ISSN: 2278-4632
Vol-10 Issue-1 January 2020

Flexibility of Pavements and Expansive Soils

[1].Monalisha Mallick, [2]Nikhil Nayak*
[1] Professor, Dept. OF Civil Engineering, NIT BBSR,*
Asst. Professor DEPT. of Civil Engineering, SEARC, BBSR
[1]monalishamallick@thenalanda.com,nikhil35@gmail.com*

Abstract:
The study results of a pavement construction built at Tebessa, Algeria, on a relatively expansive material that is primarily made of brown clayey silt are discussed in this investigation. From a portion of road construction, cores were delivered to the lab. Remolded samples were collected from the road's subgrade. The majority of the soils were discovered to have medium plasticity and medium to high expansion potential. A free swell oedometer test indicated a constant volume pressure that generated stress greater than 350 kN/m2. The following will give an explanation of how expansive soil behaves in relation to flexible pavements. Generally speaking, X-ray diffraction [XRD] is used to classify soils. programme code The behaviour of three pavement structure models was numerically simulated using Plaxis 8.2, and a free expansion test was conducted to calibrate the soil subgrade using the Soft-Soil model. The study's methodology is based on a simulation of the vertical subgrade soil displacement caused by traffic load and how that displacement affects the behaviour of flexible pavement under load. According to the process outlined in the article, a sufficient surcharge pressure is utilised to stabilise the subgrade's swellable nature.

Keywords: Flexible pavements; expansive subgrades; soil calibration; Oedometer test; Soil behavior; Finite ele- ment method

1. Introduction

In expansive soils in various locations around the world, especially in semi-arid areas, there are several pavements to be found. These soils typically have a lot of clay and are unsaturated. Many problems with buildings and major structures are caused by the expansion of clayey soils that include smectites or illites in various amounts [Baheddi et al., 2007]. These soils experience a sizable volume shift when exposed to water following a dry state. According to Snethen et al. (1975), the primary cause of the volume change in expansive soils is the hydration of the clay minerals, or more specifically, the adsorption of water molecules to the exterior and interior surfaces of clay minerals to make up for the particle's inherent change deficiency. On the other hand, if the soil is dry, it shrinks, resulting in a reduction in volume, which promotes the growth of polygonal network cracks. They suffer from severe pavement and surrounding ground degradation. According to Prasad et al. (2010) and Jones and Holtz (1973), the losses from substantial damage to roadways that cross wide expanses of soil subgrade are estimated to be in the billions of dollars worldwide.

 ISSN: 2278-4632
Vol-10 Issue-1 January 2020

With variable degrees of success, other corrective procedures have been used, including soil restoration [Snethen et al., 1975], pre-wetting [SubbaRao and Satyadas, 1980], moisture control [Marienfeld and Baker, 1999], and lime stabilisation [Thompson and Robnett, 1976]. How- ever, these techniques suffer from certain limitations respect to their adaptability, like longer time periods re- quired for pre-wettting the highly plastic clays, [Stein- berg, 1977; Felt, 1953], difficulties in constructing the ideal moisture barriers [Snethen et al., 1975], pulveriza-tion and mixing problems in case of lime stabilization [Ramana Murty, 1998] and high cost for hauling suit- able refill material for soil replacement [Chen, 1988; Snethen et al., 1975], geogrids reinforcement [Guptaet al., 2008], polymer grid reinforcement [Miura et al., 2003], geosynthetic reinforcement [Zornberg and Gup-ta, 2009], Polymer grid [Miura et al., 2003], etc.

The problem is further exacerbated when the sub- grade is expansible. Even if the pavement is correctly designed, the swelling character of the subgrade goes to distort all predictions. It is well known that larger stresses can be created when volume change of a mate-

rial occurs. The stresses reflect in the form of crack-ing, heaving and settlement of highway pavements.Therefore, a significant increase in the costs of routinemaintenance, rehabilitation and even reconstruction ofthe deteriorated pavements will be forced to the roadauthorities [Hyunwook and William, 2009]. The crack-ing phenomenon can occur through volumetric changes under changing moisture conditions in expansive sub-grades. These volumetric deformations usually result indifferential movements of flexible pavements resting onthe expansive subgrade. Consequently, structural dam-ages could happen if no special measures have alreadybeen taken during the design process [Ayman, 2007]. The method used for pavement design in Algeria is known as the-catalog structure and is based on theFrench Method that uses the elastic Burmister's modelfor a multi-layer, semi-infinite structure. It assumes asemi-analytical stress-based field where deformationsare calculated for when pavement is subjected to veryheavy traffic. However, in the case of flexible pave-ment over expansive soils, subjected to high gradientsof volume change, the method does not take into ac-count such an effect in predicting pavement behavior.Literature reveals that several locations in Tebessa,Algeria, are made of expansive soils causing pavementdeteriorations. As a part of road network maintenance, rehabilitation methods have been developed specifi-cally for the flexible pavements of a National Highway[N10], which has suffered from severe degradation inits structural integrity. The geotechnical records of N10show that it was constructed on expansive subgrades.Totally, 31 soil samples were taken, 10 of which werecore samples and 21 were from wells. The results of laboratory test classified these soils of having mediumto high expansion potential. Further soil data were ob-tained using the Principal Component Analysis [PCA].Plaxis 8.2 software package with its linear elastic, MohrCoulomb and soft soil models were used to predict soilvariations. A surcharge pressure was used to stabilizethe heave of the pavement structure.

2. Location of The Study Area

Flexible pavement on expansive soils is generally com- mon in Tebessa, Algeria. The city consists of a col- lapsed basin surrounded by mountains, with an average

altitude of 800 to 1600 m above mean sea level. The city is bounded from north by the city of Souk-Ahras, from south by El'Oued, from east by the Tunisian bor- der and from west by two cities OumEl-Bouaghi and Khenchella, with an area of 21,000 km2. The 4.5 km long highway commences at the intersection of N10 and N82 [El Kouif road] and ends at the intersection ofthe N16 [El Malabiod road] and road of Bekkaria, with an average altitude between 814 and 842 meters above mean sea level [Figure.1].

3. Classification and Soil Profile at The Test Site As a part of enhancing the mechanical behavior of Na- tional Highway N10, a soil testing program was set up to test 10 trials of the samples cored 6 m deep and 21 wells of 2 to 3 m deep using a shovel. The purpose wasto establish the geological profile of the site and to en- sure there were enough intact and disturbed samples for laboratory testing. Visual analyses of wells and core samples revealed the presence of marly clay, clayey silt and marl. Table 1 shows the geotechnical characteris- tics of samples. Classification by Casagrande chart [At- terberg Limits] – based on liquid limit LL and plastic limit PL – showed that these soils were inorganic clay with medium to high plasticity. Another classification with Dakshanamanthy and Raman [1973] – based on

same parameter of Casagrande chart for the expansion potential – showed that the expansion potential of the soils was medium to high [Figure.2]

In addition to the routine characterization testing, the X-ray diffraction [XRD] technique was used to ob- tain semi-quantitative mineralogical composition and chemical analysis [Saad and Aiban, 2006]. We have used PANalytical X'Pert PROX-Ray Diffractometer [Type MPD] to obtain the data. These XRD micro- graphs vividly confirmed that soils are marly clays with 64% of calcite and 35% of aluminosilicate [Figures. 3].

4. Calibration of Oedometer Test For FreeSwell

One-dimensional tests are instrumentals for predicting the compressibility, collapse and expansion potential of soils [Saad and Aiban, 2006]. A geotechnical investiga- tion company issued the permission of extracting un- disturbed samples from the depths of 0.3 – 3.0 m and
3.0 – 6.0 m for oedometer test for free swell.

The results of testing were used in a calibrated soft soil model programmed in computer code as Plaxis 8.2. With respect to the geometry of the test, oedometer cell is simulated using a plane strain model by elements containing 15 nodes. For 10 trials of swelling pressure, the intention was to reach as closely as possible to the

ISSN: 2278-4632
Vol-10 Issue-1 January 2020

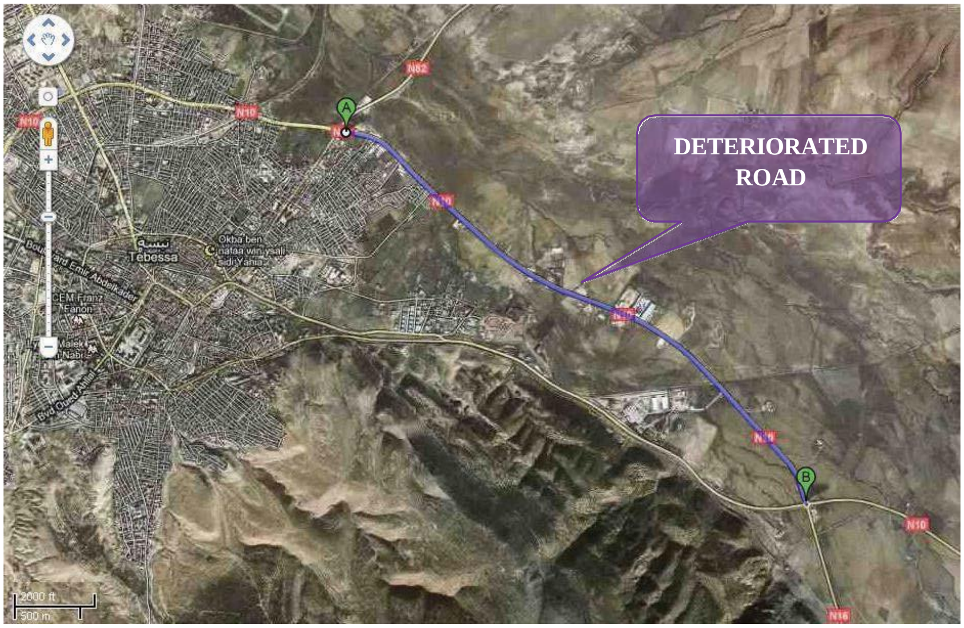


Figure 1.Satellite view of Tebessa with indication of the modelled area [Google map].

Table 1. Basic geotechnical and swelling characteristics of the samples.

Sample N°	Depth [m]	elements <0.08 mm %	Moisture content, W%	Dry density Œd kN/m^3	Wet density Œh kN/m^3	Liquid limit, LL %	Plasticity index, PI%	MB cm^3/g	Caco3 %	Swell pressure kN/m^2
1	1.18-2.00	93.4	19.36	1.56	1.82	63	38	7.3	48.38	-
2	1.30-2.00	92.4	19.26	1.56	1.84	64	36	7.8	51.38	-
3	0.25-2.00	92.4	18.36	1.56	1.86	64	37	7.8	48.36	-
4	0.30-2.00	98	14.26	1.52	1.82	62	36	7.2	49.38	-
5	0.30-2.10	92	12.89	1.70	1.86	52	32	7.1	59.38	-
6	0.30-2.00	93.6	8.37	1.49	1.64	34	16	2.0	73.55	-
7	0.20-2.00	97.6	12.82	1.71	1.88	50	30	7.1	60.26	-
8	0.20-3.00	97.6	12.71	1.70	1.89	51	31	7.0	60.00	-
9	0.30-3.00	97.2	12.84	1.72	1.87	48	32	6.9	59.38	-
10	0.20-1.50	96.4	12.6	1.70	1.86	52	32	7.18	46.28	-
11	0.70-2.30	97.6	18.33	1.47	1.75	65	39	7.17	46.15	-

ISSN: 2278-4632
Vol-10 Issue-1 January 2020

12	0.25-1.50	95.6	12.76	1.70	1.86	50	30	7.2	59.36	-
13	0.25-2.00	97.2	12.36	1.71	1.88	51	32	7.0	58.86	-
14	0.30-2.00	96.8	12.38	1.72	1.90	50	31	7.26	59.42	-
15	0.30-2.00	97.4	13.34	1.72	1.91	52	33	6.82	64.36	-
16	0.30-3.00	97.2	12.89	1.73	1.99	58	36	6.84	47.26	-
17	0.20-3.00	96.8	15.85	1.70	1.98	59	38	6.83	45.38	-
18	0.40-2.00	96.8	14.25	1.72	1.96	58	37	6.26	46.34	-
19	0.40-3.00	96.6	12.65	1.71	1.93	47	28	5.67	44.62	-
20	0.70-1.40	45.18	14.5	1.72	1 .95	34	14	4.5	61.48	-
21	0.68-1.00	98.25	18.6	1.66	1.97	52	30	6.5	44.44	-
22	0.50-6.00	91.6	27.66	1.48	1.89	50	31	4.11	49.61	200
23	0.70-2.50	97.8	17.53	1.71	2.01	56	36	6.3	53.48	240
24	0.50-3.00	98	17.24	1.70	2.01	53	31	3.95	63.33	280
25	0.40-4.00	93.8	19.39	1.63	1.99	59	42	7.8	42.76	350
26	0.50-5.00	92.4	26.60	1.47	1.86	58	37	7.14	45.59	300
27	0.60-5.50	93.4	19.83	1.70	2.03	41	40	7.3	48.78	340
28	0.40-2.40	92.6	19.84	1.56	1.86	51	33	7.8	40.00	320
29	0.40-3.00	93.8	27.72	1.56	1.95	52	33	5.9	62.20	175
30	0.50-5.00	98.6	17.74	1.58	1.85	46	30	6.0	50.00	240
31	0.70-6.00	98.6	23.06	1.60	2.04	53	35	6.1	53.97	240

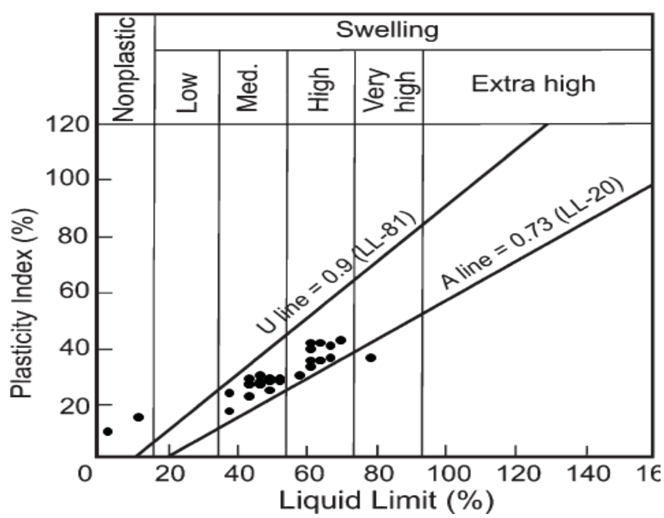


Figure 2.Classification of subgrade soils of Tebessa, based on Dakshanamanthy and Raman [1973]

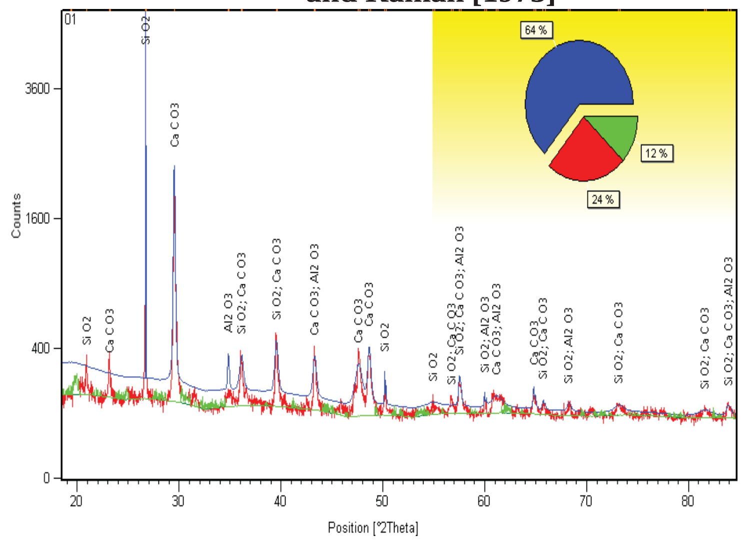


Figure 3.[a].XRD result of sample n°15 [bleu: calcite, red: silica, green: alumina]

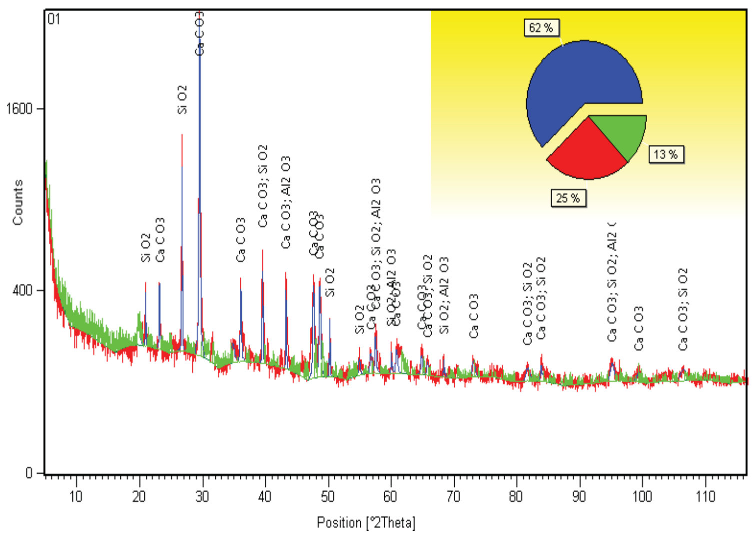


Figure 3.[b]. XRD result of sample n°29 [bleu: calcite, red: silica, green: alumina]

ISSN: 2278-4632
Vol-10 Issue-1 January 2020

real parameters seen in site. Initial values given to the soil parameters were □n=18.3 kN/m^3, □sat=21.1 kN/m3 and permeability of 0.001 m/day in undrained condi-
tions. Shear strength parameters were taken directly from the C.U test [direct shear test] which released φ=8° and C=75 kN/m2 with a dilatancy angle of T=0°. Oedometer.parameters are taken directly from the oe- dometer test results, the modified compression index
□$^□$ = Cc/2.3[1+e]=0.036 and the modified swelling in-
dex □$^□$ = 2.Cg/2.3 [1+e] = 0.035. Void ratio was e=0.47. All these parameters are essential for the model. A large number of calculations have been followed by changing the value of the parameters between their

lower and upper bounds. The results have been sum- marized in Table 2.

Figure.4 shows the results of the curves of expansion pressure; the variation of ΔH/H versus log [□]. The real curve [RC] has been compared with the numerical curve, which has been resulted from simulation. The calibration results allow for a good simulation of both the initial small strain stiffness and of the large strain behavior. The user-defined function of Plaxis was used
in order to set the pore water pressure distribution so that it simulates the expansion pressure for the calibra-tion process.

The numerical simulation of free expansion test re-

Table 2. Combination of parameter for calibration.

Parameter	Upper bound	Lower bound	Number Of steps	Selected value
Cc	0.100	0.800	70	0.290
Cg	0.020	0.100	45	0.072
C[kN/m^2]	10	100	45	82
[illegible]	1	25	25	4.4

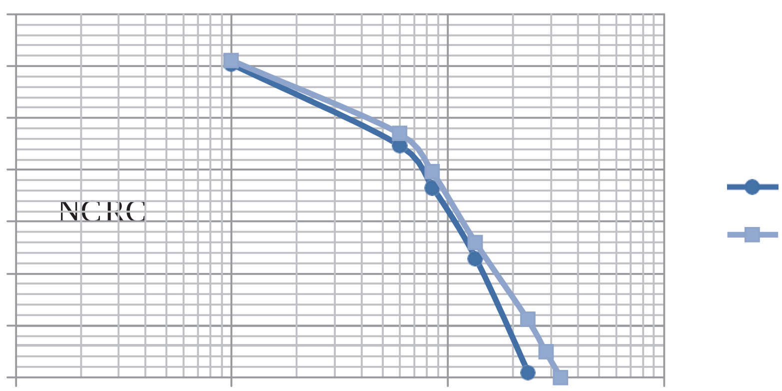
ISSN: 2278-4632
Vol-10 Issue-1 January 2020

7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00

ΔH/H

1 10

(σ) kPa



100 1000

Figure 4.Simulation of free swell test for the sample at the depth of 3 m [RC: real curve; NC: numerical curve]

vealed a value of 240 kN/m2 instead of 340 kN/m2 ob-tained from the real test. Similar deformations were ob- served in the first four loading levels. All these validate that the free expansion test overestimates the expansionpressure in the case of medium to highly expansive soils, since the free-swelling test can cause changes in structure during expansion before it returns to the zero strain state, where this state was also seen by Bultel [2001].

5. Numerical modeling

Objective

In order to cope with the complications of describing the swelling behavior of the expansive soil, research- ers have developed alternative approaches [Banu et al.,2009]. One of them being the use of finite elements, themain purpose of finite element simulation is to:

1. Get a deeper understanding as to why permanent de- formations happen in different parts of flexible pave- ments resting on expansive soils;
2. Find the best method to model the stresses. Since thestress state depends on the soil model, several

kinds of material models have been tested to estimate which soil model would give the most reliable stress distribution. This stress-strain analysis created a basis platform for

future development of models in permanent deforma- tion;

3. Find the stabilization methods to overcome the cy- cling variations of expansive subgrade.

Plaxis version 8.2 was used to model the behavior. Pavement structures have rarely been analyzed with finite element programs under dynamic loading. One reason for this is the fact that traffic loading is much more complicated than static loading normally applied in geotechnical problems. Another reason is that the material models in finite element programs have main-ly been developed for static loadings not for repetitive cyclic loading. Dynamic analysis needed to be tested with a repetitive half-sin loading; but it was found that the dynamic module of Plaxis 8.2 was not suitable for modeling the traffic loading [Leena, and Rainer, 2004]. Given the aforementioned reasons, the analysis has to be simplified. The first part of the expansion pressure analysis was to study the volumetric changes of the sub- grade by the test oedometer for free swell. With this analysis, the subgrade calibration was based on physi- cal and mechanical parameters such as cohesion [C], friction angle [φ], density and oedometer parameters of Cc, Cg and Pc. To approach as closely as possible to the actual state, a large-scale numerical model, which includes a flexible pavement structure based on an ex- pansive subgrade, was established. The structure was subjected to the loads from two trucks with dual wheel load of 650 kN/m2 per axle. Then a complete analysis of stress-strain was performed on the pavement struc- ture.

Modeling

Modeling was to simulate an existing flexible pavement structure that had suffered several damages after one year of its construction. This road had been constructed on an expansive subgrade classified as brown clayey silt. The total thickness of the pavement was 0.76 m. Clayey silt subgrade was covered with a 0.20 m layer of calcareous tufa as the improved subgrade, 0.20 m crushed gravel as subbase course, 0.20 m crushed gravel as base course and 0.06 m asphalt on top. The distresses observed on the pavement can be summarized as mild transverse and longitudinal cracks, mid-block cracking, alligator cracking along the shoulders, consolidation rutting for 0.06 m deep and finally average subsidence. A form of cracking can be seen in Figure. 5. Oedometer test for free swell showed that the vertical deformation [uplift] has a maximum height of 0.021 m with 350 kN/ m2 of expansion pressure. The groundwater level fluc- tuated between 0.00 and 0.50 m. No change in water pore pressure was considered for hydraulic analyses.

At first, passage of just one truck was simulated. Then, loading from two trucks at the same time with dual wheel of 0.60 m wide was studied. Modeling was done

by a static ax-symmetric analysis and the element mesh consisted of triangular elements each with 15 nodes. The input parameters of the structure are shown in Ta- ble 3. To simulate the variation of stresses dependant on the Young's modulus, layers were divided into sub-lay- ers with the same strength parameters but with different elasticity modulus. Use of plain-strain analysis, where the loading would have been continuous linear load- ing, could result in an overestimation of stresses and responses [Leena and Rainer, 2004]. To model the load- ing area of the dual wheel, the total load was transferred to a circular load with a known mean contact pressure. The detail of the structure and boundary conditions are illustrated in Figure. 6.

For this model, the attention was focused on the stresses and resilient deformations. The modeling initiated from values derived from the laboratory tests. The results of measurements and calculations of the resilient defor-mations were compared with each of other soil param-eters and they were modified to provide a distribution of stresses and strains as close as possible to the reality, i.e. the calibrated soil that has been mentioned previously. The magnitude and development of permanent defor-mations depend on static stress state of the material [Žlender, 2008], and how far the stress state is from the static failure line. This assumption is not entirely valid in the case of traffic load, but it gives a good estimate about sensitivity of materials for permanent deforma-tion. The static failure criterion, where it is widely adopted in geotechnical and in pavement materials, is

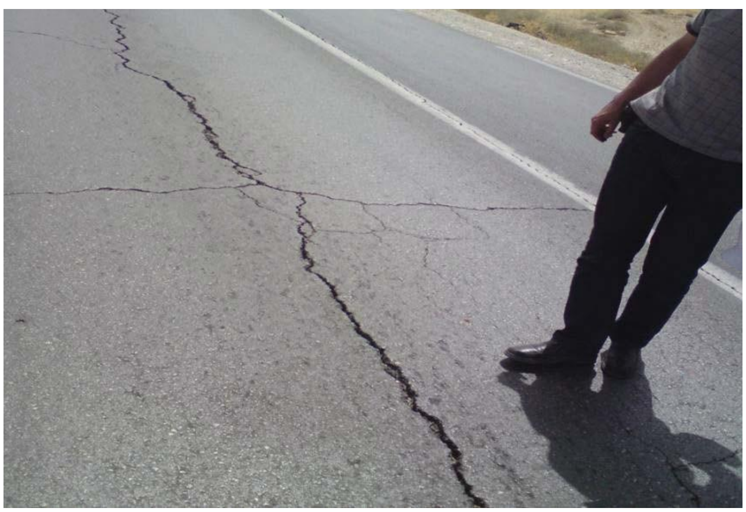


Figure 5. Cracking distress on pavement structure.

Table 3. Input parameters of the model.

Materi al	Aspha lt	Base course crushed gravel	Subbase crushe d grave l	Improve d subgrad e tufa	Subgrad e swelli ng clay
Thickness, mm	60	200	200	200	200 0
Young’s Modulus,	5400	300–200	140–	70	10–

ISSN: 2278-4632
Vol-10 Issue-1 January 2020

MN/m^2			90		8
Poisson's ratio	0.35	0.35	0.35	0.2 5	0.35
Unit weight, kN/m^3	25	21.2	22.0	20	18
Cohesion, kN/m^2	--	30	20	9	10
Friction angle [°]	--	43	44	36	25
Dilatation angle [°]	--	13	14	6	0
K_0	1	0.32	0.30	0.4	0.8

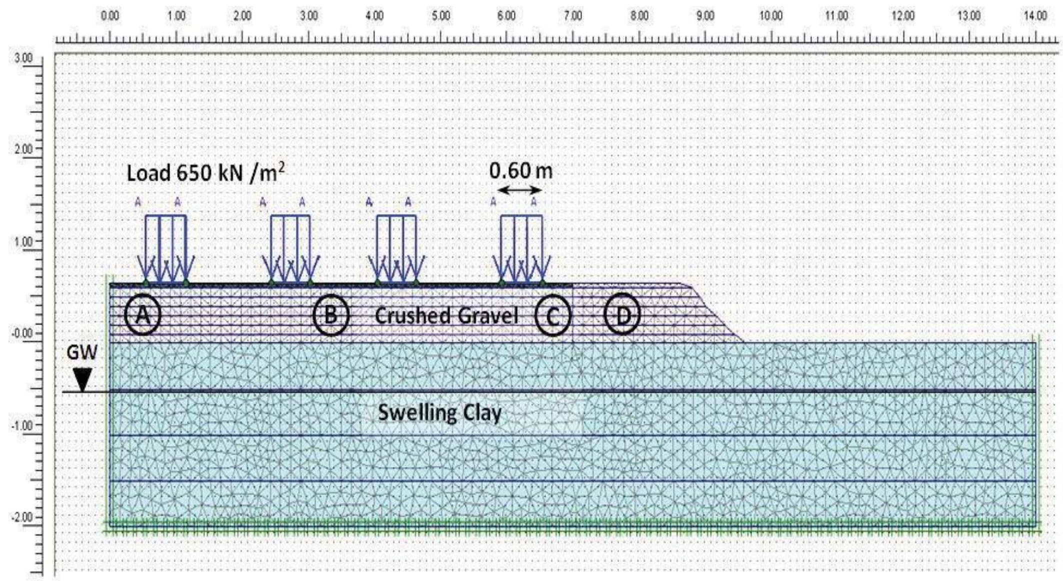


Figure 6. Numerical model: details and the boundary conditions

with the failure criterion of Mohr-Coulomb [Leena and Rainer, 2004]. However, the experimental results showthat the strength envelopes of almost all geo-materials have the nature of nonlinearity in the σn–τ stress space.In addition, linear failure criterion is a special case of failure criteria [Lianheng et al., 2010]. The behavior ofembankment connects the failure ratio to the deviatoricstress ratio, in this case the failure ratio R can be writtenas follows:

$$R = \frac{q}{q_0 + M p'}. \qquad (1)$$

C Cohesion, kN/m2

M the slope of the failure line in p'-q spacep' hydrostatic pressure, kN/m2

φ friction angle.

Since the subgrade has a swelling character, it was modeled by Soft-Soil model. This model takes into ac- count the following parameters: stress dependent stiff- ness [logarithmic compression behavior], distinctionbetween primary loading and unloading cycle [com- patible with the swelling behavior], memory for pre- consolidation stress and failure behavior according to where $M = \frac{6.sin\emptyset}{3-sin\emptyset}$

and $q = \frac{c\ .6.cos\emptyset}{3-sin\emptyset}$

the Mohr-Coulomb criterion [Brinkgreve, 2002]. The Soft-Soil model assumes that there is a logarithmic re-

R the failure ratio

q deviatoric stress, kN/m2

q0 deviatoric stress, when p' = 0

lationship between volumetric strain [□v] and mean ef- fective stress [p'], where the virgin compression can be formulated as:

$$\varepsilon_v - \varepsilon_v^0 = \lambda^* \ ln\left(\frac{p'}{p}\right)$$

Figure. 8 illustrates total displacements in the subgrade and pavement by three models after two cycles of load- ing. The Figure depicts that according to the first two

where λ^* is the modified compression index, and ε_v^0 is the initial volumetric strain.

During isotropic unloading and reloading a different path [line] is followed, which can be formulated as:

models, i.e. linear elastic where the total displacements are 1.65 x 10^{-3} and Mohr-Coulomb model with maxi- mum displacements of 1.79 x 10^{-3} m, displacements in the pavement occurred only in the contact between

$$\varepsilon_v^e - \varepsilon_v^{e0} = \kappa^* \ ln\left(\frac{p'}{p}\right) \quad (3)$$

wheels and surface course while they do not affect the subgrade. On the other hand, it was evidenced in the Soft-Soil model results that the movements had concen-

where □ □ is the modified swelling index [Brinkgreve, 2002].

6. Analysis of Results

The modeling results have been presented in Figure. 7 and 8. Three material models were applied, such as lin- ear elastic model, Mohr-Coulomb and Soft-Soil model. Linear elastic model was the first model for calculations, which look into the pavement structure and subgrade in its entirety. In the modeling, the modulus of each layer was tested as described earlier and then fixed to some practical values. Mohr-Coulomb model used the same deformation parameters in linear elastic model, along the subgrade and pavement structure. Finally, the pave- ment structure was modeled by Mohr-Coulomb model and subgrade with Soft-Soil model. The stress states of the subgrade in three models and the analysis of the structure, as illustrated in Figure.7, indicate where the plastic, tension-cut-off and cup points are situated in structure with the same calculations.

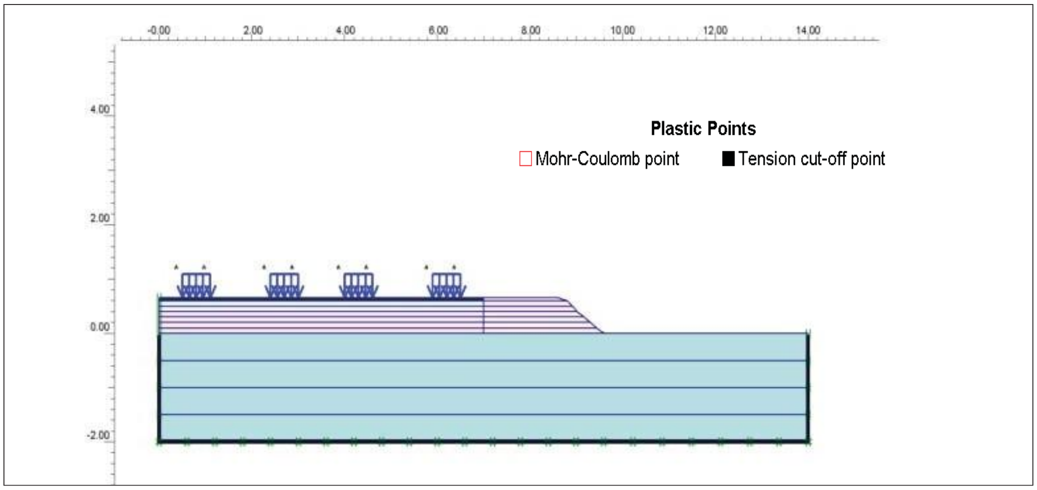
Figure. 7 shows the plastic points in subgrade and pave- ment according to the three models. For the linear elas- tic model, it shows no presence of plastic points, for Mohr-Coulomb model. It is observed that the subgrade is largely affected by tensile cut-off points, in upper part of the pavement and shoulders, with the appearance of plastic Coulomb points in transition zone between the pavement and shoulder and in contact with traffic loads. For the Soft-Soil model, concentration of cap points in the subgrade under the shoulder and middle width of the pavement, concentration of tensile stresses [tension cut-off] in shoulders and in surface course, concentra- tion of plastic Coulomb's points in contact between wheel and pavement and in the transition zone between the pavement and shoulder are clearly seen.

trated largely in subgrade beneath the transition zone between the pavement and shoulders and along the left side of the road where the pavement is slightly deformed with maximum displacements of 44.26×10^{-3} m.

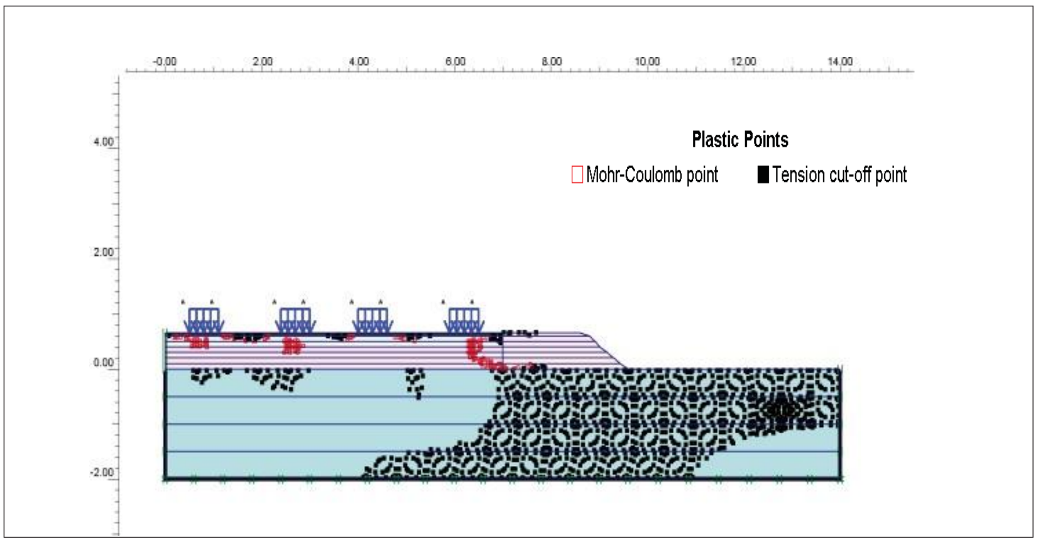
Figure. 9 represents the curves of stress path for sub- grade and surface pavement, and it reveals that the sub- grade [of expansive clay] has elasto plastic hardening behavior, and the pavement has an elasto plastic non- linear behavior with a various moduli of deformation along different parts of pavement. Figure. 10 represents the vertical displacement of subgrade and pavement structure. It shows that movements of swelling sub- grade affect different parts of pavement, where points A, B, C and D are located in the pavement structure as shown in Figure. 6

7. Stabilization by Surcharge Pressure

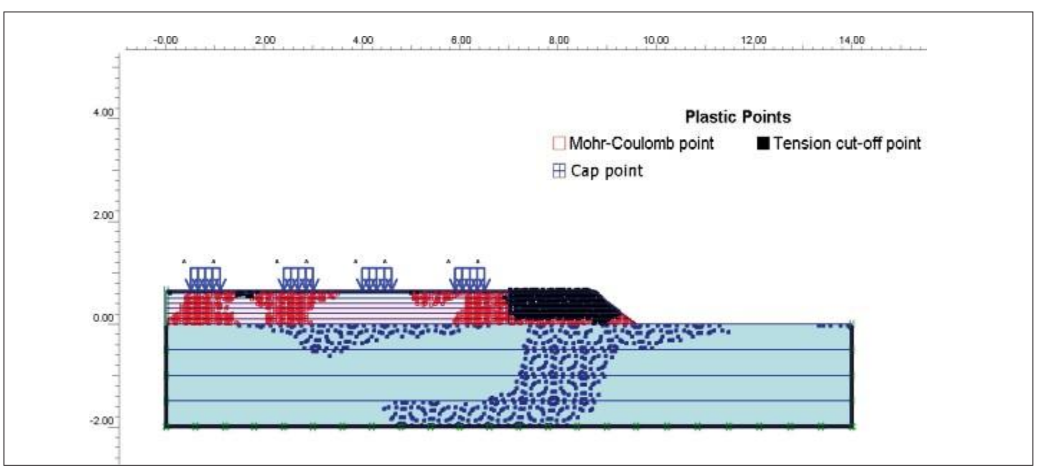
One of the following methods provides the calculated surcharge pressure: (1) constructing an inert embank- ment to a calculated height and [2] replacing the exist- ing soil to a calculated depth with inert material; the combination of the two methods can also be utilized. To overcome the distress of pavement over expansive subgrades and to limit their heaves, the subgrade is loaded by a surcharge pressure equal to the swelling pressure. The principle is to make equilibrium between the pavement structure and the subgrade by eliminat- ing the upward swelling pressure. This can be done by applying a surcharge on the subgrade (Figure. 11). The surcharge pressure is designed within a two-stage pro- cedure, replacing a part of the current subgrade with a non-expansive material and heightening the embank- ment so much that the desired surcharge pressure is obtained. The goal is to obtain equilibrium as a result of which the swelling pressure is eliminated and cyclic



Linear elastic model



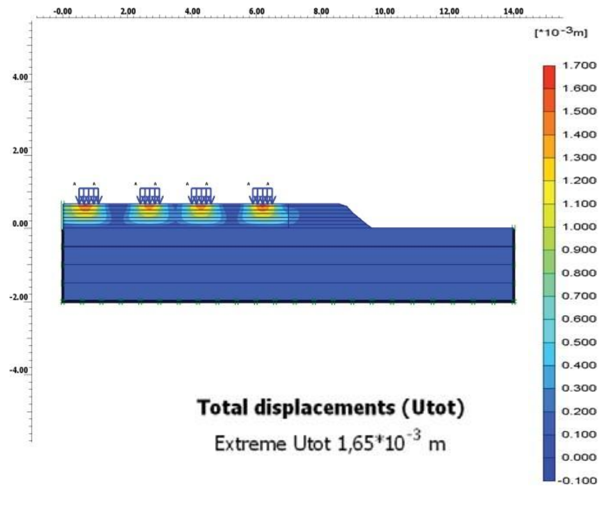
Mohr-Coulomb model



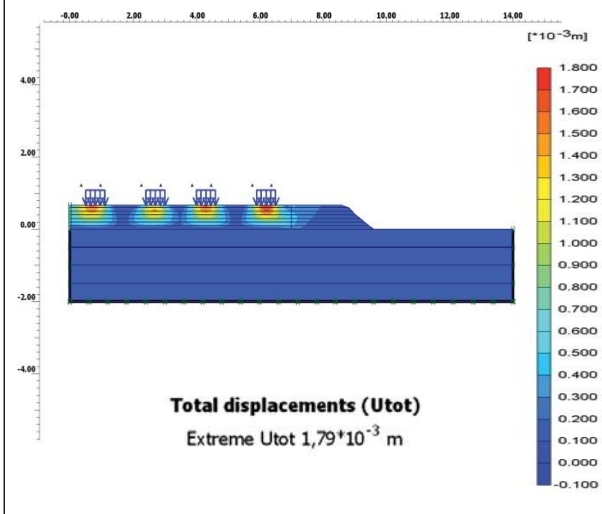
Soft-Soil model

ISSN: 2278-4632
Vol-10 Issue-1 January 2020

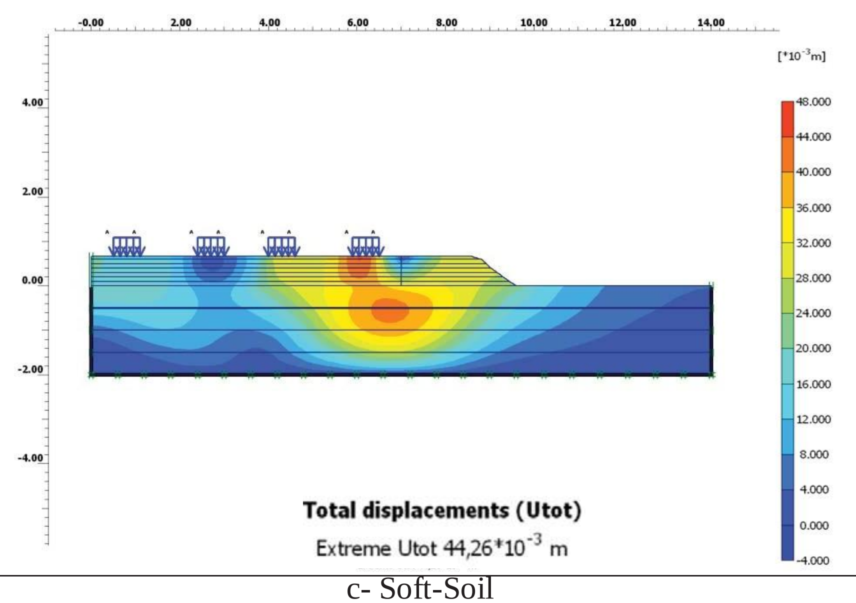
Figure 7. Plastic, tension-cut-off and Cap points



a- Linear elastic model



b- Mohr-Coulomb model



c- Soft-Soil model

Figure 8. Total displacements

variation in pavement will diminish. The subgrade soil is over-excavated in the seasonal active zone to a depth where the moisture content remains virtually constant over time leading the volume change of the soil to be

by Terzaghi [1954]. When computing the surcharge fill properties, non-traffic load conditions was considered with the purpose of stabilizing the swelling pressure.

ISSN: 2278-4632
Vol-10 Issue-1 January 2020

negligible over time.

The swelling pressure was determined by the oedom-

$$\sigma = \text{Pp}\,\text{S}$$

(4)

eter test, and after simulation, it output the value of240 kN/m2. The surcharge pressure must be the same in order to avoid the height settlements. Calculation of surcharge pressure was based on a method introduced

$$\sigma_Z = y_a \times H_L \qquad (5)$$

Then

$$P_P = y_a \times H_L \times S \qquad (6)$$

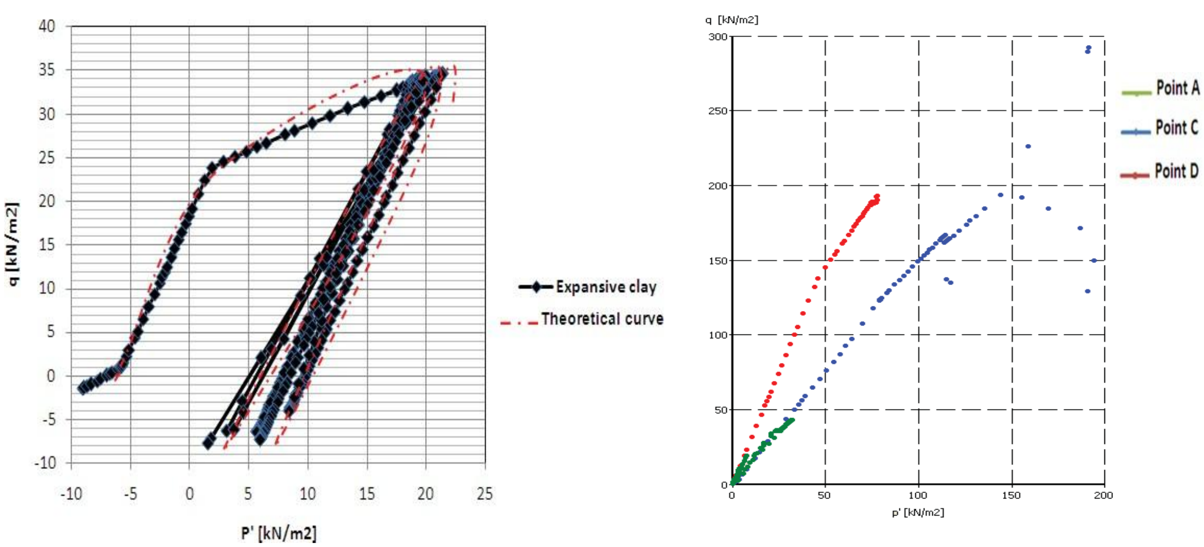


Figure 9. Stress path; left: subgrade, right: paement structure.

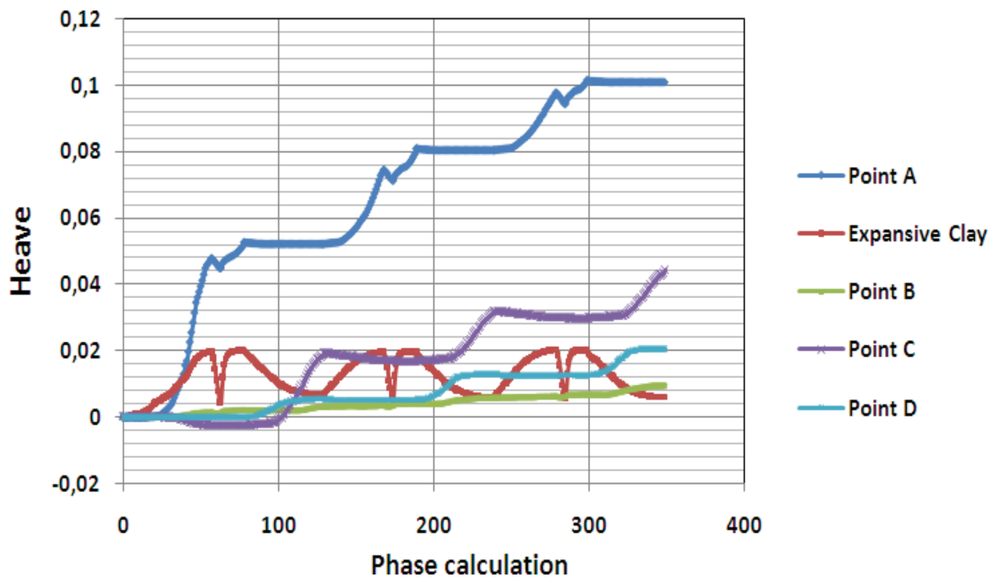


Figure 10. Vertical displacement of pavement and subgrade

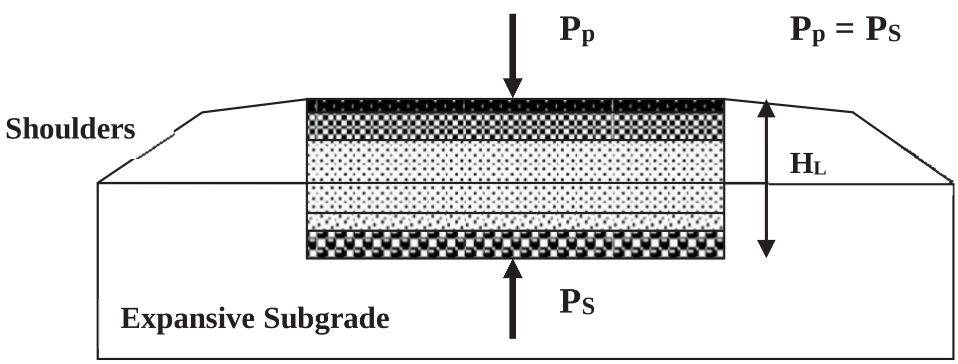


Figure 11. The principle of the stabilization of an expansive subgrade

where P_P ..is the applied vertical pressure [kN/m2], γ_a the average total density of the fill [kN/m3], HL is the se- lected fill thickness [m] and S is the area of the fill soil which is assumed as square of each side 1 m.

Calculation resulted in a total thickness of the surcharge layer of 1.1 m. The fill layer is designed including 0.40 m layer of ballast to limit the capillarity ascension, 0.10 m layer of crushed sand as anti-contamination layer, 0.40m of layer of calcareous tufa as embankment, 0.20 m layer of crushed gravel as base course and 0.06m as-phalt on the top. The earthmoving operation started in July 2007, when the precipitation is rare and the tempera- ture reaches to 45°C. The purpose was to reduce the varia- tion of moisture content. During the construction of the fill and after five years of service, the variation of heave of pavement versus the time has been plotted in Figure.

ISSN: 2278-4632
Vol-10 Issue-1 January 2020

12. The Figure shows that, prior to the fill, the heave ofthe subgrade had been 0.023 m as the maximum recorded expansion. Later, it decreases to -0.005 m at the end of the first month of the treatment. After nine months, the settle- ments are almost stabilized at -0.007 m with no cracks in pavement. Moreover, after three years the maximum set- tlement reaches -0.009 m due the secondary consolidation with no deformation in the pavement structure.

7. Conclusions

This study is focused on determination of the behavior of flexible pavements under the influence of the evo- lutionary character of swelling soils of Tebessa, Alge-

ria. Based on the results reported in this study, we can concluded that the subgrade soils of the study area are marly clays with 64% of calcite and 35% of alumino- silicate having medium to high Atterberg limits, me- dium to high swell potential and high swelling pressure about 350 kN/m2.

The surprising distribution of permanent deformations in this model shows that it is not easy to predict where deformation will occur. For similar loading conditions, it is clear that the explanation of the permanent strain distribution can only be found through the relations be- tween the stresses and stiffness of the materials [i.e. ma- terial properties and their thickness]. The calculations indicate that with linear elastic model, the deformations concentrated exclusively in the pavement depend on the traffic load, which limits the damage only in these areas with zero plastic deformations. Therefore, this model isa suitable one used for designing pavement structures with a deformation of about $1.65x10^{-3}$m.

Mohr-Coulomb model reveals that the deformations are concentrated exclusively in the pavement situated in direct contact with traffic loads, with the appearance of tension cut-off points in the surface course, subgrade side shoulder and the surrounding area of the structure pavement. It indicates there is a cracks network in these areas due to the shrinkage of the clayey soil inducing cracking in the pavement. The deformation is near the linear elastic model with $1.76x10^{-3}$ m. In Soft-Soil model, the permanent deformations are not the identi-

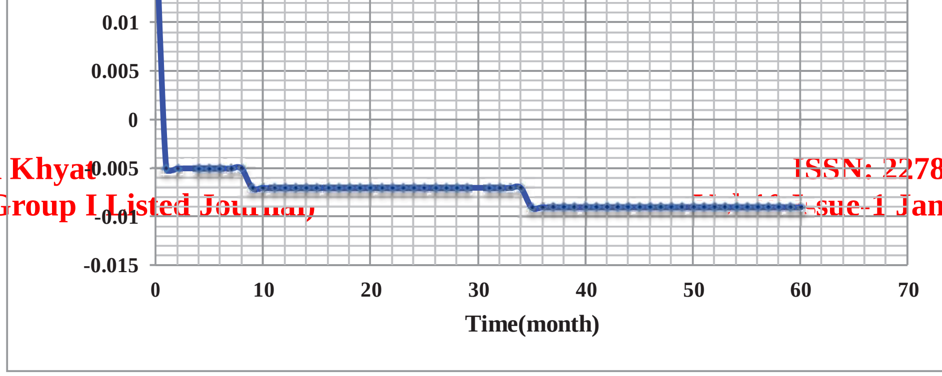


Figure 12. Variation of heave of pavement versus the time

cal throughout the structure. The model shows that the majority of displacements are in the shoulder side of the subgrade, which indicates that the source of displace- ments is the transitional subgrade between the pavement and its shoulder. These distortions arise mainly at first by the swelling subgrade, and secondly by the applica-tion of traffic load. Their value is 44.26×10^{-3} m, whichcreates an influence zone that spreads to the shoulders and the surrounding area of the pavement structure.

Deformations are confirmed by plastic points wherethe cup points are manifested in the subgrade along the shoulders and in the middle of the pavement, which indicate that the stress state in these points is equiva- lent to the pre-consolidation stress or they are highly loaded to the maximum stress level that has previously been reached. Hence, the pavement is solicited by both sides; traffic which transmits a high pressure to sub- grade and to groundwater, and deformations caused by the expansion of subgrade. Along the shoulders, there is a concentration of tension cut-off points. The points are in failure state under tensile stress, which explains the appearance of crack networks along the shoulders. The cracks promote water penetration to the pavement structure creating a zone of saturation that will reduce the bearing capacity of the soil, decrease the stability of the shoulder slope, and produce settlement and lateral displacement of shoulder. Coulomb plastic points con- centrated in the contact zones of traffic indicate where the Mohr's stress circle touches the Coulomb failure envelope. Therefore, it limits the state of failure along the shoulders; thus, imbalance area is created.

It is also found that the subgrade has a hardening be- havior, i.e. elasto plastic type of hyperbolic model. The soil deforms according to the charge induced by traffic with an increase in the plasticity limit, then, the am- plification of the surface load. This behavior affects considerably the deformation in different parts of the pavement where deformations are not identical in its different parts.

These observations accord fairly with Laroche's work [1973], which describes perfectly the behavior of flex- ible pavements over expansive soils and conform per- fectly to the results of this research. Therefore, deterio- ration taking place in the subject road is not caused only by the traffic load but rather by the slow cyclic variation of swelling soil. The soft soils generate large strains as

function of the magnitude of external loads where the results adapt perfectly with the Soft-Soils model.

Stabilization of cyclic movements of expansive sub- grade is neutralized by applying surcharge equal to the swelling pressure. This is done by applying surcharge fill pressure equal to the swelling pressure. The applica- tion of the surcharge fill has reached the value of -0.005 m at the end of the first month. After nine months, the settlements had been stabilized with -0.007 m of de- formation and no cracking in the pavement. Moreover, after three years, the settlements increase just slightly to -0.009 m due the secondary consolidation in the pave- ment structure.

8. References

- Ayman, A. (2007) "Numerical simulation of a trial wallon expansive soil in Sudan", Plaxis Bulletin issue 21

- Baheddi, M., Djafarov, M. and Charif, A. (2007) "De- sign method of flexible continuous footing on swelling clayey soils", Medwell journals, Journal of engineering and applied sciences, 2[3], pp.531-539

- Banu, S., Mustafa, A., Mustafa, V. and Fikret, K. (2009) "Prediction of swelling pressures of expansive soils using artificial neural networks". Elsevier Scien- tific Publishing, Advances in Engineering Software, 41,pp 647–655

- Brinkgreve, R.B.J. (2003) "Reference Manual Plax- is2D", Delft University of Technology & PLAXIS bv, Netherlands.

- Brinkgreve, R.B.J. (2003) "Material Models Plax- is2D", Delft University of Technology & PLAXIS bv, Netherlands.

- Bultel, F. (2001) « Prise en compte du gonflement des terrains pour le dimensionnement des revêtements des

tunnels », Thèse de Doctorat, école nationale des pontset chaussées, Paris [in french].

- Chen, F.H. (1988) "Foundations on Expansive Soils",Elsevier Pub Co, Amsterdam

- Dakshanamanthy, V. and Raman, V. (1973) "A Simple Method of Identifying an Expansive soil", Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, 13 [1], pp 97–104

- Duhamel, D., Nguyen, V., Chabot, A and Tamagny, P. (2003) « Modélisation de chaussées viscoélastiques ». 16ème Congrès Français de Mécanique, Nice, pp 1-5 [in french].
- Felt, E.J. [1953] "Influence of Vegetation on Soli Moisture Content and Résultent Soil Volume Chang- es.", Proceedings of 3rd Conference on SMFE, Vol 1, Zurich

- Gupta, R., McCartney, J. S., Nogueira, C. L and Zorn- berg, J. G. (2008) "Moisture Migration in Geogrid Re- inforced Expansive Subgrades", The First Pan American Geosynthetics Conference &

Exhibition, Cancun, Mexico

- Hyunwook, K. and William, G. (2009) "Finite ele- ment cohesive fracture modeling of aairport pavementsat low temperatures", Elsevier Scientific Publishing, Cold Regions Science and Technology, 57, pp 123–130

- Işık, Y. [2006] "Indirect Estimation of the Swelling Percent and a New Classification of Soils Depending on Liquid Limit and Caption Exchange Capacity", El- sevier Scientific Publishing, Engineering Geology ,85,pp 295–301

- Jones, D.E. and Holtz, W.G. (1973) "Expansive Soils- the Hidden Disaster", Civil Engineering, Vol. 43, No. 8

- Laroche, C. (1973) «" Étude des sols pour la route Waza Maltam, Sur Argile Gonflante ». Revue généraledes routes n° 486 [in french]

- Leena, K. and Rainer, L. (2004) "Modelling of the

stress state and deformations of APT tests". VTT Tech- nical Research Centre of Finland. In: Proc. of the 2nd International Conference on Accelerated Pavement Testing.

- Lianheng, Z., Liang, L., Feng, Y. and Jing, Z.(2010) "Evaluating the Effects of the Magnitude of Pseudo- static Force on Reinforced Slopes based on the Non- linear M-C Failure Criterion", EJGE, Vol. 15, Bund. J,pp1017-1052

- Marienfeld, M.L. and Baker, T.L. (1999) "Paving Fab- ric Interlayer as a Pavement Moisture Barrier", Trans- portation Research Circular, E-C006

- Miura, N., Sakai, A., Taesiri, Y., Yamanouchi, T. and Yasuhara, K. (2003) "Polymer grid reinforced pave- ment on soft clay grounds", Elsevier Scientific Publish- ing, Resources, Geotextiles and Geomembranes, vol- ume 9, Issue 1, 1990, pp 99–123

- Prasad, D.S.V., Kumar, M. and PrasadaRaju, G.V.R. (2010) "Behavior of reinforced sub bases on expansivesoil subgrade, Global Journal of Researches in Engi- neering, Vol. 10 Issue 1,Ver 1.0

- RamanaMurty, V. (1998) "Study on swell pressure and the method of controlling swell of expansive soil",Ph.D. Thesis, Kakatiya University, REC, Warangal,A.P.

- Saad, A. and Aiban. A. (2006) "Compressibility and swelling characteristics of al-khobar palygorskite, east- ern Saudi Arabia", Elsevier Scientific Publishing, En- gineering Geology 87, pp 205–219

- Snethen (1975) "Review of engineering experiences with expansive soils in highway subgrades", US ArmyEngineer Water Ways Experiment Station, FHWA, USA

- Steinberg, M.L. (1977) "Ponding an Expansive Clay Cut: Evaluations and Zones of Activity", TRR-641, TRB, pp 61-66

- SubaRaob, K.S. and Satyadas, G.C. (1980) "Prewet- ted and Overloaded Expansive Soil",