the self-

weight of the upper beam w' , on the normalized deflection of the upper and lower beam respectively for the non-dimensional parameters $P'_s = 2$, $r = 2$, $h' = 0.5$, $R = 150$, $\gamma'_1 = 0.32$,

- Figures show that with the increase of normalised self-weight of the upper beam w' from 0.077 to 0.12, the normalised deflection of the upper beam y'_1 decreased by around 4 % at the centre and 3.5 % at the edge of the beam whereas, the normalized deflection of the lower beam y'_2 is decreased by 3.6% at the centre and 3.3% at the edge of the beam.
- It is observed that as the swelling pressure P'_s increases to 10, the decrease in the deflection due to the increase in the self-weight of the upper beam from 0.077 to 0.12 is found to be around 1 % at the centre and edge of the upper and lower beam.

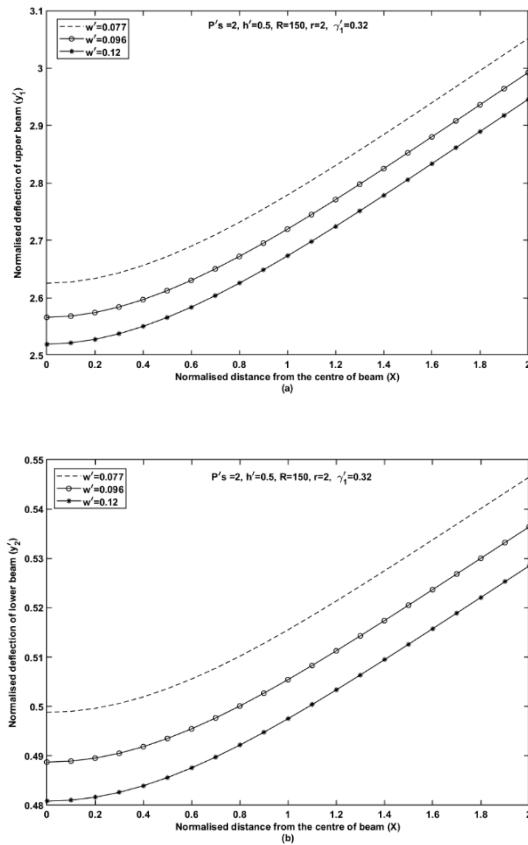


Figure 9. (a) Effect of self-weight of the upper beam on the deflection of the upper beam (b) Effect of self-weight of the upper beam on the deflection of the lower beam

It can be concluded from the result that in the case of higher swelling pressure $P'_s = 10$, increasing the self-weight of the upper beam is not very significant.

4.5 Effect of unit weight of base layer soil on the deflection of beams

Fig. 10(a) and 10(b) show the effect of the unit weight of base course soil, on the normalized deflection of the upper and lower beam for the parameters $P'_s = 2$, $h' = 0.5$, $R = 150$, $r = 2$, $w' = 0.096$ respectively.

- Figures show that with the increase in the value of γ'_1 from 0.28 to 0.36, the normalised deflection of the upper beam y'_1 decreases by around 3 % at the centre

and 2.8 % at the edge of the beam. Whereas for the lower beam the deflection y'_2 is about 2.8% at the centre and 2.4% at the edge of the beam.

- With the increase in swelling pressure P'_s to 10, the decrease in the deflection of the upper and lower beam is around 0.5 % at the centre and the edge of both the upper and lower beam.

The results for the higher swelling pressure, increasing the unit weight of base course soil have a negligible impact on the deflection of upper and lower beams

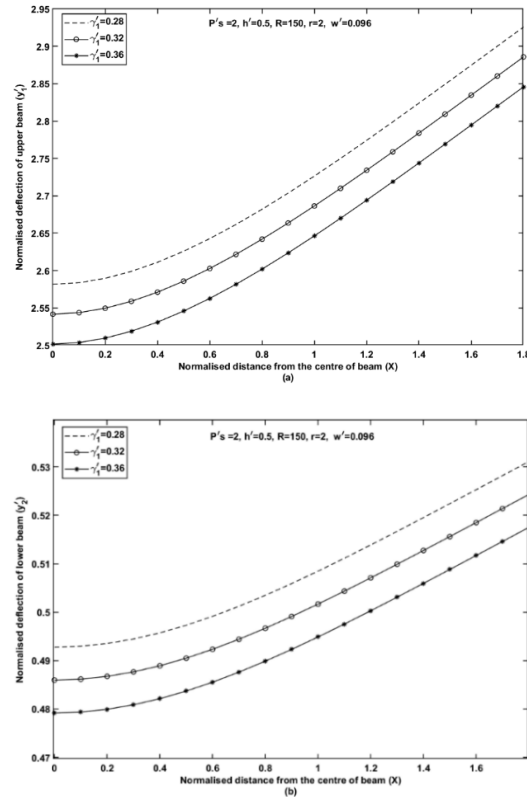


Figure 10. (a) Effect of unit weight of base-layer soil on the deflection of the upper beam (b) Effect of unit weight of base-layer soil on the deflection of the lower beam

5 CONCLUDING REMARKS

Based on the results and discussion presented in the earlier section following generalized conclusions have been drawn.

- Maximum deflection (heave) in the upper and lower beam is seen at the edge of the upper and lower beam.
- The relative stiffness of the base layer soil and subgrade soil (r), is found to be the most significant parameter affecting the response of the model. It is observed that the relative stiffness of the base layer soil and subgrade soil is directly proportional to the percentage variation in the normalised deflection of the upper beam with the constant of proportionality approximately equal to 1.
- The relative flexural rigidity of the upper and lower beam (R) does not affect the flexural response of the

upper beam. This may be due to the presence of a more rigid upper beam, the 'beam effect' of geosynthetic reinforcement does not get mobilised enough under loading and swelling stresses.

- The significance of the parameters to decrease the deflection of the upper beam is recognized as $r \gg h' \gg w' \gg \gamma'_1 \gg R$, i.e. r is the most significant parameter whereas R is the least significant parameter. This suggests that improving the stiffness of the soil is a better option than the inclusion of reinforcement at the interface of the base layer and expansive soil.
- The non-dimensional solution presented herein can help design low-volume concrete roads in a wet or temperate climate zone, where swelling is a more predominant mode of failure than the shrinkage of soil.

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