Dynamic coupled analysis for rutting in flexible pavement foundations under cyclic loading

循环荷载下柔性路面路基变形的动力耦合分析

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Abstract: A state of the art is briefly reviewed on rutting in flexible pavement foundations under cyclic loading. Based on the dynamic soil-pore water coupled system theory, this paper focuses on the numerical modelling of rutting in flexible pavement foundations under plane strain condition. A new simple viscoelastic dynamic model for predicting the cyclic response of soils is developed to predict the nonlinear response of pavement materials under cyclic loading. The proposed model takes into account the shear modulus, the damping ratio, pore water pressure and permanent deformations of pavement foundation materials. An illustrative example is given for the prediction of rut depth development in a flexible pavement.
Key words: rutttting; flexible pavement; pavement foundation; dynamic coupled analysis; cyclic loading
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Biography: HUANG Yu, male, born in 1973, lecturer of the Department of Geotechnical Engineering, Tongji University, Shanghai. He has been awarded the degrees of Bachelor and Doctor in Geotechnical Engineering by Tongji University. His current research embrace geotechnical engineering design and evaluation, soil dynamics, pile foundation and geotechnical earthquake engineering.
摘 要: 首先简要综述了循环荷载下路基变形的研究现状。然后, 基于饱和土体的动力耦合分析理论, 研究了平面应变条件下的柔性路面路基变形的数值分析模型和方法, 并建立了一个简单的粘弹性动力计算模型来预测路基在循环荷载下的动力反应。该模型可以全面考虑路基材料的剪切模量、阻尼比、孔隙水压力以及永久变形。最后, 以一个柔性路面为例, 应用本文方法在循环荷载下的路基变形发展进行了分析。
关键词: 车辙; 柔性路面; 路基; 动力耦合分析; 循环荷载

1 Introduction*

Foundation of pavement generally consists of the part of the construction blow the asphalt or cement-bound base and surfacing (Fig. 1). For thin-surfaced and unsurfaced roads, it will effectively embrace the whole construction. The pavement foundation plays two roles. Firstly, it must provide a short-term pavement to carry construction traffic and provide an adequate platform for the placement and compaction of the expensive high-quality manufactured asphalt or concrete placed above. Subsequently, the foundation must perform in a long term as a support system to the completed pavement for the required design life.

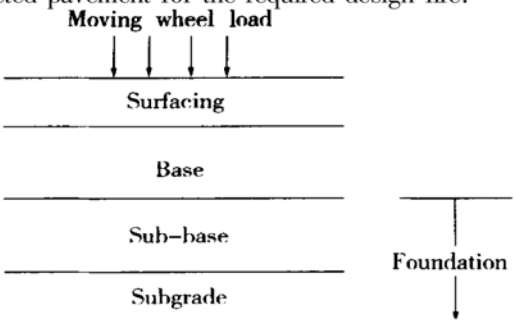

Fig. 1 Definition of pavement foundation

Rutting is the result of permanent strains, which accumulate within the constituent materials. Rutting is one of the failure mechanisms for flexible pavements. One of the main aspects of the design philosophy for flexible road pavements is limitation of the rut development in the pavement structure. Rutting development control has become an important design philosophy for road pavement during recent decades. According to the AASHO Road Test[1], about over 60% rutting of the flexible pavement occurred in the foundations. It's important to prevent the accumulation of permanent strains in the flexible pavement foundations.

The main factors affecting rut response in flexible pavement foundations are mainly as follows:

(1) The nature of vehicle loads to be accommodated, such as load level, numbers, frequency, load sequence et al.. Moreover, the permanent deformation behaviours of granular materials and soils are dependent on their stress history.

(2) The physical and mechanic behaviours of granular aggregates and soils themselves. The physical properties include saturation conditions, moisture content, density, grading, fines content, aggregate type, aggregate shape (angularity and roundness) and so on. The mechanic behaviours are both static properties and dynamic properties of foundation materials.

(3) Construction technology and quality. The critical load-carrying period for the pavement foundation is during construction, when the construction traffic loads are applied directly to the subbase. The construction processes may have an important influence.

(4) Other factors. Temperature and moisture are two climatic factors that affect pavement deflections.

Brown has given a comprehensive review of the state of the art of pavement soil mechanics in the 36th Rankine Lecture[2]. He pointed out that there is an urgent need to

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improve the application of soil mechanics in pavements. As a problem in pavement soil mechanics, rutting in flexible pavement has also been poorly understood in both Pavements and Geotechnics. Additionally, in comparison to resilient behaviour, few research has been devoted to plastic response and permanent deformation development in pavement foundations. The reasons of this situation maybe relate to the complex factors affecting rut response of flexible pavement foundations. The first problem is perhaps lack of appropriate constitutive equations in the form of relationships between applied stress and the resultant plastic strain for given numbers of load applications. And, another reason is the fact that monitoring the buildup of permanent deformation in these materials is a very time-consuming and basically destructive process.

Mechanistic design methods for flexible pavement in current use are usually based on limiting only two critical pavement responses:

(1) Tensile strain at the bottom of the asphalt layer.

(2) Vertical compressive strain at the top of the subgrade for overall pavement rutting.

The main shortcomings of this criterion are listed as follows:

(1) The elastic deformation isn't a good measure of the permanent deformation.

(2) The design methods assume that rutting in granular base and subbase layers is small for all cases, and don't take into account the deformation mechanisms of pavement foundations, such as consolidation, distortion, and attrition.

On the other hand, practical prediction and engineering calculations for rut depth are traditionally based on total stress approaches, but there is an argument that realistic prediction of the behaviour of soil masses can only be achieved when the model considers the interaction of the soil skeleton and the pore fluid. The vehicular loading generates an increase of the pore water pressure in pavement foundations, while simultaneously there is dissipation of the pore water pressure due to consolidation. Effective stresses have been shown to control the deformation behaviour of granular materials and soils. The moduli are functions of the level of effective stress and therefore excess pore water pressures must be continually updated during analysis, and their effects on the moduli taken progressively into account.

The main objective of this paper is to predict the rutting of the foundation of a flexible pavement under repeated vehicular loading. The paper presents the followings:

(1) A dynamic calculation model including a set of relationships of stress, strain, pore water pressure buildup and permanent deformation for predicting the response of flexible pavement foundations under cyclic vehicular loading.

(2) A dynamic effective stress based coupled analysis of a framework for the generation and dissipation of pore-water pressures and the rutting of the foundation of a flexible pavement.

(3) An illustrative example given for the prediction of rutting in a flexible pavement.

2 Dynamic calculation model of soil

The following discussions about soil properties were based on triaxial and resonant column tests for Shanghai soils at Tongji University.

A hierarchy of constitutive models is available for the dynamic response of soils to cyclic loading. The models range from the relatively simple hysteretic nonlinear models to complex elastic-kinematic hardening plastic models. Elastic-plastic methods are usually based on a kinematic hardening theory or a boundary surface theory[3,4]. These methods are the rigorous and precise physic and mechanic approaches. If the actual loading paths are in accordance with the stress paths used in calibrating the model, the predictions are good. But they sometimes incorporate some parameters not usually measured in field or laboratory test and make heavy demands on computing time.

Equivalent linear procedures seem to work quite well provided the behaviour of the structure is not strongly nonlinear and significant pore pressures do not develop. To simplify the research relatively, a viscoelastic model was developed in this paper, which can model nonlinear behaviour in terms of effective stresses, to provide the generation and dissipation of pore water pressures and to predict the rutting of a flexible pavement foundation under cyclic loading. Only the results of resonant column tests and dynamic triaxial tests are needed to obtain its parameters. It is important to recognize, compared with elasto-plastic models, this model simulates the soil dynamic behavior in a cycle of loading as a whole, not in detail.

2.1 Maximum shear modulus, G_{max}

Maximum shear modulus, G_{max}, is also called initial shear modulus or low-amplitude shear modulus. It represents the slope of stress-strain backbone curve at the origin. Laboratory tests have shown that soil stiffness is influenced by cyclic strain amplitude, void ratio, mean principal effective stress, plasticity index, number of loading cycles, and overconsolidation ratio. A large amount of experimental data has been accumulated to evaluate the shear modulus of various soils at very small levels of strains.

G_{max} can be estimated in several different ways. Generally, the measured shear wave velocities v_s obtained by most seismic geophysical tests can be used to compute G_{max} as

$$G_{max} = \rho v_s^2 \tag{1}$$

Another way is the empirical relationships between G_{max} and parameters of various in situ test (SPT, CPT, DMT, PMT et al.)[5~8]. The maximum shear modulus is also obtained from laboratory test data. The maximum shear modulus can be expressed as:

$$G_{max} = AF(e)(\sigma_0')^n \tag{2}$$

where $F(e)$ is a function of the void ratio, σ_0' is the mean

Table 1　Empirical relationships of maximum shear modulus for sand and clay: $G_{max} = AF(e)(\sigma_0')^n$[9]

Soil type	References	A	$F(e)$	n	Soil Material	Test method
Sand	Hardin-Richard(1963)	7000	$(2.17-e)^2/(1+e)$	0.5	Round grained Ottawa sand	Resonent column
	Hardin-Richard(1963)	3300	$(2.97-e)^2/(1+e)$	0.5	Angular grained crushed quartz	Resonent column
	Shibata-Soelarno(1975)	42000	$0.67-e/(1+e)$	0.5	Three kinds of clean sand	Ultrasonic column
	Iwasaki et al. (1978)	9000	$(2.17-e)^2/(1+e)$	0.38	Eleven kinds of clean sand	Resonent column
	Kokusho (1980)	8400	$(2.17-e)^2/(1+e)$	0.5	Toyoura sand	Cyclic triaxial
	Yu-Richard(1984)	7000	$(2.17-e)^2/(1+e)$	0.5	Three kinds of clean sand	Resonent column
Clay	Hardin-Black(1968)	3300	$(2.97-e)^2/(1+e)$	0.5	Kaolinite, etc.	Resonent column
	Marcuson-Wahls(1972)	4500	$(2.97-e)^2/(1+e)$	0.5	Kaolinite, $I_p = 35$	Resonent column
	Marcuson-Wahls(1972)	450	$(4.4-e)^2/(1+e)$	0.5	Bentonite, $I_p = 60$	Resonent column
	Zen-Umehara(1978)	2000~4000	$(2.97-e)^2/(1+e)$	0.5	Remolded clay, $I_p = 0\sim 50$	Resonent column
	Kokusho et al. (1982)	141	$(7.32-e)^2/(1+e)$	0.6	Undisturbed clays, $I_p = 40\sim 85$	Cyclic triaxial

Note: σ_0': kPa, G_{max}: kPa

Table 2　Empirical Relationships of maximum shear modulus for gravels: $G_{max} = AF(e)(\sigma_0')^n$[10]

References	A	n	Material	Sample size	Test method
Prange(1981)	7230	0.38	Ballast, $D_{50} = 40$ mm, $U_c = 3.0$	Dia: 100 cm, Length: 60 cm	Resonant column
Kokusho et al. (1981)	13000	0.55	Crushed rock, $D_{50} = 30$ mm, $U_c = 10$	Dia: 30 cm, Length: 60 cm	Triaxial
Kokusho et al. (1981)	8400	0.60	Round gravel, $D_{50} = 10$ mm, $U_c = 20$	Dia: 30 cm, Length: 60 cm	Triaxial
Tanaka et al. (1987)	3080	0.60	Gravel, $D_{50} = 10$ mm, $U_c = 20$	Dia: 10 cm, Length: 20 cm	Triaxial
Goto et al. (1987)	1200	0.85	Gravel, $D_{50} = 2$ mm, $U_c = 10$	Dia: 30 cm, Length: 60 cm	Triaxial
Nishio et al. (1985)	9360	0.44	Gravel, $D_{50} = 10.7$ mm, $U_c = 13.8$	Dia: 30 cm, Length: 60 cm	Triaxial

Note: $F(e) = (2.97-e)^2/(1+e)$

principle effective stress, A is a constant, n is a stress exponent. A summary of these empirical formulae was provided for sand, clay and gravel in Table 1 and 2 respectively.

If attention is drawn to the overconsolidated clay, Eqn (2) should be modified as

$$G_{max} = AF(e)(\mathrm{OCR})^k(\sigma_0')^n \tag{3}$$

where k is an overconsolidation ratio exponent.

In this paper, the following relationship between G_{max}, the void ratio e, the over consolidation ratio OCR, and the confining pressure σ_0' has been used:

$$G_{max} = D\frac{\mathrm{OCR}^k}{0.3+0.7e^2}p_a\left[\frac{\sigma_0'}{p_a}\right]^{0.5} \tag{4}$$

where p_a is atmospheric pressure, and k is a coefficient which relates to plasticity index of soil. Based on the above relationship and experimental data, an acceptable curve fitting for Shanghai saturated soft soil is obtained as follows: $D = 353$ for clay, $D = 451$ for silt, and $D = 485$ for sand. In this paper, all soils were treated as normally consolidated soil, then OCR= 1.

2.2　Strain-dependent modulus and damping

It is well known that the deformation characteristics of soil are highly nonlinear and this is manifested in the shear modulus and damping ratio, which vary significantly with the amplitude of shear strain under cyclic loading.

The proposed expression of the secant shear modulus G at a strain amplitude γ is

$$\frac{G}{G_{max}} = 1 - H(\gamma) \tag{5}$$

In the model, the function $H(\gamma)$ is

$$H(\gamma) = \left[\frac{\left(\frac{|\gamma|}{\gamma_r}\right)^{2B}}{1+\left(\frac{|\gamma|}{\gamma_r}\right)^{2B}}\right]^A \tag{6}$$

where γ_r is a reference or yield strain; and A and B are two dimensionless parameters.

It is suggested that values of γ_r for Shanghai saturated soft soil could be determined by the empirical relationship

$$\gamma_r = C\cdot\sqrt[3]{\sigma_0'} \tag{7}$$

where σ_0' is the effective mean principal stress in kPa, and C is an empirical parameter. Table 3 shows a summary of the experimental numerical values for the three parameters A, B, and C obtained for Shanghai saturated soft soil.

Table 3　Reference value of parameters: A, B, and C

Soil type	A	B	C
Clay	1.62	0.42	0.00013
Silt	1.12	0.44	0.00017
Sand	1.10	0.48	0.00022

The curve of shear modulus ratio G/G_{max} of Shanghai clay with γ is compared with the experimental data in Figure 2.

For Shanghai saturated soft soil, the variation of the damping ratio, D, with strain level is

$$\frac{D}{D_{max}} = \left[1-\frac{G}{G_{max}}\right]^\beta \tag{8}$$

where $D_{max} = 0.30$ for clay, $D_{max} = 0.25$ for silt and sand; and $\beta = 1.0$. A comparison between the proposed model and experimental data is shown in Figure 3.

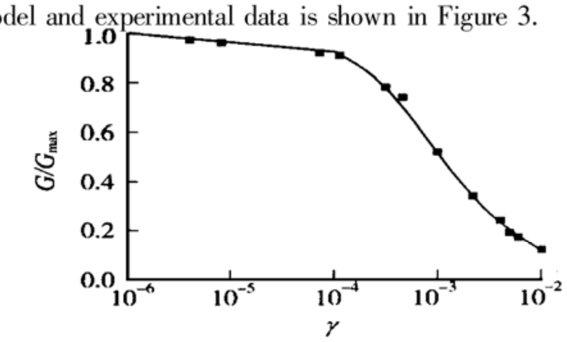

Fig. 2 The relationship between shear modulus ratio and shear strain of Shanghai clay

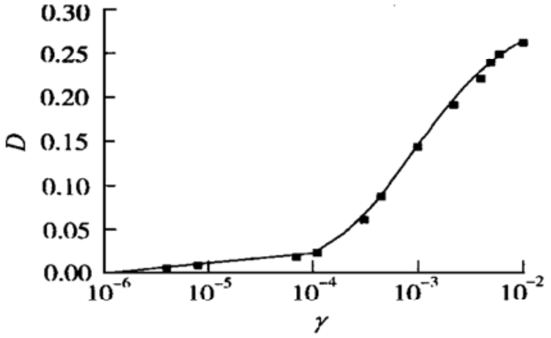

Fig. 3 The relationship between damping ratio and shear strain of Shanghai clay

2.3 Pore water pressure buildup

Laboratory tests can reveal the manner in which excess pore pressure is generated. On the basis of the results from undrained cyclic triaxial test, the pore water pressure buildup of Shanghai clay and silt may be expressed as

$$p/\sigma_0' = aN^b \tag{9}$$

where p is the pore water pressure; N is the number of uniform stress cycles; and a and b are two experimental parameters which are determined by the ratio of the dynamic shear stress to the effective confining pressure. Table 4 shows the reference value of a and b for Shanghai clay and silt. The curve of pore water pressure ratio p/σ_0' and N of Shanghai mucky clay is compared with the experimental data in Figure 4.

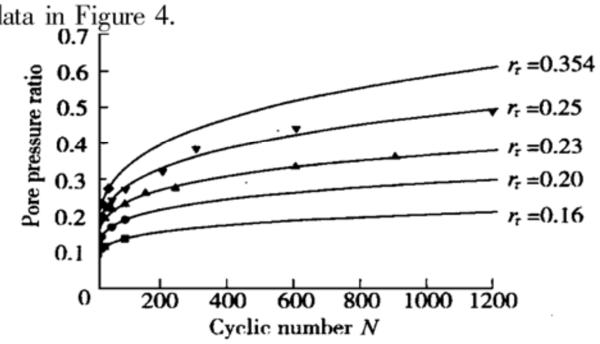

Fig. 4 Variation of pore water pressure ration and N of Shanghai mucky clay

Table 4 Reference value of a and b for Shanghai clay

Soil type	a	b
Clay	$0.274\ r_\tau^{0.767}$	$0.375\ r_\tau^{0.431}$
Silt	$0.273\ r_\tau^{0.711}$	$0.348\ r_\tau^{0.394}$

Note: $r_\tau = \tau_d/\sigma_0'$; τ_d = dynamic shear stress

For Shanghai sand, the development of pore water pressure in laboratory cyclic loading tests is of the form

$$\frac{p}{\sigma_0'} = (1 - ms_l)\frac{2}{\pi}\arcsin\left|\frac{N}{N_f}\right|^{\frac{1}{2\theta}} \tag{10}$$

where s_l is static stress level; m and θ are experimental parameters, for Shanghai sand $m = 1.1$ and $\theta = 0.7$; and N_f is the accumulative number of cycles at the same stress level required to produce a peak cyclic pore water pressure ratio of 100% under undrained conditions.

2.4 Permanent deformation

The deformation under cyclic loading can be divided into the volumetric and deviatoric components, ε^p and γ^p. The volume change, ε^p, comes from dissipation of excess pore water pressure and can be calculated through consolidation equation. For Shanghai saturated soft soil, γ^p is formulated according to the undrained cyclic triaxial tests as follows[11]

$$\gamma^p = \frac{r^*}{d - (d - 20)r^*} \tag{11}$$

where d is an experimental parameter, $d = 8$ for Shanghai sand, $d = 3$ for Shanghai clay. r^* is

$$r^* = \frac{r - r_s}{r_f - r_s} \tag{12}$$

where r_s is initial dynamic stress ratio; r_f is dynamic stress ratio at failure.

3 Dynamic analysis method

It is well known that the behaviour of geomaterials, and in particular of soils, is governed by the interaction of their solid skeleton with the pore fluid. The concept of effective stress is here of paramount importance. This paper was concerned with the interaction of the pore water with the solid soil skeleton in the flexible pavement foundations under fully saturated conditions. This two-phase behaviour belongs to the coupled problems Class II[12]. The coupling occurs through the governing differential equations of both solid mechanics and transient seepage.

The equations can be solved numerically under given boundary and initial conditions by the finite element method. The displacements $\boldsymbol{u}$ are described in terms of the nodal values $\bar{\boldsymbol{u}}$ as

$$\boldsymbol{u} = \boldsymbol{N}\bar{\boldsymbol{u}} \tag{13}$$

with a similar discretization for the pressures,

$$\boldsymbol{p} = \boldsymbol{N}_p\bar{\boldsymbol{p}} \tag{14}$$

where $\bar{\boldsymbol{p}}$ is the vector of nodal pressure values. $\boldsymbol{N}$ and $\boldsymbol{N}_p$ are appropriate shape functions. Performing spatial discretization, the finite element formulation are derived as

$$\begin{vmatrix} \boldsymbol{M} & \boldsymbol{0} \\ \boldsymbol{0} & \boldsymbol{0} \end{vmatrix}\begin{vmatrix} \ddot{\bar{\boldsymbol{u}}} \\ \ddot{\bar{\boldsymbol{p}}} \end{vmatrix} + \begin{vmatrix} \boldsymbol{C} & \boldsymbol{0} \\ \boldsymbol{Q}^T & \boldsymbol{S} \end{vmatrix}\begin{vmatrix} \dot{\bar{\boldsymbol{u}}} \\ \dot{\bar{\boldsymbol{p}}} \end{vmatrix} + \begin{vmatrix} \boldsymbol{K} & -\boldsymbol{Q} \\ \boldsymbol{0} & \boldsymbol{H} \end{vmatrix}\begin{vmatrix} \bar{\boldsymbol{u}} \\ \bar{\boldsymbol{p}} \end{vmatrix} = -\begin{vmatrix} \bar{\boldsymbol{f}} \\ \bar{\boldsymbol{q}} \end{vmatrix} \tag{15}$$

where

$$\boldsymbol{M} = \int_\Omega \boldsymbol{N}^T\rho\boldsymbol{N}\,d\Omega \tag{16}$$

is the well-known mass matrix.

$$\boldsymbol{Q} = \int_\Omega \boldsymbol{B}^T\boldsymbol{m}\boldsymbol{N}_p\,d\Omega \tag{17}$$

is a couple matrix with m being a vector equivalent to the Kronenecker.

$$S = \int_{\Omega} N_{\mathrm{p}}^{\mathrm{T}} \frac{1}{Q} N_{\mathrm{p}} \mathrm{d}\,\Omega \tag{18}$$

is a compressibility matrix (frequently taken as zero), where Q is related to the compressibility of the fluid.

$$K = \int_{\Omega} B^{\mathrm{T}} DB \mathrm{d}\,\Omega \tag{19}$$

is a stiffness matrix.

$$H = \int_{\Omega} (\nabla N_{\mathrm{p}})^{\mathrm{T}} k \nabla N_{\mathrm{p}} \mathrm{d}\,\Omega \tag{20}$$

is a permeability matrix, where k is the permeability. $\bar{f}$ is nodal load vector, $\bar{q}$ is nodal seepage discharge vector, C is a damping matrix. In this paper, Reyleigh damping was used and then C is assumed to be

$$C = aM + bK \tag{21}$$

where a and b are two Rayleigh damping factors. The natural frequency of pavements typically varies from 10~ 15 Hz[13, 14].

Many alternative time integration procedures are available for above equations, such as Newmark method and Wilson- θ method.

The main analysis procedures are as follows:

(1) Calculate initial static state of soil.

(2) Determine parameters of the dynamic calculation model.

(3) Calculate dynamic state of soil.

(4) Calculate pore water pressure increment and undrained residual strain.

(5) Repeat step 2~ 4 until the end of cyclic loading.

(6) Continue post-cyclic static analysis until the full dissipation of pore water pressure.

4 Application to the rutting of a flexible pavement subgrade

Three formulations typically have been considered for pavement modelling: plane strain, axisymmetric, and three dimensional. However, the problem for pavements is essentially one of plane strain, since the strains causing rutting develop in the transverse direction, no net strains accumulated longitudinally. And the plane-strain model requires relatively little computational time and memory.

A two-dimensional (2-D) plane-strain computer program was written to determine the rutting in flexible pavement foundations. It gives the total rut depth due to shear deformation as well as the volumetric deformation due to the pore water pressure generated.

A flexible pavement consisting of 15 cm asphalt over a clay subgrade was considered under plane strain condition. It was assumed that the pavement subgrade is saturated. The pavement section is shown in the Fig. 5 with an aggregate fill overlying a soft clay subgrade. The clay layer is assumed to overlay rigid rock.

The region of interest is discretised into four-node quadrilateral elements under plane strain conditions. The side boundaries are assumed fixed only in the horizontal direction. Horizontal movements are restricted at the side

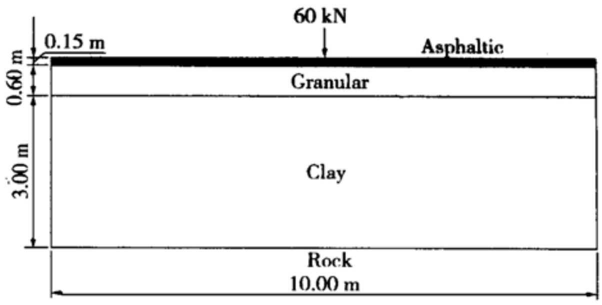

Fig. 5 Section through the pavement

boundaries and vertical movements are restricted at the bottom. A cyclic load of 60 kN was assumed to be applied centrally. A load frequency of 2 Hz was applied over 20 cycles.

Initial static stress state has an important effect on soil dynamic behavior. It should be determined before dynamic analysis. The initial stress state was obtained by using the Duncan- Chang model for the granular and soils in the paper.

$$E_{\mathrm{s}} = Kp_{\mathrm{a}} \left| \frac{\sigma_3}{p_{\mathrm{a}}} \right|^{n} \left| 1 - \frac{R_{\mathrm{f}}(1 - \sin\varphi)(\sigma_1 - \sigma_3)}{2c\cos\varphi + 2\sigma_3 \sin\varphi} \right| \tag{22}$$

where σ_1, σ_3 are maximum and minimum principal stress; K and n are dimensionless modulus number and exponent; R_{f} is failure ratio; c and φ are cohesion and angle of internal friction of soil; and P_{a} is atmospheric pressure. The variation of Poisson's ratio of soil is neglected here. The model parameters used for this analysis are shown in Table 5. The soil parameters were taken from the parameters identified for Shanghai soil.

Table 5 Calculation material parameters

Calculation parameters	Granular material	Clay
$\rho/(\mathrm{kg \cdot m^{-3}})$	2100	1900
c / kPa	0	14
$\varphi/(°)$	30	12
K	500	300
n	0. 5	0. 5
R_{f}	0. 8	0. 8
Poisson's ratio	0. 3	0. 4
Permeability/ $(\mathrm{m \cdot s^{-1}})$	1.5×10^{-4}	1.5×10^{-7}

The other dynamic properties of pavement foundation materials were those used in the above proposed dynamic constitutive models. Because there were no tests of gravel materials in the paper, their properties were assumed the same as those of sands.

Figure 6 shows the rutting development of pavement surface at the loading point. The maximum rut depth is 5. 34 mm. It is noted that at the end of cyclic loading, the transient settlement of pavement is only 2. 15 mm and the main rutting is developed after the end of loading. Therefore, the rutting of pavement foundations should be estimated by the step-by-step numerical computation until the complete dissipation of excess pore water pressure induced by cyclic loads.

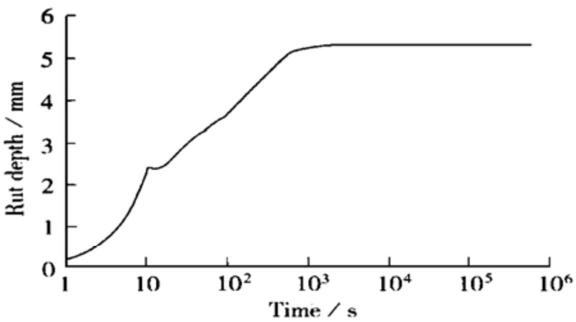

Fig. 6 Predicted rutting development of foundation

5 Conclusions

A procedure to predict rutting in flexible pavement foundations is presented. This is based on the dynamic coupled analysis method for soil-pore water interactions.

On the basis of resonant column test and dynamic triaxial test data of Shanghai soil, a new simple viscoelastic model including a set of relationships of stress, strain, pore water pressure and permanent deformations is developed. The procedure to calculate the rutting with the dynamic model is also reported in detail.

The procedure and model need to be verified by experimental data from full-scale pavements before recommendations can be made for design purpose.

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关于召开“建设行业信息系统相关软件通用标准技术研讨会”的通知

由建设部信息化工作领导小组办公室、建设部科技司、建设部标准定额司主办,建设部建筑制品与构配件产品标准化技术委员会、北京理正软件设计研究院和中国建筑标准设计研究所承办的建设行业信息系统相关软件通用标准技术研讨会定于在北京召开。现将会议有关事项通知如下:

1. 会议内容: ①国内外相关信息技术及标准的发展情况介绍; ②工程建设地理信息系统软件通用标准技术研讨; ③建设企业管理信息系统软件通用标准技术研讨; ④建设信息平台数据通用标准技术研讨; ⑤国内外先进技术演示交流。

2. 会议时间: 2001 年 12 月 20~ 21 日, 12 月 19 日报到。

3. 会议地点: 岭南饭店(地址: 北京市海淀区岭南路 34 号)。

4. 联系方式: ①北京理正软件设计研究院 地址: 北京市西城区车公庄大街甲 4 号物华大厦 A1108 邮编: 100044 联系人: 董青, 姚凤英 电话: 010- 68002237/ 8 转 604/ 605 传真: 68002237/ 8 转 603 E- mail: suggest@ lizheng. com. cn; ②建设部信息化工作领导小组办公室 地址: 北京市三里河路 9 号 邮编: 100835 联系人: 尚春明、仝贵婵 电话: 010- 68394535, 68393282 传真: 010- 68394530 E- mail: tonggch@ mail. cin. gov. cn

(北京理正软件设计研究院 供稿)