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Dear researchers, colleagues and readers,

Transportation is the means by which all people are connected, all human activities occur. Nowadays, in strong globalizing process, community activities have not been limited by countries’ borders; thus transportation becomes non-confrontiers.

We, transportation makers, in this moment, have had a common forum to together discuss, contribute, share and dedicate.

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We wish you and our transportation career were achieved, prosperous and fruitful.

Science is non-limitation,
Transportation is non-border,
Friendship is non-confrontiers,
Aim toward the future, we will do our best to make transportation:
More intelligent and effective,
Faster and safer,
Cleaner and greener,
With that objective, by this forum, we together connect, endeavour, research, create, contribute, share and devote.

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Planning for coastal road using tide resistance cement & concrete
APPLICATION OF CONTINUOUS COMPACTION TECHNOLOGY IN THE QUALITY CONTROL AND ACCEPTANCE OF HIGH SPEED RAILWAY SUBGRADE

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Summary: Based on the theory of continuous compaction technology, we put forward the VCV (Vibratory Compaction Value) index measured by a roller compactor, in order to evaluate the quality of subgrade compacted in a continuous way. The relationships between the VCV index and the currently used indexes K30, Evd, Ev1 and Ev2, was explored by analyzing the experimental data collected from two test sections on the Jing-Hu High Speed Railway. The result shows that the VCV index has a linear relationship with the currently used indexes.

Keywords: Continuous Compaction Control Technology; Compaction Quality; Relationships; Control and Acceptance.

INTRODUCTION

With the rapid development of high-speed railways in China, the requirement for the filling and compaction quality of railway subgrade has become increasingly strict. To ensure safe and smooth train operation, subgrade structure is supposed to meet higher standards in stiffness, strength, stability and uniformity, which lies in the control on compaction quality. In relevant technical standards, indexes for the quality control on subgrade compaction mainly include subgrade index K30, dynamic deformation modulus Evd, deformation modulus Ev1 and Ev2 [1]. These indexes are determined by time-consuming point tests. The results of tests, however, partially reflect the compaction quality of subgrade, incapable of describing the properties of subgrade as a whole. Moreover, point tests are inapplicable to subgrades of coarse aggregate. The incapability of point test and the need to obtain the quality information in real time [2-4] give rise to the continuous compaction control (CCC) technology.

The CCC technology was originated in 1970s [5]. Early researches in Sweden have revealed that the vibration characteristics of the drum are not stable as the subgrade is compacted. The early work led to the adoption of compaction meters in Europe for almost 30 years. Furthermore, the improved knowledge of drum/soil interaction has enabled the extraction of
more relevant soil properties, e.g., stiffness and modulus.

In this paper, we introduced the VCV index measured by a roller compactor with monitoring functions, to evaluate the quality of subgrade compacted. Then we overviewed the continuous compaction indexes, and analyzed the relationships between the VCV index and the currently used indexes.

I. BASIC THEORY OF CONTINUOUS COMPACTION TECHNOLOGY

During compaction, the drum’s response of a roller compactor varies following the different status of compacted soil. When the soil is relatively loose, the drum’s vibration response complies well with the sinusoidal oscillating signal excited by the roller compactor. With an increase in the number of compactions, the soil becomes more compressed and acts a stronger force on the drum, thus causing a distortion in the in-situ signals. Apart from the principal frequency, the response signal also includes some other frequency components, of which the first harmonic frequency is major (see fig 1). Given the fact that the degree of distortion relates to the stiffness of materials being compacted, the vibration signals can work as an indicator of the compacted soil status. The roller compactor automatically adjusts vibration frequency and amplitude, according to the variation of soil mechanics, until the target is achieved.

Fig 1. Frequency features of the response signal of excitation drum under different compaction status

II. CONTINUOUS COMPACTION INDEX

In 1976, GEODYNAMIK and DYNAPAC in Sweden jointly developed a compaction meter to measure an evaluation index CMV, a ratio of the principal frequency to the first harmonic frequency. Thereafter, adopting a terrameter, Bomag company in German successfully measured the Omega value, which reflects the amount of energy transmitted to soil, and further converted it into $E_{vib}$ with a capability to provide the kinetic modulus of soil; Ammann company in Switzerland introduced a stiffness index $K_s$ as the assessment of mechanistic soil properties. Though the evaluation indexes are different, they all need to measure the dynamic drum responses of roller compactors.

CMV index is applicable to the case when vibration between the roller compactor and the
compressed materials is linear one, e.g., compacting fine granular material. When compacting coarse filling materials, however, the dynamic response signals often include several other frequency components. For harder materials, the excitation drum “jumps” transiently, which causing an abnormal decrease in CMV index, i.e., the CMV index failing to reflect the stronger stiffness of harder materials. Meanwhile, a variety of parameters of a roller compactor must be known before calculating $K_s$ and $E_{vib}$, such as the excitation forces, masses of vibration elements, excitation frequency, and travelling velocity. Thus, based on the elastic/plastic and vibration theories, we presented a kinematics method as an application proposal under CCC technology, in which the subgrade response force is used as an index for the evaluation of compaction qualities. For the modelling purpose, the roller compactor is assumed to be separated with the material during compaction. Here, we analyze the excitation drum, and consider the force imposed by the subgrade to the roller. A one-degree of freedom kinetics model is established:

$$F(x) = Mx + Psin\omega t$$  

(1)

Where $M$ denotes mass of vibration elements; $F(x)$ subgrade structure response, $X$ model displacement; $P$ excitation force and $P = me\omega^2 = 4\pi^2 mef^2$; $e$ eccentric moment; $m$ eccentric mass; and $\omega$ vibration circular frequency.

Equation (1) is virtually nonlinear, and hard to solve. In view of the relationship between the subgrade response force and its plastic deformation, the subgrade response force is able to reflect the variations of the compaction status, and can be used as the continuous index to evaluate the compaction quality of subgrade\textsuperscript{[7]}. Theoretical analyses\textsuperscript{[8-11]} revealed the subgrade response has a linear relationship with the drum acceleration. Thus, the acceleration of excitation drum, a substitute of the response force index, is used as the continuous compaction index. After processing, the acceleration signal is transformed into the vibration compaction value (VCV). In this paper, we indentify the relationship between VCV and the subgrade response to validate it.

III. IN-SITU TEST

Two railway sections near Zhou County, Shandong Province, on the Jinghu High Speed Railway, were selected: DK564+035~185m (section A), and DK564+750~816m (section B). At both testing fields, vibration compaction test and plate loading test were carried out\textsuperscript{[12]}. The test points in conventional tests for $K_{30}$, $E_{vib}$, $E_{v1}$, and $E_{v2}$, were picked and grouped into three levels according to their VCV values, i.e, high, intermediate, low.

The subgrade width and thickness for sections A and B are 28 m and 30 cm. The compaction was performed for 15 drum traces. Section A was compacted in-situ, and section B is a fully compacted one, mainly for comparison purpose. The filling materials for both sections are of the similar sand soil.
Tab 1. Roller Compactor Parameters and Point Numbers in Conventional Tests

<table>
<thead>
<tr>
<th>Roller Compactor Parameters</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Working mass</td>
<td>20,000Kg</td>
</tr>
<tr>
<td>Excitation drum mass</td>
<td>10,000Kg</td>
</tr>
<tr>
<td>Excitation drum width</td>
<td>2,140mm</td>
</tr>
<tr>
<td>Working frequency</td>
<td>28/32Hz</td>
</tr>
<tr>
<td>Vibration force</td>
<td>355/220KN</td>
</tr>
<tr>
<td>The Total Amount of Points for Conventional Tests</td>
<td>220</td>
</tr>
<tr>
<td>$K_{30}$</td>
<td>80</td>
</tr>
<tr>
<td>$E_{vd}$</td>
<td>120</td>
</tr>
<tr>
<td>$E_{v1}$ plus $E_{v2}$</td>
<td>50</td>
</tr>
</tbody>
</table>

IV. RESULTS ANALYSIS

For the direct use of the VCV index in the quality control and acceptance of railway subgrade, the relationships need be established between the VCV index and the currently used indexes $K_{30}$, $E_{vd}$, $E_{v1}$ and $E_{v2}$, and this has been carried out by analyzing the experimental data taken from two test sections along the route of the Jing-Hu High Speed Railway project. To process a linear correlation analysis, the method of Lease double multiplication is applied, which is recommended by overseas standards\textsuperscript{[13]}. As in Sweden, the requirement for correlation coefficient $R$ shall not less than 0.60, as in German, $R$ shall not less than 0.70.

4.1. Relationship between VCV and $K_{30}$

Based on the experimental data collected from the two sections, it is concluded that there has a linear relationship between VCV and $K_{30}$, as shown in fig 2. The correlation coefficients are $R_A = 0.65$, $R_B = 0.75$, revealing. Compared with section B, the data of section A are more discrete distributed. This is because in case of section A, the filling and compaction work were just completed before testing, and thus the soil status of section A kept changing over time, which could increase the discreteness. In case of section B, the compaction work was carried out previously (more than 7 days), and the conventional tests were performed after the vibratory compaction test, ensuring a relatively stable soil state and eventually less discreteness.

![Graph 1: Relationship between VCV and $K_{30}$ from test sections A&B](image1)

*Fig 2. Relationships between VCV and $K_{30}$ from test sections A&B*
4.2. Relationship between VCV and $E_{vd}$

A series of $E_{vd}$ trace tests (4, 6 and 8 passes) were carried out in the rolling process of section A. The linear regression method was employed to analyze the correlation between two indexes, as shown in fig 3. The data in Fig.3 exhibit higher discreteness, probably because $E_{vd}$ tests were performed in a manner of batches, part of the tests conducted one day after rolling, and the rest 4 days after the completion of $K_{30}$ and $E_{v2}$ tests. If grouping the scattering points in fig 3 into L1 and L2, points in L1, which were tested lately, exhibit higher values. Fig 4 presented the test result of section B. With the correlation coefficient of 0.76, it is demonstrated that there lies a fairly good correlation between VCV and $E_{vd}$.

![Fig 3. VCV-E$_{vd}$ relationship from test section A](image)

![Fig 4. VCV-E$_{vd}$ relationship from test section B](image)

4.3. Relationship between VCV and $E_{v1}$, $E_{v2}$

Based on the experimental data for $E_{v1}$, $E_{v2}$ and VCV, a correlation analysis for two sections was carried out, as shown in figs 5 and 6. Due to the long duration for $E_{v2}$ test, the surface of the subgrade was developed into a heterogeneous status, causing high discreteness among its experimental data. In Fig.6, the correlation coefficients vary between 0.85 to 0.90, revealing a fairly good correlation between VCV and $E_{v1}$, $E_{v2}$.

![Fig 5. VCV-$E_{v1}$, $E_{v2}$ relationships from test Section A](image)

(Continuously see page 11)
THE MODERN SITUATION IN INLAND WATER TRANSPORT IN RUSSIA

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Summary: This article contains the comprehensive information concerning the modern situation in inland water transport in Russia. There is a dense network of rivers and canals in the Eastern part of the country. On the other hand after realizing some great infrastructure projects in the European part there is the deep-water transport network of total length 6,5 thousand kilometers, which takes half of the inland water freight traffic. The future of inland waterways is positive as it could be viable alternative to overcrowded roads and railways. The expansion of international cooperation of Russia with European states would make it possible in prospect to arrange freight transportation along the Big European Waterway Ring. On its part Russia takes steps to gradually opening the inland waterways for international navigation.

Kea words: Inland water transport, inland waterways, hydraulic facilities, deep-water transport network, modes of transport, navigation, shipping companies, vessels of the sea-river type, international transport corridors, Agreement on the most important international waterways in Europe, The Big European Waterway Ring.

Russian inland waterways or rivers with unique hydraulic facilities are the important natural asserts of this country. The length of the network of the navigable inland waterways of the country is over 100 thousand kilometers. All the network is open to public navigation for the carriage of goods and passengers. They are intensively used for both purposes, servicing the territory of 61 entities of the Russian Federation (out of 87 available). All types of transportation can be carried from April to November.

The inland waterways have always played an important role in the history of the Russian state. The state has always paid great attention to the necessity of waterways' improvement and regulation of the legal status of the water transport. As far back as 1649 the first Law put forward a guarantee for free navigation in the system of inland waterways. At the turn of the 18-th and the beginning of the 19-th century water transport economically experienced a stormy development. The further development of the waterways occurred in the 30-s to 60-s of the last
century. In 1933 the White Sea - Baltic Sea Canal was launched, followed by the unique canal, named after Moscow city, then Volga - Don canal in 1952 and finally in 1964 the existing Volga-Baltic Sea waterway was upgraded. Great hydraulic engineering facilities are being erected on the Volga, Kama, Nizny Don, Ob and Yenisey rivers. Finally the construction and upgrading hydraulic engineering facilities within the European part of the country have created there a deep-water transport network connecting the White Sea, the Baltic Sea, the Caspian Sea, the Black Sea and the Azov Sea. As the result the entire rout from St.Petersburg to Rostov-on-Don through Moscow, Perm and Astrahan has depths in high water periods of at least 4 meters excluding some areas. The overall length of the deep-water transport network of the European part of the country is 6,5 thousand kilometers. It takes more than a half of the tonnage of the inland waterways.

Similar projects of restoring and building infrastructure of inland waterways began in Europe after World War II. Nowadays the European Union has a dense network of rivers and canals, linking up the basins of the rivers which flow into the Atlantic and the North Sea. They are: Siene, Rhine, Scheldt, Elbe and Oder. Recently this network was linked to the Danube basin by the Rhine - Main - Danube Canal. In the six Member States which can use this network, inland waterway transport carries 9% of goods traffic.

River transport in Russia is part of the whole transport system and historically has taken one of the leading positions in servicing large industrial centers located on the waterways. The competitive advantages of inland water transport over the other modes of transport are well known. They are: high safety, environment friendly and energy-efficiency. So the navigable inland waterways provide a cost-effective means for moving major bulk commodities, such as grain, coal and petroleum. It takes little space and has the ability to transfer part of the tonnage from overcrowded roads and rail network.

River transport is of special importance for the northern and eastern regions of this country. In this parts of Russia there is a lack of railway infrastructure network, but a dense network of rivers. The ratio of inland waterways network length to surface area is twice as much as in the Russian Federation on average. That is why inland water transport carries 65% to 90% of goods traffic while its modal share in Russia on average in the year of 2009 is only 2-3%. It should be stressed that the importance of inland water transport in Russia' economy is not determined by the modal share - about 2-3%. But it is determined by the functions performed by it. They are: providing delivery of goods to the Polar regions of the country, which are difficult to reach by the other modes of transport. Also goods are carried to Siberia and the Far East regions, that is to the territories which are to the East of the Ural mountains. As the carriage of goods to these regions is very expensive, so the state financially supports the participated shipping companies.

By the 1-st of January of the year 2009 about 2700 companies have received licenses for commercial activities in inland water transport including over 1500 companies for transportation of cargoes and passengers. Only 30% of them carry 75% of goods traffic. Among the leading companies there are: North-West, Volga, Moscow shipping companies. The
volume of cargoes transported before the economic crisis of the 90-s was 500 million tons including 50 million tons carried to the Polar regions and 90 million passengers. Since 1991 the volume of cargoes transported and also the financial support of inland waterways maintenance have been reduced. In the year 2008 more than 150 million tons of cargo were transported, 25 million of which, that is about 20% of the above figure, were transported to the Polar regions. And about 30 million tons were international carriages.

International cargo goes by specially designed vessels suitable for both river and sea. Such vessels carry cargoes from Russian river ports to sea ports of different countries of Europe, Asia and Africa all year round. The most frequent are carriages to Scandinavian countries (Finland, Sweden), to the countries of Northern Europe (Holland, Denmark), Southern Europe (Italy, Greece, Turkey) and also to Germany, Belgium and China. There are about 700 vessels of the sea - river type. In the winter season when there is no navigation such vessels work at sea.

Under the conditions of the deepening international cooperation of Russia with European states one of the most important problems is the integration of transport systems on the entire European territory from the Urals to the Channel Tunnel and from the White and the Baltic Sea to the Mediterranean.

An important event in this direction was the summit of the ministers of transport of Europe and Asia in 1994 at which a concept of international transport corridors (ITC) was formulated and approved. There are 9 of them, three of which (NN 1,2 & 9) run through the territory of Russia. There is the river port of Kaliningrad within ITC N 1 which is part of the Kaliningrad integrated sea port. ITC N 2 starts in Germany, runs through Poland, Belorussia and then Russia (through Moscow & Nizny Novgorod) and further towards the Urals, Siberia and Middle Asia. The largest river transshipping complexes of Moscow and Nizny Novgorod are situated within ITC N 2. ITC N 9 connects the countries of Northern Europe (Finland) and Baltic countries with the Caspian, the Azov and the Black Sea basins. ITC N 9 includes the inland waterways of the Volga and the Don, the Volga - Baltic Sea waterway and the Volga - Don Canal and river ports situated on these waterways. A special feature of ITC N 9 is a possibility of connection with ITC N 7 (the Danube river). This provides the possibility of the cargo transportation from Russia to the countries of Middle and Southern Europe.

In 1996 in Geneva the Agreement on the most important international waterways in Europe was adopted. In accordance with this Agreement the deep water transport system of the European part of Russia is classified as a main waterways (directions E 50, E 60 and E 90. The Nizhniy Don, the Kama, the Volga, the Svir, the Neva and Ladoga, Onega and Beloye lakes with the canals connecting them are included in this system.

Thus, in prospect it’s possible to arrange international cargo transportation along the Big European Waterway Ring. It includes waterways from St. Petersburg to Volgograd, the Volga-Don Canal, the Azov and Black Seas, the Danube-Maine-Rhine waterway and further the North and Baltic Seas to St. Petersburg.
The problem is that vessels under foreign flags are not allowed to enter the Russian inland waterways and also Russian ships can’t enter the Danube, the Rhine and other European rivers.

On its part Russia takes steps to gradually opening the inland waterways for international navigation. For example, during the navigation some foreign ships visit a number of Southern-Russian ports (Rostov, Azov, Donetsk, Astrakhan and others). They do it only according to the special state permission. However a complete opening of the inland waterways of the Russian Federation for international navigation must be based on the parity approach to this problem. In order to open the Russian inland waterways for international navigation big financial investments and technological improvements must be made. Some locks and dams must be renovated and some new constructed to improve the navigability of the rivers. It will provide the guaranteed four-meter depth along the deep-water transport network including ITC N9.

Thus the integration of transport systems of Russia and Europe can soon be mutually profitable to all sides and will effect on international cooperation and foreign trade.

APPLICATION OF CONTINUOUS COMPACTION TECHNOLOGY ...

(Following page 7)

V. CONCLUSIONS

Though the correlation coefficients meet the requirements, the degrees of correlations are not the same owning to.

(1) Subgrade Structural Variation. The status of subgrade changes over time, which accounts for the poor repeatability of plate loading tests.

(2) Effective area. The VCV is a value averaged over the width of drums. The effective area in a plate loading test merely accounts about 3.5% of compaction test area.
(3) Operational Errors. Errors caused by improper operation in tests are virtually inevitable. The locations of some points in plate loading tests for $K_{30}$, $E_{vd}$, $E_{v1}$ and $E_{v2}$ might be overlapped, which leads to a distortion of the results.

(4) Data Processing. In the case of hard subgrade, the bouncing/jumping of drum would contribute the decrease of VCV index, and fail to reflect the real properties of the soil. The deviation as shown in fig.3 provides the evidence for this case.

Through the comparison of the two test sections on Jing-Hu High Speed Railway, it is proven that, when railway subgrade is in stable state, there lies strong correlation between the vibration compaction value VCV and $K_{30}$, $E_{vd}$, $E_{v1}$ and $E_{v2}$.

The error analysis reveals that, the post completion status of subgrade had the utmost influence on the correlation results, and the uniformity of subgrade as well as the dimensional effect also affected the final results.

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INTRODUCTION TBN METHOD IN Z-DOMAIN USING IN SIGNAL PROCESSING

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MC. PHD. NGO THANH BINH

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Summary: This paper refers to a method of digital signal processing based on the z-Domain for PVA model (Position Velocity Accelleration model), and develops the TBN algorithm (calculation of N-mean discrete time) to improve the quality of controllers used in trajectory moving objects.

Key words: Kalman Filter, PVA, TBN, GPS/INS Integration, Trajectory moving objects.

I. GENERAL RECOGNITION

In GNSS systems (Global Navigation Satellite Systems), Kalman filters are often used to limit influence of noise to obtain accurate results. Different types of filters are offered different computational tools (KF - Kalman Filter, EKF - Extended Kalman Filter, UKF - Unscented Kalman Filter), mainly applied to model PV-T (Position Velocity - Time) in the integrated GNSS and INS (Inertial Navigation System) or other sensors. The most studied integrated systems used in trajectory moving objects is GPS/INS system, considering the continuous system in the time domain (Continuous time-invariant state-space model).

PVA model considered in the s-domain is explained by the Laplace operator:

\[
\begin{align*}
\eta(t) &\xrightarrow{1/s} a(t) &\xrightarrow{1/s} v(t) &\xrightarrow{1/s} p(t)
\end{align*}
\]

**Fig 1. State model in Laplace domain**

Implement the formula in time domain:

\[
\dot{x}(t) = Fx(t) + Lw(t)
\]  \hspace{1cm} (1)

Where: • the initial conditions \( x(0) \sim N(m(0), P(0)) \), caculate from compass and noise \( \eta \);
• $F$ and $L$ are constant matrices, which characterize the behaviour of the model;

• $w(t)$ is a white noise process with a power spectral density $Q_c$.

\[
\begin{bmatrix}
\dot{p}(t) \\
\dot{v}(t) \\
\dot{a}(t)
\end{bmatrix} =
\begin{bmatrix}
0 & 1 & 0 \\
0 & 0 & 1 \\
0 & 0 & 0
\end{bmatrix}
\begin{bmatrix}
p(t) \\
v(t) \\
a(t)
\end{bmatrix} +
\begin{bmatrix}
0 \\
0 \\
1
\end{bmatrix}
\eta(t)
\]  

(2)

We can get continuous time states by calculating heterogeneous states of the system by the formula:

\[
x(t) = e^{Ft}x(0) + \int_0^t e^{F(t-\tau)}g\eta(\tau)d\tau
\]

(3)

Where $e^{Ft}$ is the matrix exponential of $F$ and $x(0)$ is the initial condition.

In GNSS applications, to program the micro-processors we need to consider the domain of discrete states and find a way to represent the state of continuous-time vector $x(t)$ into the discrete state $x[k]$ as the appropriate status formula. The chosen methods are mainly based on the Taylor series expansion for formula (3) with $t = nT$, where $T_s = \frac{1}{f_s}$ is proper sampling interval that satisfied Nyquist theorem; $f_s$ is sampling frequency. However, the problems we consider here is positioning, and disregard constituting control blocks and the closed-loop stability as in the case of UVAs (Unmanned Aerial Vehicles), so we only consider a system without a controller. From there we apply different appropriate assessments and processors; the most common of which is to use Kalman filter.

Kalman filter is applied for filtering the error of this PVA model with:

\[
x_{hi} = \begin{bmatrix}
\delta r^n \\
\delta v^n \\
\varepsilon^n \\
d \\
b
\end{bmatrix}^T; 
F = 
\begin{bmatrix}
0 & 1 & 0 \\
0 & 0 & 1 \\
0 & 0 & 0
\end{bmatrix}; 
L = 
\begin{bmatrix}
0 \\
0 \\
0 \\
1
\end{bmatrix}
\]

Where $\delta r^n, \delta v^n$ are error of position and velocity in 3 directions; $\varepsilon^n$ is the bias toward, with yaw ($\psi$) is calculated from GPS or using electronic compass; $d$ is drift error (gyro); $b$ is bias error (accelerometer).
Input signals of the system and input signals of the filter need to be differentiated. Theoretical model suggests the system’s input signal as $x_{in} = \begin{bmatrix} \delta r^n & \delta v^n & \varepsilon_d & d & b \end{bmatrix}^T$. However, GPS signal only provides non-directional $P$, $V$ parameters. While measuring, if there’s any signal to be compared, then it will be filtered. Thus input signal of KF processor are just non directional $P$, $V$ errors; this is different from the system models in theory with assumptions that all parameters will give positive values. GPS signal contains 2 error values POS and Vel ($\delta$ non directional); we take 2 output signals as position error POS ($\delta r^n$) and velocity error Vel ($\delta v^n$); acceleration component has negligible value give the 3 row of matrix $F$ as $[0 \ 0 \ 0]$. When developing the matrix, at the initial position, we record $\nabla(t) = \eta(t)$; which is the error of INS when object stands still. Measuring noise $r_k$ is error due to GPS. Consequently, matrix $H$ and measuring noise $r_k$ are computed as following:

$$H = \begin{bmatrix} 1 & 1 & 0 \end{bmatrix} ; \quad r_k = \begin{bmatrix} \varepsilon_d & \varepsilon_v & 0 \end{bmatrix}^T$$

With model POS transmitting error Vel in a finite band-width; in another word only filtering $v^n$, matrix $H$ and measuring noise $r_k$ are computed as following:

$$H = \begin{bmatrix} 0 & 1 & 0 \end{bmatrix} ; \quad r_k = \begin{bmatrix} 0 & \varepsilon_v & 0 \end{bmatrix}^T$$

It can be said that this model does not reflect the accurate state of the system, thus leaving the calculated error to be significant. In addition, this model is not friendly with presenting digital systems, which means it would get too complicated once functions are formulated closest to a mathematical model with detailed mathematical formulas. This is the reason why extended Kalman filters have been developed to accommodate the complicated mathematical functions (Eun-Hwuan Shin, 2005; Sarkka, 2008). Another source of error is the noise from the systems in a continuous time state, which theoretically, is white noise with constant spectral density. This kind of noise is only theoretical as in an ideal model, and will be negligible in mathematical computations. Several filter models such as EKF2, AUKF ... try to accommodate heavy numerical data crunching, when developed in computations will be difficult because of time issue.
II. S-Z TRANSFORMATIONS

Matrix exponential according to formula (3) developed by Taylor series, simplified from (1) is as following:

\[ e^{FTs} = I_s + FT_s + \frac{FT_s^2}{2!} + \ldots \]  \hspace{1cm} (4)

Therefore the calculation for PVA model will be:

\[
\begin{bmatrix}
  p[n] \\
  v[n] \\
  a[n]
\end{bmatrix} = 
\begin{bmatrix}
  1 & T_s & \frac{T_s^2}{2} \\
  0 & 1 & T_s \\
  0 & 0 & 1
\end{bmatrix}
\begin{bmatrix}
  p[n-1] \\
  v[n-1] \\
  a[n-1]
\end{bmatrix} + 
\int_{0}^{T_s} v \eta(\tau)d(\tau) \hspace{1cm} (5)
\]

In signal processing, there are different processing rules with different simplification methods. Using the type IID (Independent and Identically Distributed random variables), analog noises of system \( H_s(f) \) will be divided into two two-sided bandwidths \( f_s \) and values of \( \eta[n] \) are computed by \( H_s(f) \). The power spectrum of noise sequence is lied within \( \left( \frac{f}{\frac{2}{f_s}}, \frac{f}{\frac{2}{f_s}} \right) \); we have \( \sigma^2_\eta = \frac{N_s F_s^2}{2} \). Transform into \( z \) domain with input points \( x[n] = x(nT_s) \), explain \( H_s(f) \) in the form of \( H(z) \) we’ll have output point \( y[n] \) as below:

![Fig 3. Simplifying system by time](image)

Thus output signal \( y(t) \) follow fig 3 will be developed as following:

\[ y(t) = \int_{-\infty}^{+\infty} H_s(f)X(f)e^{j2\pi ft} \, df \]  \hspace{1cm} (6)

To discrete (6) we transform:

\[ y(nT_s) = y[n] = \int_{-\infty}^{+\infty} H_s(f)X(f)e^{j2\pi fnT_s} \, df \]  \hspace{1cm} (7)

According to Papoulis (Signal Analysis, 1997), (7) will be developed as following:
\[ y[n] = \int_{-\infty}^{\infty} X(f)H(e^{j2\pi f_0})e^{j2\pi f_0} df \]  

(8)

Compared to (7), we can limit signal as:

1. \( X(f) = 0 \) khi \(|f| > B_s \)

2. \( H(e^{j2\pi f_0}) = H_a(f) \) khi \(|f| < B_s \)

Using transformation rule with approximately \( H_a(s) = \frac{1}{s} \), as following:

\[ a[n] = a[n-1] + T_\eta[n-1] \]

(9)

\[ v[n] = v[n-1] + \frac{T}{2}(a[n] + a[n-1]) \]

(10)

\[ p[n] = \frac{T}{3}(v[n] + 4v[n-1] + v[n-2]) + p[n-2] \]

(11)

III. COMPUTATION METHOD TBN FOR PVA MODEL

With TBN method, the authors separates 2 ranges of compensative errors which are positive range (+) and negative range (−) in order to avoid accumulation of error values. Positioning and velocity computations are within a finite range, and the following parameters:

\[ \eta_\eta[n] = a[n-1] + T_\eta[n-1] \]

\[ sTv[n] = v[n-1] + (a[n] + a[n-1])^2 \]

\[ sTp[n] = (v[n] + 4v[n-1]+ v[n - 2]) + p[n - 2] \]

Develop functions by the simplification in z domain we have:
\[ p[n] = p[n-1] + T_v[n-1] + \frac{T_v^2}{2} a[n-1] + \frac{T_v^3}{6} \eta[n-1] \quad (12) \]

\[ v[n] = v[n-1] + T_a[n-1] + \frac{T_a^2}{2} \eta[n-1] \quad (13) \]

\[ a[n] = a[n-1] + T_\eta[n-1] \quad (14) \]

This means the model PVA in z domain at this point would be represented in the following form:

\[
\begin{bmatrix}
    p[n] \\
    v[n] \\
    a[n]
\end{bmatrix} =
\begin{bmatrix}
    1 & T_v & \frac{T_v^2}{2} \\
    0 & 1 & T_v \\
    0 & 0 & 1
\end{bmatrix}
\begin{bmatrix}
    p[n-1] \\
    v[n-1] \\
    a[n-1]
\end{bmatrix} +
\begin{bmatrix}
    \frac{T_v^3}{6} \\
    \frac{T_a^2}{2} \\
    T_\eta
\end{bmatrix} \eta[n-1] \quad (15)
\]

In which \( \eta[n] \) is digital system noise. In practice, we will navigate this noise by keeping the object in still state and measure data at output INS. This is a random parameter, affected by operational temperature bandwidth and bias error data (accelerator), drift error data (gyro), and the original angle compared to the North axis \( \psi_n \). Data on \( \eta[n] \) multiplied by one calculation is the acceleration error (non directional). When computing, this value needs to be identified, so that the values for compensative POS, Vel can be calculated.

In order to program for the micro-processor, we use TBN methods as mentioned above in model (15) which gives the final result for \( p[j] \). The set of averages of 2 continuous points \( p[j], p[j+1], \ldots \) draws an orbit approximately close enough to the actual orbital movements.

**Computational rules following TBN method:**

Is continuous within a finite signal range. With any terrestrial vehicle, maximum acceleration is \( a_{\max} = \mu \times g \), whereas \( \mu \leq 1.0 \). Choose \( \mu_{\max} = 1 \), meaning the condition of \( a_{\max} \leq 9.81 \text{m/s}^2 \) is always satisfied. To simplify, we calculate with acceleration in 2D plane, limited by acceleration \( a_{\max} = 9.81 \text{m/s}^2 \), and maximum velocity at \( < 300 \text{ km/h} \). With \( v_{\max} \) known, in a period of time \( T_s \) we will be able to find the finite range value of velocity \( \Delta v_{\max} \) (technically \( |\Delta v_{\max}| \)); this means the distance between 2 continuously measuring points cannot be beyond this value. Multiply \( \Delta v_{\max} \) with duration of \( T_s \) will give the finite range value of \( \Delta p_{\max} \). At any measuring point, if sum of measuring value and noise is \( > \Delta v_{\max} \) then we chose \( \Delta v_{\max} \). The same steps are applied to \( \Delta p_{\max} \); unnormal error values, too positive or too negative error values beyond the ranges, can be eliminated.
Follows the orbit of averages between two continuous points. To avoid accumulation of errors, we add on error value every other measuring points, and subtract error value every 2 measuring points. Calculations can also be done by using amplitude values, in another word, add and subtract the absolute value of error. Basically, positioning points will be recorded as following:

\[
(x_0 + \varepsilon_1) \rightarrow (x_1 - \varepsilon_2) \rightarrow (x_2 + \varepsilon_3) \rightarrow (x_3 - \varepsilon_4) \rightarrow (x_4 + \varepsilon_5) \rightarrow \ldots
\]

With: \( \varepsilon_i = |\text{POS}_{i}^{\text{INS}} - \text{POS}_{i}^{\text{GPS}}| \)

Combining 2 calculation rules above, instead of always adding on error \( \hat{e} \) as in previous methods, we respectively add on 2 error values which are positive compensation \( (+\varepsilon_{2i+1}) \) and negative compensation \( (-\varepsilon_{2i+2}) \), under the condition of finite range being \( \leq \Delta p_{\text{max}} \). The graph will illustrate the average of 2 continuous values according to steps \((0)(1,2)(3,4)(5,6)\ldots\)

Mathematically, instead of graphing the orbital movement as the set of \( \sum_{i=0}^{n} \text{POS}_{i}^{\text{GPS}} \) when using GPS, or \( \sum_{i=0}^{n} \text{POS}_{i}^{\text{INS}} \) when using INS, or \( \sum_{i=0}^{n} (\text{POS}_{i}^{\text{INS}} + \hat{e}_i) \) when using filter; with TBN, orbital movement is the set of \( \sum_{i=0}^{n} (\text{POS}_{2i}^{\text{INS}} + \varepsilon_{2i+1}) \) and \( \sum_{i=0}^{n} (\text{POS}_{2i+1}^{\text{INS}} - \varepsilon_{2i+2}) \). Values of points on the orbit is:

\[
\text{POS}_{j+1}^{\text{real}} = \frac{(\text{POS}_{2i}^{\text{INS}} + \varepsilon_{2i+1}) + (\text{POS}_{2i+1}^{\text{INS}} - \varepsilon_{2i+2})}{2} = \frac{\text{POS}_{2i}^{\text{INS}} + \text{POS}_{2i+1}^{\text{INS}} + \varepsilon_{2i+1} - \varepsilon_{2i+2}}{2} \tag{17}
\]

\[
\text{POS}_{j+1}^{\text{real}} = \frac{\text{POS}_{2i}^{\text{INS}} - \text{POS}_{2i+2}^{\text{INS}}}{2} = \frac{\text{POS}_{2i+1}^{\text{GPS}} - \text{POS}_{2i+2}^{\text{GPS}}}{2} \tag{18}
\]

The set of \( \text{POS}_{j+1}^{\text{real}} \) in formula (18) will illustrate object’s orbital movement. TBN processor can be combined with KF processor and be added onto computations with input signal \( r^0 \); in order to improve signal tracking capability compared to regular Kalman filter.

IV. SIMULATION AND APPLICATIONS

In this article, the authors first work on simulation in Matlab with signal transmission plus random noise, in order to generate random error for sensor INS, and assumed measuring signal transmitted by function plus random noise in order to generate random error for GPS. Simulation results is positive, fluctuation of velocity varies widely, however fluctuations of positions is negligible, and simulation orbits tends to lead back to actual orbit after forced interference.
Next, the authors developed an algorithm combining GPS/INS that is practical. Within the allowed framework of scientific research code KC.06.02/06-10, the authors have initiated the development and manufacture of the front end device which was tested in monitoring train movement. Experimental results were positive. As we all know, at positions where the road platform is weak, or ties of the rail track are broken, then the train movement will fluctuate accordingly. In order to measure and monitor the downgraded rail ways, we can put measuring device on the rail track platform to record fluctuation values. The controller will record data, and calculate parameters, and transmit result to the central. Based on the recorded data about the oscillation degree of the train, we can tell the state of the railway and maintenance requirement.

Technical property of the method is to use sensor INS to identify acceleration and angle velocity compared to object’s coordinate axis (b-frame) computing oscillation of object in 2D plane. This combined with positioning algorithm in GPS.
can provide state of railway and suggest maximum velocity allowed for moving trains on each different rail track. Currently, the authors are working on other applications of TBN algorithm.

V. CONCLUSION

The fundamental of previous problems of processing data for the classic model PVA is to use controller units in time-invariant range in s domain. The new method, called TBN method, proposed here to is to calculate in time-variant range in z domain, where the states and noise are computed continuously by n steps and state of s is computed by z; graph is of average points of 2 continuous points.

Theoretical results give positive simulation outcomes. Currently, this method is in experimental trials to prove practicality. The basic feature of the algorithm is that it has solved the problem of accumulation of calculation errors by adding/subtracting error values continuous and takes the average of those in addition to the condition of finite range. This solution can be applied in many terrestrial objects, and also for UVA; while proposing a new prospective approach in navigational applications.

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[6]. Jouni, Sakkar (2008); Optimal filtering with Kalman filters and smoother; Helsinki University of Technology, P.O.Box 9203, FIN-02015 TKK, Espoo, Finland.
[7]. Thanh Binh Ngo, Hung Lan Le, Thanh Hai Nguyen (2009); Survey of Kalman Filters and Their Application in Signal Processing; Proceedings of AICI'09.
World experience shows the need for insurance of vehicles (cars, motorcycles) as well as road users in cases of traffic accidents (RTA). Insurance not only protects each owner of the vehicle from the economic losses in the event of an accident, but also carries out various activities aimed at reducing the accident rate in road transport.

In the late 20th centuries most of developed countries announced their national programs to reduce traffic accidents and policy organization of road safety. One of the aspects of vehicle insurance was international cooperation. Many countries joined IAIS (International Association of Insurance Supervisors).

IAIS promulgates rules and standards and insurance guidelines as well as provides training and assistance on insurance supervision and organization of workshops. The principles and standards of the Association are the general orientation and a basis for issuing regulations on insurance law agreed between the members, applying to the insurance management bodies, insurance companies and insurance associations.

In future each country independently adopts its legislation or regulations.

For example, by the agreement with Germany the Russian Union of Insurers (RSA) has used a committee model of "Green Card", and on January 1, 2009 Russia joined this system as well as other countries to address various key areas of exchange of delegations through seminars to improve their works.

The Agreement on the establishment of a "Green Card" among committee members in different countries and applying a uniform criteria have begun immediately after end of World War II.

**Summary:** The article deals with the insurance of vehicles in different countries, as well as gives an estimate of losses from road accidents in Vietnam. It shows the role of insurance in countermeasures to improve road safety.
In 1949 at the international conference on insurance in London recommendations of the Council of Economic and Social Affairs were approved and the Commission Office was formally established with headquarters in London. In 1951 the first official meeting of the Committee's Office was held and the model agreement between the committee member countries was approved and a uniform criteria was adopted for "Green Card".

Agreement on "Green Card" officially came into force on 01 January 1953. Accordingly, the civil liability insurance card for motor vehicles issued by any member country is valid on the territory of another member countries. Up to date the “Green Card” system has 45 countries from Europe, Asia and Africa - all countries in Europe including Russia (from January 1, 2009), as well as Turkey, Israel, Morocco, Tunisia and Iran.

In 2006, the headquarter of Commission Office moved to Brussels.

Table 1 shows the Member countries participating in the "Green Card" system.

<table>
<thead>
<tr>
<th>Member Country</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Limit for one victim</td>
<td>Limit for all victims</td>
<td>Limit for one vehicle</td>
<td>Limit for all vehicles</td>
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<td>50 000 000</td>
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<td>774 685</td>
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<tr>
<td>Israel</td>
<td>Unlimited</td>
<td>Unlimited</td>
<td>No compulsory insurance</td>
<td>No compulsory insurance</td>
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<td>1 000 000</td>
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<td>1 602 000</td>
<td>1 602 000</td>
<td></td>
</tr>
</tbody>
</table>
In some countries the damage is unlimited. This means that if the court recognizes the need to provide for life for families of victims, the insurance company will pay this fee.

In some countries in Africa and Asia the rare has been an international insurance system of its own, similar to the "Green Card", they are:

- Red Card - 6 countries in Central Africa.
- Brown Card - 14 African countries.
- Orange Card - 19 countries in Africa and Asia.

A numbers of mentioned countries participate in the Green Card system in Europe.

The above data is updated as of September 13th 2010.

In addition, most of countries are compulsory to participate in civil liability insurance for motor vehicle owners. This type of insurance can not protect motor vehicle owners for vehicle theft or failure due to natural disasters, floods ... Therefore, many vehicle owners purchase additional voluntary insurance vehicles "Casco" (from Spanish language, Casco is a body skeleton of the ship).

This type of insurance is to protect vehicle owners from physical damage due to theft, natural disasters, floods ... and it does not cover property insurance carrier. Insurance companies commit compensation when incidents happen under the provisions in the contract.

Vehicle insurance in Vietnam has been carried out since January 1st 1998 when implementation of Resolution № 30/HDBT on “regime of civil liability insurance for motor vehicle owners." According to this Resolution it is not compulsory for vehicle owners to buy...
motor vehicle insurance when the accident happens. The compulsory insurance for civil liability of motor vehicle owner is specified in the Circular No. 126/2008/TT -BTC by the Ministry of Finance on December 22nd, 2008. Motor vehicle insurance is the key insurance business in many insurance companies that effects significantly to the people.

A high growth rate of motor vehicles is one of the main reason contributing to the development activities of motor vehicle insurance in our country. Figure 1 describes the change in number of motorized vehicles in the period 2000 -2008. In this period the average growth rate of motor vehicles reached 22.25%, in which 10.85% of cars and 22.64% of motorcycles.

![Fig 1. The change in the number of motorized vehicles in Vietnam in the period 2000 - 2008](image)

The business of vehicle insurance in Vietnam includes the insurance of material body of vehicle, compulsory insurance for civil liability of motor vehicle owner, insurance for passenger car and driver, insurance of civil liability of vehicle owners for carrying cargo on the vehicle. Among of such insurances, the insurance of material body of vehicle has greater value both as a source of revenue and of compensation. However, the insurance of material body of vehicle reaches only 40% for motor vehicles and 15% for motorcycles; compulsory insurance for civil liability of motor vehicle owner also reaches only 75% for cars and 35% for motorcycles.

The fee applies to compulsory insurance of civil liability is 3USD/year for motorcycles, the average of 38 - 87USD/year for non - transport business cars, 43 - 177USD/year for transport business cars and 70-123USD/year for trucks, 100USD/year for buses. Level of responsibility for human damage caused by motor vehicles in case of death is $2,500/person/accident, in case of injury the highest compensation is $2,000 (depending on the injury rate). Level of liability for property damage caused by motor vehicles is $2,500/accident. Fee for passenger car is from $1 - $2.5/seat/person; the level of liability insurance when the accident occurred for passenger car is from $1000 - $2500 /person /accident.

In the first year of application of compulsory insurance for civil liability of motor vehicle
owner insurance, the participation rate is about 75% for cars and 35% for motorcycles. The regulations on sanctions, fines for owners of motorized vehicles on roads without insurance make more people participate in buying insurance. Therefore the market shares of this insurance business are very potential.

As of December 1, 2009 motor vehicle insurance is the one which has the highest revenue, accounting for 30% of total revenue for non-life insurance. In the period of 2000 - 2009 total revenues motor vehicle insurance increased more than 4 times from $52 million to $214.7 million. Figure 2 shows the amount of compensation for material damage of motor vehicle caused by traffic accidents over the years.

![Fig 2. Shows the total amount of compensation in each insurance operation from 2000 to 2009 (million USD)](image)

**Table 3. Compensation for different types of insurance for the years 2000-2009. (Million dollars)**

<table>
<thead>
<tr>
<th>Year</th>
<th>Compensation for victims in road accident (fatality and injured)</th>
<th>Compensation for material loss from damage to vehicles in road accident</th>
<th>Compensation for damage to carriage of goods in case of road accident</th>
</tr>
</thead>
<tbody>
<tr>
<td>2000</td>
<td>22,075</td>
<td>32,864</td>
<td>9,770</td>
</tr>
<tr>
<td>2001</td>
<td>28,128</td>
<td>35,220</td>
<td>9,690</td>
</tr>
<tr>
<td>2002</td>
<td>33,763</td>
<td>38,106</td>
<td>10,075</td>
</tr>
<tr>
<td>2003</td>
<td>26,592</td>
<td>41,719</td>
<td>10,730</td>
</tr>
<tr>
<td>2004</td>
<td>24,440</td>
<td>37,290</td>
<td>11,330</td>
</tr>
<tr>
<td>2005</td>
<td>22,094</td>
<td>37,835</td>
<td>11,725</td>
</tr>
<tr>
<td>2006</td>
<td>23,511</td>
<td>38,520</td>
<td>14,825</td>
</tr>
<tr>
<td>2007</td>
<td>23,881</td>
<td>56,150</td>
<td>19,465</td>
</tr>
<tr>
<td>2008</td>
<td>24,092</td>
<td>73,151</td>
<td>26,430</td>
</tr>
<tr>
<td>2009</td>
<td>30,492</td>
<td>85,511</td>
<td>25,128</td>
</tr>
</tbody>
</table>

Level of average compensation for a motorcycle accident in 2000 was $624, and it was increased to $1,105 in 2009. Level of average compensation for a car accident in 2000 was...
$2,602, and it was increased to $5,202 in 2008. The increase of compensation rate is due to the following reasons:

- Increase of 1.35 times in compulsory insurance fees;
- Increase of the value of contracts because of high cost of vehicles;
- Increase in costs for repairs and replacement of components and assemblies of vehicles;
- In the civil liability insurance for motor vehicle owners, compensation for human losses in most cases does not consider fault of vehicle owners and drivers.

Figure 3 shows the variation of numbers of traffic accidents, fatalities and injuries caused by traffic accidents over the years;

Figure 4 shows the cost of compensation for victims of traffic accidents for period 2000 - 2009.

Along with the development of the insurance companies, the level of compensation for traffic accident victims is also increasing year by year. e.g. One fatality was compensated $
1,500 in 2000 but increase to $2,500 in 2009; an average of one injured person was compensated $500 in 2000 but increase to $1,025 in 2009.

Some insurance companies introduce package of high quality insurance, participants receive the modern medical services with a team of skilled physicians therefore it is attracting more hard - to - pleased customers and promoting the insurance market.

One of the two insurance operations which were first developed in Vietnam over 45 years ago but so far, the revenue from cargo insurance fee obtained by Vietnam insurance companies is still very modest and far away from the other insurance operations which developed later. Turnover of imported goods with insurance occupies only about 35%; exported goods are more humble, never exceeded 5%.

Data on cargo transportation insurance business of transportation service enterprises shows that all insurance companies for cargo insurance are profitable.

![Material loss from damage to the carriage of goods in case of accident](image)

In spite of increasing in revenue of cargo insurance market of 2.8 times from 2000 to 2009 (from US$16.85 million to US$47.88 million), it is still not commensurate with its inherent potential. Compensation rate ranges from 52% -56%.

In comparison with the growth of imported goods turnover and the total quantity of national goods transportation, the growth of cargo insurance is still low and inadequate revenue.

Being evaluated as profitable business sector, transportation cargo insurance is being considered as the most difficult business of non - life insurance. Cargo insurance has been a kind of voluntary insurance for a long time without any subsidies by the government. However cargo insurance still plays an important role in case of traffic accidents during the transportation of goods by road.

**Insurance activities for road works:**

For construction of roads, safety issues are focused right from the design stage. Cost for insurance works is approved and included in project cost. Its insurance rate is 5% of project cost.

It is very necessary to purchase the insurance for transport works in general and for road works in particular in Vietnam climate condition. The changes of natural disasters such as...
flooding, landslides in mountainous areas where road works are under construction have been causing damages as up to several hundred billions Vietnamese dong. In this case, if the works are not insured, investors may lose the ability to invest for reconstruction and to overcome the consequences of natural disasters.

There is currently no statistics on the insurance business of this type of insurance for damage to road works or facilities as a result of a traffic accident.

The scope and scale of Vietnam's insurance market is still small, not commensurate with ability and potential development of socio - economy. Total revenue of the insurance industry accounts for 2% of GDP, while the contribution of the insurance industry in the countries in the ASEAN region is 2.5% - 7%, and the world average of 8%.

Some insurance areas are very potential but have not yet been untapped in the right way, for example: insurance of passengers reaches 39%, material insurance for body of vehicle is 40% of motor vehicles and 15% of motorcycles; compulsory insurance for civil liability motor vehicle owners is only 33%. The traffic characteristic of Vietnam is that the number of motorcycle vehicles is 19 times more than the number of cars; 70% of cases and 60% fatalities in road traffic accidents are involving motorcycles while motorcycle itself is a low safety index in transportation type and the drivers are very seriously injured when accidents occurred. Therefore, the role of motorized vehicle insurance should be promoted more.

A share of revenue of motor vehicle insurance is contributed by Insurance companies to build a fund for propaganda and esurience of traffic safety, develop a propaganda programs on Vietnam television, do oversea study tours for practical experience in the countries in the region and the world, support investment for projects being implemented in order to reduce black spots on traffic accidents on the national highway system.

Insurance market has contributed actively to stabilize production and life of people, raise capital for the industrialization and modernization of the country. This is an industry with many special characteristics and important significant for the economy. Therefore the role of government in managing and developing the insurance business is very enormous. In coming period, the Government should implement well the management, create a favorable legal environment, and also offer preferential policies for the insurance industry to develop stably and in the right direction.

References
DISPATCH COORDINATION BETWEEN HIGH - SPEED AND CONVENTIONAL RAIL SYSTEMS

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Summary: Cross - line trains, as a link between high - speed and conventional rail networks, will increase the complexity of transport organization and lead to significant challenges in dispatch coordination between the two systems. Based on the characteristics of high-speed transport organization, this paper deals with the necessity of dispatch coordination between high - speed and conventional lines from the following two perspectives: the operation of cross - line trains and work coordination in connection stations. An adjustment model for the operation of high - speed trains, taking cross - line trains into account, is established. Finally, the dispatch system is described in terms of construction and process. Methods for organizing dispatch are proposed, and the processes of coordination adjustment under normal and unexpected situations are analyzed. The discussion in this paper may serve as a theoretical basis for the development of high - speed rail dispatch systems.

Key words: high - speed rail; conventional rail; dispatch; coordination

I. INTRODUCTION

Since high-speed (HS) and conventional rails are two independent, but inter - related systems, the combination of high - speed trains and cross-line trains is considered a proper transport mode for a completely accessible railway network. With this mode, one may not only expand the accessibility of high - speed passenger transport, but may also make full use of the capacity of high - speed and conventional rail networks. However, cross - line trains, as the link between high - speed and conventional rail networks, will increase the complexity of transport organization, and present a significant challenge to dispatch coordination between the two systems. Work organization in connection stations is directly related to the degree of connection between high - speed and conventional rail networks, and is one of the main problems of transport dispatch. Theories of dispatch coordination between high - speed and conventional rail networks is of practical significance for optimizing transport organization, improving dispatch efficiency, and ensuring traffic safety. In 1973, Szpiegel [1] first proposed an optimal train schedule, using linear programming and the branch-and-bound method for train scheduling on a
single line. In 1983, Araya [2] put forward an optimal rescheduling for online train control in perturbation. Then Jovanovic [3] proposed the introduction of a heuristic technology into integer programming for train dispatch. This method significantly reduced the number of search nodes, and thus greatly improved the calculation speed. In China, studies of the optimization of railway dispatch have focused on establishing real-time models for the adjustment of train operation and for designing the related algorithms, with the optimization objectives as follows: highest rate of punctuality [4], least delay time [5], higher average travel speed [6], and improvement of the satisfaction of the passengers during the train operation [7], etc. Nie [8] investigated train operation adjustment in high-speed lines, and proposed the general principles for high-speed rail dispatch. Xie [9] studied key issues of train control in high-speed lines, such as the train operation adjustment model, the optimal adjustment of electric multiple units (EMUs), and optimization research for utilization of arrival and departure tracks. Xie established the framework of high-speed rail dispatch system.

However, conventional studies have focused on high-speed and conventional rail systems independently, without full consideration of the dispatch coordination between the two systems. In this paper, the coordination demands of high-speed and conventional rails, the optimization model of dispatch coordination, and the establishment of organization are studied.

II. CHARACTERISTICS OF HIGH-SPEED RAIL TRANSPORT ORGANIZATION

Because of its superiority in technology and economy, the high-speed rail system has some special features regarding transport organization that are different from the conventional rail system:

(1) In China, both the high-speed trains and the cross-line trains, trains with different speeds, run on the same high-speed line. Cross-line transport takes into account the characteristics of passenger flow, which can effectively ease the limited capacity of the conventional rail system and bring the capacity of high-speed rail system into full play, despite the unsaturated flow in the beginning.

(2) For adjusting train operation on the conventional line, freight trains usually need to wait or avoid passenger trains unconditionally. In high-speed rail systems, however, in order to ensure the travel speed and punctuality rate of each train, the numbers of overtaking and refuges should be equal for trains at the same level, and a passenger train should not wait in a train station for a long time.

(3) In order to ensure safety during the operation of high-speed rail systems, a long-time comprehensive maintenance window (more than 2 hours for ballastless track, 4 hours for ballasted track) needs to remain open [10], and the affected zone around the window has a significant impact on the line capacity as well. Generally, high-speed passenger trains are not allowed to wait for long during the maintenance window period. The operation of cross-line and evening-morning passenger trains is one of the difficult problems in the organization of high-speed rail networks.
III. COORDINATION DEMANDS OF HIGH - SPEED AND CONVENTIONAL RAILS

According to the features of rail networks, at the beginning of construction and operation of high - speed lines, the mode of ‘division - cooperation’ between high - speed and conventional lines is key to better transport capacity within the rail network.

3.1. Organization of cross - line trains

The means of passenger transfer and the cross-line transport will have a certain impact on the coordination between high-speed and conventional lines. In order to coordinate the operation between them, one should consider two main aspects:

(1) In transfer stations, high-speed and conventional trains should have a good connection in their arrival and departure time, which creates a high demand for punctuality. Since trains in conventional lines have many different levels, resulting in large speed deviation and complex transport organization, punctuality with conventional line dispatch is a challenge.

(2) When using cross-line transport, the needs of high - speed rail networks for high speed, high density, and high security demand a much more rigid timetable. Cross-line trains should avoid the comprehensive maintenance window time for high - speed lines, which limits, to some extent, the feasible range of conventional lines. Meanwhile, the optimal arrival and departure times of passenger trains makes it more difficult to operate cross - line trains in conventional lines. If trains are delayed in conventional lines, it will be very difficult to cross into high - speed lines on time. If this cannot be handled properly, the delay on conventional lines will spread into high - speed lines, causing disorder in the operation of high - speed rail networks. Therefore, the punctuality of cross - line trains in conventional lines should be ensured, and it is necessary to prepare some reserved train paths during the process of railway timetable drawing.

3.2. Coordination in the connection stations of high-speed and conventional lines

According to experiences in the operation of high - speed lines, as long as high - speed lines are connected with other lines, the transport organization at the connection point should be addressed. For example, in France, because high - speed trains can be operated off high - speed lines, the main point of its transport organization lies in the coordination at the connection point of the extended and high - speed lines, but not in the high - speed lines themselves. In Germany, as high - speed hub stations have many leading-in directions, utilization of arrival and departure tracks and connection of trains turn out to be the focus of its transport organization. Therefore, in joining high - speed and conventional rail networks, the connection stations are one of the
key points for transport coordination. Transport organization and coordination in connection stations is the major work of dispatch coordination between high-speed and conventional lines, including the following three aspects:

(1) Technique coordination

The interconnection between high-speed and conventional trains can be realized once the standardization of technical operation is fixed, which will provide good conditions for cross-line trains. The purpose of coordination is to ensure the optimal operation and best passing mode of passenger trains.

(2) Organization coordination

High-speed and conventional lines should follow their own functions and ensure the smooth operation in daily transport production. Thus, one must establish the division methods of controlling authority for dealing with equipments of high-speed and conventional stations. This kind of problem exists mainly in hub stations of passenger transport. Based on previous studies, high-speed and conventional lines should both adopt two dispatch systems, independent, but interrelated. Since high-speed lines have been integrated into stations of conventional lines, there have been problems of jurisdiction division for the two systems. According to the operation of high-speed trains, two premises should be guaranteed on the study of jurisdiction division. First, there must be certain connecting high-speed running paths on the high-speed lines that should not be disturbed by any other operation or shunting work. Second, the dispatch system of high-speed lines, especially the command authority for train-running, should be consecutive. Trains running on high-speed lines should always be under the control of the high-speed dispatch system.

(3) Information coordination

In order to realize a favorable interconnection between high-speed and conventional rail systems, interoperability of information must be ensured so that resource sharing can be available and operation efficiency can be improved. Information coordination requires the consistency of information between the two rail systems. In the subsystems of connection stations, standards for input and output of information should be uniform. The method of providing information should be uniform, and one subsystem should provide information rapidly and timely, so that other corresponding subsystems can respond quickly.

IV. OPTIMIZATION MODEL OF DISPATCH COORDINATION

Compared with conventional lines, high-speed lines are only open for passenger trains. There are not only high-speed trains of class A at speeds of more than 300 km/h and class B1 at speeds of 250 km/h, but also cross-line trains of class B2. Affected by trains in conventional lines, cross-line trains may deviate from the planned operation line while entering high-speed stations, resulting in interference with trains in high-speed lines. High-speed train operation adjustment means to create a comprehensive organization of self-line and cross-line trains,
making the latter operate safely and efficiently under the premise of guaranteeing the normal operation of the former.

As up - line and down-line trains do not disturb each other on high - speed lines, we take down - line trains for illustration in this paper.

Variable description: $T_s$ is the starting time of adjustment phase; $T_e$ is the end time of adjustment phase; $T_0^H$ is the ensemble of trains of class A during the adjustment phase, and $N_D^H = \left\{ T_0^H \right\}$, $i = 1,2,L,N_D^H$; $T_0^M$ is the ensemble of trains of class B during the adjustment phase, and $N_D^M = \left\{ T_0^M \right\}$, $i = 1,2,L,N_D^M$; $T_0^O$ is the ensemble of trains of class B during adjustment phase, and $N_D^O = \left\{ T_0^O \right\}$, $i = 1,2,L,N_D^O$; $T_D$ is the ensemble of trains during the adjustment phase: $T_D = T_0^H \cup T_0^M \cup T_0^O$, $N_D = \left\{ T_0 \right\}$, $i = 1,2,L,N_D$; $\omega_i$ is the priority value of train $i$ during the adjustment phase, $\forall i \in N_D$, satisfying $\sum_{i} w_i = 1$; $\lambda(i)$ is the type of train $i$, where $\lambda(i) = 1$ denotes the train of class A, $\lambda(i) = 2$ the train of class B, and $\lambda(i) = 3$ the train of class B, $\forall i \in N$; $S_D$ is the ensemble of stations, where stations are numbered one by one along the down - line direction, and $K_D = S_D$, $k = 1,2,\cdots,K_D$; $B_D$ is the ensemble of intervals, where intervals are numbered one by one along the down - line direction, and $K_D - 1 = B_D$, $k = 1,2,\cdots,K_D - 1$; $G_D$ is the set of train paths according to the train operation plan; $G_D = \left\{ K_1^i, K_2^i,\cdots,K_{k(i)}^i \right\}$, $\forall i \in T_D$, $G_D \subseteq S_D$, where $K_1^i$ and $K_{k(i)}^i$ are, respectively, station $t$ and the last station passed by train $i$ according to the train operation plan; $J_D$ is the operation routine of train $i$ during the adjustment phase: $J_D = \left\{ k_1^i, k_2^i,\cdots, k_{k(i)}^i \right\}$, $\forall i \in T_D$, $J_D \subseteq S_D$, where $k_1^i$ and $k_{k(i)}^i$ are, respectively, station $t$ and the last station passed by train $i$ during the adjustment phase; $t_0^{i,k}$ is the minimum net - operating time of trains of class $\lambda(i)$ during interval $k$, $\forall i \in T_D$, $\forall k \in B_D$; $s_0^{i,k}$ is the minimum stop time of train $i$ in station $k$, and if train $i$ starts or arrives in station $k$, its minimum stop time equals zero, $\forall i \in T_D$, $\forall k \in G_D^i$; $y_0^{i,k}$ is the scheduled arrival time of train $i$ at station $k$, and if train $i$ starts from station $k$, its scheduled arrival time equals scheduled departure time, $\forall i \in T_D$, $\forall k \in G_D^i$; $y_0^{i,k}$ is the scheduled departure time of train $i$ at station $k$, and if train $i$ starts from station $k$, its scheduled departure time equals scheduled arrival time, $\forall i \in T_D$, $\forall k \in G_D^i$; $\phi_0^{i,k}$ is a 0 - 1 variable, if down - line train $i$ passes by down - line station $k$, it is valued 0, and otherwise 1, $\forall i \in T_D$, $\forall k \in J_D^i$; $\phi_0^{i,k}$ is the additional starting time of trains of class $\lambda(i)$ at station $k$, $\forall i \in T_D$, $\forall k \in S_D$; $P_0^{i,k+1}$ is the additional stopping time of trains of class $\lambda(i)$ at station $k+1$, $\forall i \in T_D$, $\forall k \in S_D$; $c_0^{i,k}$ is a 0 - 1 variable, if train $i$ has passenger service at station $k$, it is valued 1, and otherwise 0, $\forall i \in T_D$, $\forall k \in J_D^i$; $X_0^{i,k}$ is the actual arrival time of train $i$ at station $k$, $\forall i \in T_D$, $\forall k \in J_D^i$; $Y_0^{i,k}$ is the actual departure time of train $i$ at station $k$. \[ \text{INTERNATIONAL COOPERATION ISSUSE OF TRANSPORTATION - Especial Issue - No.03} \]
k, \( \forall i \in T_D, \forall k \in J_D \); \( x_{D_{ik}}^{ik} \) is the planned arrival time of train \( i \) at station \( k \), \( \forall i \in T_D, \forall k \in J_D \); \( y_{D_{ik}}^{ik} \) is the planned departure time of train \( i \) at station \( k \), \( \forall i \in T_D, \forall k \in J_D \). \( H_{D_{ik}}^{ik(k)} \) is the allowed earliest arrival time of the cross-line trains at the terminal station \( K_{D_{ik}}^{(k)} \). According to the train levels, punctuality of arrival and departure, and requirements for travel time, we choose the minimum weighted deviation of trains’ arrival and departure time as the optimization objective, by introducing \( \Delta_{D_{ik}}^{ik} \) as the deviation of the actual arrival time and scheduled arrival time of train \( i \) at station \( k \):

\[
\Delta_{D_{ik}}^{ik} = |x_{D_{ik}}^{ik} - x_{D_{ik}}^{ik}|
\]

as the deviation of the actual departure time and scheduled departure time for train \( i \) at station \( k \):

\[
\Delta_{D_{ik}}^{ik} = |y_{D_{ik}}^{ik} - y_{D_{ik}}^{ik}|
\]

as follows:

\[
\min z = \sum_{i \in T_D} \omega_i \sum_{k \in J_D} (\Delta_{D_{ik}}^{ik} + \Delta_{D_{ik}}^{ik}) \tag{1}
\]

S.t

\[
X_{D_{ik}}^{ik} \geq Y_{D_{ik}}^{ik} + t_{D_{ik}}^{ik} + \theta_{D_{ik}}^{ik} + \phi_{D_{ik}}^{ik} + \beta_{D_{ik}}^{ik} + \gamma_{D_{ik}}^{ik} + \delta_{D_{ik}}^{ik} + \epsilon_{D_{ik}}^{ik} \tag{2}
\]

\[
c_{D_{ik}}^{ik} = 0 \text{ and } c_{D_{ik}}^{ik} = 0, \forall i \in T_D, \forall k \in V_D
\]

\[
X_{D_{ik}}^{ik} \geq X_{D_{ik}}^{ik} + \phi_{D_{ik}}^{ik} + \gamma_{D_{ik}}^{ik} + \delta_{D_{ik}}^{ik} + \epsilon_{D_{ik}}^{ik} \tag{3}
\]

\[
c_{D_{ik}}^{ik} = 0 \text{ and } c_{D_{ik}}^{ik} = 1, \forall i \in T_D, \forall k \in V_D
\]

\[
X_{D_{ik}}^{ik} \geq Y_{D_{ik}}^{ik} + (t_{D_{ik}}^{ik} + \alpha_{D_{ik}}^{ik}) + \phi_{D_{ik}}^{ik} + \beta_{D_{ik}}^{ik} + \gamma_{D_{ik}}^{ik} + \delta_{D_{ik}}^{ik} \tag{4}
\]

\[
c_{D_{ik}}^{ik} = 1 \text{ and } c_{D_{ik}}^{ik} = 0, \forall i \in T_D, \forall k \in V_D
\]

\[
X_{D_{ik}}^{ik} \geq Y_{D_{ik}}^{ik} + (t_{D_{ik}}^{ik} + \alpha_{D_{ik}}^{ik}) + \beta_{D_{ik}}^{ik} \tag{5}
\]

\[
c_{D_{ik}}^{ik} = 1 \text{ and } c_{D_{ik}}^{ik} = 1, \forall i \in T_D, \forall k \in V_D
\]

\[
H_{D_{ik}}^{ik(k)} \leq X_{D_{ik}}^{ik(k)} \leq Q_{D_{ik}}^{ik(k)}, \forall i \in T_D \text{ and } K_{D_{ik}}^{(k)} \in J_D \tag{6}
\]

Among the above expressions, (1) is the objective function, which will minimize the deviation of trains’ arrival and departure time; (2) represents constraints of running time in sections; (3) represents constraints of stop time at stations; (4) represents constraints of headway; (5) represents constraints of the earliest feasible departure time; and, (6) represents constraints of cross-line time of cross-line trains.

Among the exact algorithms at present, branch - and - bound and cutting - plane are considered two effective algorithms for resolving the problem. Because it is difficult to find the cutting plane, it is proposed this model be resolved by the branch - and - bound algorithm.

V. ESTABLISHMENT OF ORGANIZATION

In order to achieve the dispatch coordination between high - speed and conventional lines, functional coordination and information sharing should be taken into consideration. As conventional lines in China are freight - oriented, the existing dispatch system is made up of
four main types of work: train operation, locomotives, freight, and plan. However, the dispatch on the high-speed lines mainly targets passenger trains. Therefore, it focuses on passenger-oriented service. Based on this, conventional rails should independently control the freight dispatch systems (freight - DS), locomotives - DS, and planning - DS, while high-speed rails should not only establish its own dispatch system to satisfy its business requirements, but also consider as well the sharing of certain dispatch systems with conventional rails, such as train operation and management - DS, EMU - DS, and passenger service - DS. The power supply - DS and comprehensive maintenance system, under the commanded of high-speed dispatch centre, correspond respectively to the power supply dispatch room and engineering affairs dispatch room of conventional lines. Therefore, the corresponding dispatch platforms of high-speed and conventional lines include train operation and management - DS, EMU-DS, passenger service - DS, power supply - DS, and comprehensive maintenance system. In the high-speed rail dispatch system, the dispatch centre controls directly all the dispatch platforms and just provides information for these platforms for the conventional rail dispatch centre. The organization establishment of high-speed and conventional rail dispatch systems is shown in fig 1.

In fig 1, the solid arrow indicates direct control, and the dotted arrow indicates information exchange. Similarly, in the conventional rail dispatch system, in addition to planning - DS, locomotive - DS, and freight - DS, the dispatch centre in conventional stations should also establish the train operation and management - DS, EMU - DS, passenger service - DS, power supply - DS, and comprehensive maintenance system. These five systems are also controlled by the dispatch centre in railway stations, and provide information for the high-speed rail dispatch centre.

VI. COORDINATION

The key point of dispatch coordination lies in the transport organization of cross-line
trains and the dispatch of work in connection stations. Dispatch coordination between high-speed and conventional lines is shown in fig 2. In fig 2, arrows that start from the related department of the dispatch centre to the corresponding department or task involved in dispatch indicate that this department is in charge of a certain dispatch task; bidirectional arrows represent information exchange during the execution of different tasks. The dispatch of train operation and management, EMU, and passenger service are key sections of cross-line train operation and work organization in connection stations. Through coordination between dispatch systems of high-speed and conventional lines, the information exchange of cross-line trains and the proper interconnection of command on all lines can be achieved. A solid arrow indicates the direct influence or control while the dotted arrow indicates information exchange.

In fig 2, letters from A to W are defined as follows:

**Fig 2. Dispatch coordination between high-speed and conventional lines**

A: Regional dispatch center of high-speed rail; B: Comprehensive maintenance dispatch; C: Power supply control; D: EMU dispatch; E: Passenger service dispatch; F: Train operation management dispatch; G: Train operation plan management; H: Cross-line EMU routing plan adjustment; I: Vehicle-allocation plan adjustment, crew plan adjustment, and maintenance plan adjustment; J: Reserve EMUs in the operation depot and EMU maintenance base; K: Cross-line EMU monitoring; L: Intercommunication of signal, line-state, and safety-control information; M: Train-operation plan, EMU routing plan, vehicle-allocation plan, crew plan, maintenance plan, comprehensive maintenance plan, power-supply plan, and station-operation plan; N: Train-operation adjustment in the high-speed rail zone; O: Technical operation in high-speed rail stations; P: Technical operation in conventional station (high-speed yard); Q: Train-operation adjustment in the conventional railway zone; R: Passenger organization in the high-speed train station, ticket business, and indication system; S: Cross-line passenger organization on the conventional railway line, cross-line ticket business, and cross-line indication system; T: Cross-line train-operation information; U: Passenger transport dispatch; V: Railway bureau dispatch center; W: Intercommunication of power-supply information.
Jurisdictional division of these two dispatch systems must obey the “single command” principle. One yard, track, access, or turnout group can only be controlled by one system, and monitored by another system. The dispatch staff in base stations can only be commanded by one related dispatch platform of one system.

In connection stations, the work related to the train - route of high - speed trains are operated by the dispatch system of high - speed rails, while others are operated by that of conventional rails. As to cross - line trains, the train-route of getting on and off high - speed lines is controlled by those in charge of receiving trains; that is, the train - route of getting on high - speed lines is controlled by the high - speed dispatch system, while that of getting off is controlled by the conventional lines dispatch system.

In unexpected situations, train operation needs to be adjusted and the process of dispatch coordination of high - speed and conventional lines is shown in fig 3.

![Fig 3. Coordination process in unexpected situations](image)

The passenger service dispatch of high - speed lines and the passenger transport dispatch of conventional lines are managed and fed back according to passenger flows, ticket affairs, and other information of cross - line trains (as shown by “1” in fig 3). Train operation, technical operation in stations in high - speed rail sections, and the work related to the high - speed station yard in conventional stations are organized and adjusted in the same time by the dispatch of passenger service, train operation and management, and EMU of high - speed lines. Trains on conventional lines are organized and adjusted in the same time by the dispatch of passenger transport and that of train operation and management (as shown by “2” and “3” in fig 3). Via sharing and transferring the operation information of cross - line trains, the dispatch coordination between high - speed and conventional rails can be realized (as shown by “4” in fig 3).

VII. CONCLUSIONS

Operation of cross-line trains and work coordination in connection stations are key tasks of
dispatch coordination between high-speed and conventional rails. By analyzing the necessity of dispatch coordination between high-speed and conventional lines, this paper builds an adjustment model for high-speed train operation with the operation of cross-line trains taken into consideration. The dispatch organization of high-speed and conventional rails is set up according to the operation of cross-line trains and the work coordination in connection stations. The dispatch coordination between high-speed and conventional lines is described, and the process of coordination adjustment in unexpected situations is analyzed.

This research provides a certain theoretical basis for building a high-speed rail dispatching system. The detailed settings and function segmentations of dispatch organization, and system design and development remains as the main work of optimization of high-speed rail dispatch.

References


STUDYING SEMI ACTIVE SUSPENSION SYSTEM USING BALANCE CONTROL STRATEGIES TO IMPROVE RIDE COMFORT

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Summary: The smooth motion is one important factor of the automobile quality. There are many measures to improve the smooth movement of the car, in which researchers in Vietnam and in the world interested in semi-active control of suspension system. This paper presents the content Balance control method applied to semi-active suspension system with two control regular are Balance Control "on-off" and continuous Balance control.

I. INTRODUCTION

During road traffic, the road surface is the main source of excitation causing vehicle vibration that influences driver and passengers. The study of suspension system is one of the most effective ways to improve ride comfort. Because of economical energy consumption and good ride quality, the semi-active suspension system is a keen interest for many researchers.

The semi-active suspension systems have been studied since 1970 [1]. Nowadays they are quite popular in modern vehicles. There are a variety of control algorithms for semi-active suspension systems, one of which is balance control strategy.

Fig 1. Semi-active suspension system in Cadillac SRX
II. CONTENT

In this part we consider a one-mass model (fig 2a) with excitation $x_0(t)$, spring rate $k$ and damping factor $c$ which has 1 degree of freedom.

Fig 2. Passive model and Semi-active model using balance control strategy
a) Passive suspension system; b) Semi-active suspension system

Vibration equation is given in the following form:

$$m\ddot{x} + F_k + F_d = 0$$

(1)

Where: $F_k$ and $F_d$ are spring force and damping force corresponding.

$$F_k = k(x - x_0)$$

(2)

$$F_d = c(x - x_0)$$

(3)

The relations between $m\ddot{x}$, $F_k$ and $F_d$ in case of harmonized excitation are shown in figure 3.

Fig 3. Relation between force acting on the sprung mass “m” in case of harmonized excitation

_____ : Damping force ($F_d$); ____ : Spring force ($F_k$) and ______: Inertial force ($m\ddot{x}$)
The amplitude of acceleration of sprung mass “m” in harmonized excitation depends on damping force and spring force due to the following equations [2]:

\[
\begin{align*}
|\ddot{x}| &= \left| \frac{F_k + F_d}{m} \right|, & t_0 < t < t_0 + \frac{\tau}{4} \\
|\ddot{x}| &= \left| \frac{F_k - F_d}{m} \right|, & t_0 + \frac{\tau}{4} < t < t_0 + \frac{3\tau}{4} \\
\end{align*}
\]

Where: \( t_0 \) is the time during which the spring force is “zero”; \( \tau \) is the frequency of vibration.

During vibration, one would like to have small \( \ddot{x} \), however in accordance with the equations 4, 5 and fig 3, the rise of damping force causes increment of amplitude of the acceleration in one part of the cycle of vibration. After that the amplitude of \( \ddot{x} \) will be reduced if \( F_k \) and \( F_d \) have the same magnitude. When increasing the exciting frequency, it is dominated by damping force \( F_d \). Rakheja, S. and S. Sankar [2] proposed balance control strategy using active damper (fig 2.b) which can be: hydraulic damper with throttle, friction damper, MR damper, ER damper, electromagnetic damper ….

This strategy maintains that the damping force increases the acceleration of sprung mass when the damping force and spring force have the same sign. The 2 - state active damper (On - Off), at the “off” state when damping force and spring force act on the same direction \((x - x_0)(x - x_0) > 0\), and vice versa at “on” state when \((x - x_0)(x - x_0) \leq 0\). Therefore the damping force is against the spring force and the strategy is called Balance Control [2].

### 2.1. Continuous Balance Control

In order to maintain the equality of damping for and spring force at the “on” state:

\[
F_{SA} = \begin{cases} 
-k(x - x_0) & (x - x_0)(x - x_0) \leq 0 \\
0 & (x - x_0)(x - x_0) > 0 
\end{cases}
\]

Damping coefficient for active damper (fig 4):

\[
C_{SA} = \begin{cases} 
-k(x - x_0) & (x - x_0)(x - x_0) \leq 0 \\
\frac{x - x_0}{x - x_0} & (x - x_0)(x - x_0) > 0 
\end{cases}
\]
When the relative velocity \((x - x_0)\) is very small, the damping coefficient is closed to infinity, which cannot happen for the real damper. Therefore the damping coefficient for active damper \(C_{SA}\) must continuously vary within the interval \((C_{\text{max}}, C_{\text{min}})\) according to the manufacturer’s desire. The value of \(C_{SA}\) can be determined as the following:

\[
C_{SA} = \begin{cases} 
\max \left[ C_{\text{min}}, \min \left[ \frac{-k(x - x_0)}{x - x_0}, C_{\text{max}} \right] \right] & (x - x_0)(x - x_0) \leq 0 \\
C_{\text{min}} & (x - x_0)(x - x_0) > 0 
\end{cases}
\]  

(8)

In this case, the value of damping force is plotted as seen in fig 5.
2.2. “On-off” Balance Control

The “on-off” balance control is studied to simplify the working of the damper [2]. In the two states, the active damper is controlled at maximum state or minimum state or high and low state correspondingly. In this case, the damping force is determined as:

\[
F_{SA} = \begin{cases} 
    C_{on} (x - x_0) & (x - x_0)(\dot{x} - \dot{x}_0) \leq 0 \\
    0 & (x - x_0)(\dot{x} - \dot{x}_0) > 0 
\end{cases} \tag{9}
\]

Where: \(C_{on}\) is the damping coefficient of damper “on-off” at the “on” state.

The relation between damping force in “on-off” balance control and \((x - x_0)\) and \((\dot{x} - \dot{x}_0)\) is shown in fig 6.

\[\text{Fig 6. Damping force } F_{SA} \text{ with respect to } (x - x_0) \text{ and } (\dot{x} - \dot{x}_0) \text{ in case of “on-off” balance control}\]

2.3. Simulation and evaluate semi-active suspensions system using Balance Control

a. Exciting sources

Study the exciting source from road - surface irregularities as impulse unit (stepped and sine unit) and random case (Highway Hanoi - Langson [7]) (fig 7).

\[\text{Fig 7. Road profiles}\]

b. Evaluation criteria

Ride comfort level is evaluated by Root Mean Square of the vertical acceleration and pitch.
acceleration of vehicle body according to random excitation and amplitude peaks with impulse-unit excitation. Moreover, the Root Mean Square of dynamic wheel load is used to assess the safety of handling characteristic.

\[
\text{RMS}(\ddot{Z}_j) = \sqrt{\frac{\sum_{j=0}^{T} (\ddot{Z}_{j})^2}{T}} \quad (10)
\]

\[
\text{RMS}(\ddot{\varphi}) = \sqrt{\frac{\sum_{j=0}^{T} (\ddot{\varphi}_j)^2}{T}} \quad (11)
\]

c. Vehicle longitudinal vibration model

The vehicle longitudinal vibration model is shown in figure 8. The model has 4 degrees of freedom: vertical displacement of center of gravity \( Z_3 \), pitch angle \( \varphi \) and vertical displacements of unsprung masses \( Z_1, Z_2 \).

The vibration equation is given as:

\[
\begin{align*}
J, \ddot{\varphi} &= - \left[ k_{12} (Z_1 - Z_4) + c_1 (\dot{Z}_1 - \dot{Z}_4) \right] l_1 + \left[ k_{22} (Z_2 - Z_2) + c_2 (\dot{Z}_2 - \dot{Z}_2) \right] l_2,
\end{align*}
\]

\[
\begin{align*}
m_3, \ddot{Z}_3 &= k_{12} (Z_1 - Z_4) + k_{22} (Z_2 - Z_2) + c_1 (\dot{Z}_1 - \dot{Z}_4) + c_2 (\dot{Z}_2 - \dot{Z}_2) - f_{d1} + f_{d2},
\end{align*}
\]

\[
\begin{align*}
m_1, \ddot{Z}_1 &= -k_{11} (Z_1 - q_1) + k_{12} (Z_1 - Z_2) + c_1 (\dot{Z}_1 - \dot{Z}_1) + f_{d1},
\end{align*}
\]

\[
\begin{align*}
m_2, \ddot{Z}_2 &= -k_{21} (Z_2 - q_2) + k_{22} (Z_2 - Z_2) + c_2 (\dot{Z}_2 - \dot{Z}_2) + f_{d2}.
\end{align*}
\]

\[\text{Fig 8. Vehicle longitudinal vibration model}\]
Where:
\[
\begin{align*}
Z_1 &= Z_3 - \varphi l_f \\
Z_2 &= Z_3 + \varphi l_r
\end{align*}
\] (13)

The equation (12) written in State-Space yields:
\[
\begin{align*}
x &= A.x + B.u \\
Z &= C.x + D.u
\end{align*}
\] (14)

Where: \(x = [\varphi Z_3 Z_1 Z_2 \dot{\varphi} Z_3 \dot{Z}_1 Z_2]^T\): vector of state functions;

\(Z = [\varphi Z_3 F_1 F_2]^T\): vector of output parameters;

\(F_1 = k_{i1} (Z_1 - q_1)\): dynamic wheel load at the front axle;

\(F_2 = k_{21} (Z_2 - q_2)\): dynamic wheel load at the rear axle.

\(u = [f_{d1} f_{d2} q_1 q_2]^T\): vector of excitation.

\[A = \begin{bmatrix}
O_{(4x4)} & I_{(4x4)} \\
-M^{-1}Q & -M^{-1}P
\end{bmatrix};
B = \begin{bmatrix}
O_{(4x4)} \\
-M^{-1}Q & -M^{-1}P
\end{bmatrix};
C = \begin{bmatrix}
-G_1.M^{-1}.Q & -G_1.M^{-1}.P
\end{bmatrix};
D = \begin{bmatrix}
G_1.M^{-1}.Q & G_1.M^{-1}.P
\end{bmatrix};
\]

\[G_1 = \begin{bmatrix}
1 & 0 & 0 & 0 \\
0 & 1 & 0 & 0
\end{bmatrix};
G_{21} = \begin{bmatrix}
0 & 0 & k_{i1} & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & k_{21} & 0 & 0 & 0 & 0
\end{bmatrix};
G_{22} = \begin{bmatrix}
0 & 0 & -k_{i1} & 0 \\
0 & 0 & 0 & -k_{21}
\end{bmatrix};
\]

\[J = \begin{bmatrix}
0 & 0 & 0 & 0 \\
0 & m_1 & 0 & 0 \\
0 & 0 & 0 & 0 \\
0 & 0 & 0 & m_2
\end{bmatrix};
M = \begin{bmatrix}
(c_{11} & c_{21} + c_{22} & 0 & 0 \\
0 & (c_{11} & c_{21} + c_{22} & 0 \\
0 & 0 & c_{11} & 0 \\
0 & 0 & 0 & c_{22}
\end{bmatrix};
P = \begin{bmatrix}
(c_{11} & c_{21} + c_{22} & 0 & 0 \\
0 & (c_{11} & c_{21} + c_{22} & 0 \\
0 & 0 & c_{11} & 0 \\
0 & 0 & 0 & c_{22}
\end{bmatrix};
\]

\[Q = \begin{bmatrix}
(k_{12} & k_{22} & -k_{22} & 0 \\
-k_{12} & k_{22} & c_{11} & 0 \\
-k_{12} & k_{22} & 0 & 0 \\
-k_{12} & k_{22} & c_{11} & 0
\end{bmatrix};
\]

\[H = \begin{bmatrix}
-l_f & l_r & 0 & 0 \\
1 & -1 & 0 & 0 \\
0 & 1 & 0 & k_{21}
\end{bmatrix}
\]
d. Results and evaluations of the balance control strategies

Table 1. Specification of the studied vehicle

<table>
<thead>
<tr>
<th>No.</th>
<th>Parameter</th>
<th>Denotation</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Unsprung mass at the front axle/rear axle</td>
<td>( m_1/ m_2 )</td>
<td>36/36</td>
<td>kg</td>
</tr>
<tr>
<td>2</td>
<td>Sprung mass</td>
<td>( m_3 )</td>
<td>540</td>
<td>kg</td>
</tr>
<tr>
<td>3</td>
<td>Moment of inertia</td>
<td>( J )</td>
<td>14.10(^4)</td>
<td>kg( \cdot )m(^2)</td>
</tr>
<tr>
<td>4</td>
<td>Stiffness coefficient of front tire /rear tire</td>
<td>( k_{11}/ k_{21} )</td>
<td>16.10(^3)/16.10(^4)</td>
<td>N/m</td>
</tr>
<tr>
<td>5</td>
<td>Spring rate at the front axle /rear axle</td>
<td>( k_{12}/ k_{22} )</td>
<td>16.10(^3)/16.10(^4)</td>
<td>N/m</td>
</tr>
<tr>
<td>6</td>
<td>Damping coefficient at the front axle /rear axle</td>
<td>( c_1/ c_2 )</td>
<td>1400/1400</td>
<td>N.s/m</td>
</tr>
<tr>
<td>7</td>
<td>CG distance between the front and rear axles</td>
<td>( l_f/ l_r )</td>
<td>1.6/1.4</td>
<td>m</td>
</tr>
</tbody>
</table>

The velocity of the studied vehicle with impulse-unit excitation is 2 (m/s) and for the case of random excitation is 20 (m/s). The comparisons between semi-active suspension system using balance control strategies and passive suspension system are shown in fig 9.

![Fig 9. Comparisons between semi-active suspension system using balance control strategies and passive suspension system: (a- Impulse-unit excitation; b- Random excitation)](image)

It is indicated in the results that ride comfort criteria values in case of semi-active suspension system using balance control strategies are smaller than the ones of passive suspension system (100%). For continuous balance control strategy RMS(\(\bar{Z}_3\)), RMS(\(\bar{\phi}\)) are just 70%, and 75% in comparison with “on-off” balance control.

III. CONCLUSION

The semi-active suspension system have been worldwide studied to improve ride comfort.
The present paper introduced continuous balance control and “on-off” balance control. The simulation results in the case of 4-degree of freedom model showed the efficiencies of the control balance strategies and would be a typical sample for the applications in other vibration models.

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APPLICATION OF JERK IN ...

(Following page 66)

Usually, in order to keep stability, a passenger either preventively shifts his mass centre against inertial force \( F_c \) creating overturning moment \( M_c = F_c \cdot h \) (fig 1), or bends a knee in order to lower mass centre and reduce arm \( h \). Gravity force \( G \) creates restoring moment \( M_G = G \cdot x \). Comparing figures 1 and 2, it gets clear why standing sideways towards direction of jerk action is preferable; in this case arm \( x \) is significantly greater.

As it is known, if you pull a paper sheet with a standing lipstick sharply, the latter falls; but if the sheet approaches maximum acceleration gradually, the lipstick would remain standing. Analogically for curvilinear motion, comparing two cases with equal lateral acceleration amplitudes \( q_1 = q_2 \), vehicle stability is better in that where jerk is lower. While \( j \) reaches \( j_{\text{int}} \), it is necessary to transfer braking forces. Similarly, a passenger has to transfer weight from one leg to another.

Usually, speaking of critical speed \( v_{cr} \), corner radius is considered constant \( R = \text{const} \). But regarding transitional curvilinear motion, both speed \( v \) and radius \( R \) are subject to change. Therefore it is expedient to use concept of critical lateral acceleration:

\[
q_{cr} = \frac{v_{cr}^2}{R_{cr}}
\]
Consequently, in accordance with formulae of critical speed [6], expressions for critical lateral acceleration can be deduced. Critical lateral acceleration with respect to skidding:

\[ q_{cr,j} = g\phi \]

Critical lateral acceleration with respect to overturning:

\[ q_{cr,h} = g\frac{B}{2h} \]

Thus, time of reaching emergency situation for given current jerk \( j_0 \):

\[ \Delta t_{cr} = \frac{q_{cr} - q_0}{j_0} \]

IV. CONCLUSION

In other words, criticality of road traffic situation is directly proportional to lateral jerk excited in the beginning of driver’s control action (steering or braking). Analogically for a passenger, if his reaction time \( T_R \) is greater than \( \Delta t_{cr} \), a stability loss follows.

Normative meanings of lateral jerk for different emergency situations are subject to be determined empirically [7]. So far, it is way too early to speak about absolute meanings of critical lateral jerk. In the future will arise a need to substantiate theoretically physical interrelation between lateral jerk and stability loss, to determine meanings of critical jerk as an independent quantity that characterizes initiation and development of the stability loss process.

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COMPARATIVE ANALYSIS ON DYNAMIC BEHAVIOR OF TWO HMA RAILWAY SUBSTRUCTURES

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Summary: A numerical analysis using a finite element program was performed on three structures: hot mix asphalt (HMA) reinforced trackbed (RACS-1), HMA directly supported trackbed (RACS-2), and traditional Portland Cement Concrete (PCC) slab track (SlabTrack). Although the comprehensive dynamic responses of RACS-1 were similar with SlabTrack, HMA layer can positively affect the stress distributions. In particular, the horizontal stresses indicate that the resilience of RACS-1 was improved relative to SlabTrack. In addition, HMA reinforced substructure has the capacity to recover the residual vertical deformation. The effective depth for weakening dynamic loadings is mainly from 0 to 2 m, this being especially true at 0.5 m. The results from the analysis show that HMA is a suitable material for the railway substructure to enhance resilient performance, improve the stress distribution, weaken dynamic loading, and lower the vibration, especially at the effective depth of 2 m. The HMA constructed at the top of the stone subbase layer allows the vertical modulus a smooth transition. In terms of the comprehensive dynamic behaviors, RACS-1 is better than SlabTrack, while the results for RACS-2 are inconclusive and require further research.

Key words: High-speed railway; HMA; railway substructure; FEM analysis

1. INTRODUCTION

Around the world there is a trend for both high-speed and heavy-axle loadings trains, a
trend which requires stronger and smoother railway substructures [1]. For the traditional and regular railway trackbed, the pulverization of granular ballast and settlement of the substructure demands extensive maintenance to meet the safety requirements for high speeds [2]. Ballastless tracks have been widely used for high-speed railway construction by many countries like Japan [3] and Germany [4] to deal with those problems. These ballastless tracks often adopt Portland cement concrete (PCC) to distribute loads from rails to the subgrade. However, these PCC railway substructures also tend to have significant maintenance issues such as cracking, thus, making it difficult to maintain the track, as well as problems with noise and vibration [5-6].

Visco-elastic characteristics with sufficient strength and resilience [7] of hot mix asphalt (HMA) could make it better suited for the requirements of high-speed railway substructures. Japan [8], Germany [9], Italy [10] and the US [11-12] have conducted considerable research, including theoretical analysis, as well as laboratory and field tests on railways with HMA substructures. In China, HMA is currently used as a waterproof material to cover the top of both sides of the subgrade surface and along the middle of parallel railway lines. It is also called surface asphalt mixture as impermeable (SAMI) [13-14]. HMA is being considered as a structural layer, and the related research, supported by National Natural Science Foundation of China (NSFC), is just in its early stages.

This paper summarizes the basic calculations considered as part of the NSFC project for the research pertaining to model testing in the lab. Dynamic responses of three railway substructures were calculated using the finite element method (FEM) program ABAQUS [15]. The railway substructures were the Japanese ballastless track design, known as concrete slab track named SlabTrack, and two structures named RACS-1 and RACS-2. These last two structures had an HMA layer at different positions in the substructures. The horizontal stresses, vertical deformation, and acceleration were analyzed.

II. OBJECTIVES

Dynamic behaviors of HMA substructures are compared to the corresponding responses from the slab track.

The schematic cross-sections of the three structures are shown in fig 1. Fig 1(a), referred to as RACS-1, uses an HMA layer between a PCC base and the crushed stone subbase. Fig 1(b), RACS-2, uses an HMA layer to replace the PCC base. Fig 1(c) is the design of SlabTrack as the reference of this analysis. The full cross-section with parallel track was modeled to avoid the complicated processing of symmetric boundary conditions because of non-linear materials like HMA. Moreover, the adjacent train loadings affect each other for the real dynamic responses tested in field.
III. FINITE ELEMENT METHOD MODELS

3.1. Parameters of the layered railway substructures

Each concrete slab is 4,930 mm long, and the distance between two fasteners is 600 mm, excluding the gap at the joint (usually 70 mm). In total, 200 MPa was used as the dynamic modulus assumed to be the same as the resilient modulus due to the high stiffness for the mortar layer, which is for the purpose of adjusting and made from asphalt emulsion, cement, fine aggregate, water, and some special additives like aluminite powder.

The thickness of the HMA layer was 200 mm, and the widths of HMA in RACS-1 and...
RACS-2 were the same as the crushed stone subbase and PCC base, respectively. The HMA dynamic modulus was also considered to be 200 MPa using the vulnerable condition to represent its stiffness during the warmest months [16]. The SAMI layer was not considered as part of the load carrying structure. For the subgrade bed, a dynamic modulus of 150 MPa was used for the top of the layer and 120 MPa for the bottom of the layer. All of the element types were solid. The parameters for the calculation are listed in Table 1.

### 3.2. Assembly of numerical models

The bottom of each model was constrained in all six degrees of freedom (DOF). Symmetric boundary condition was applied for the cut section along the rail direction. There were no other constraints for the models. For simplicity, a continuous condition for interlayer contact was used. The mesh of models was generated by an 8-node inducing integrated element, C3D8R, because of its higher accuracy with lower computational cost. Fig 2(a) shows the schematic model after meshing.

### 3.3. Scheme of load and calculation

#### 3.3.1. Load selection

For simplicity, a uniaxial uniform pressure of 125 kN was used on each side of the rail track to simulate the load passing from rail and fastener to slab track; each loading area was 100 mm × 200 mm and 1 435 mm was used for the central spacing. The sphere of dynamic influence was in the area of nine sleepers in a row and each was placed at an interval of 0.6 m (d). Assume the train speed is 200 km/h (v), and the loading time difference dt to a single sleeper is 3 600d/v = 0.010 8 s. Although 0. 005 4 seconds was adopted as the time step of this numerical analysis, the loading-time curves shown in fig 2(b) after interpolating were used to improve the calculating accuracy.

<table>
<thead>
<tr>
<th>Structural layer</th>
<th>Geometry parameters (m)</th>
<th>Materials</th>
<th>Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Modulus (MPa)</td>
</tr>
<tr>
<td>Concrete slab</td>
<td>RACS-1 SlabTrack</td>
<td>2.4 w 0.19 h</td>
<td>Cement concrete</td>
</tr>
<tr>
<td>Mortar</td>
<td>RACS-1 RACS-2 SlabTrack</td>
<td>2.4 w 0.05 h</td>
<td>CA sand mortar</td>
</tr>
<tr>
<td>Concrete base</td>
<td>RACS-1 SlabTrack</td>
<td>3.0 w 0.3 h</td>
<td>Cement concrete</td>
</tr>
<tr>
<td>HMA</td>
<td>RACS-1 RACS-2</td>
<td>8.8 w 0.2 h</td>
<td>Asphalt mixture</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.0 w 0.3 h</td>
<td></td>
</tr>
<tr>
<td>Layer Description</td>
<td>RACS-1</td>
<td>RACS-2</td>
<td>Grading Medium</td>
</tr>
<tr>
<td>----------------------------</td>
<td>--------</td>
<td>--------</td>
<td>----------------------------------------------------</td>
</tr>
<tr>
<td>Stone subbase</td>
<td></td>
<td></td>
<td>Graded broken stone/sand gravel</td>
</tr>
<tr>
<td>Subgrade</td>
<td></td>
<td></td>
<td>A, B filler, improved soil</td>
</tr>
<tr>
<td>Bank beneath bed</td>
<td></td>
<td></td>
<td>A, B, C filler, improved soil</td>
</tr>
</tbody>
</table>

*Note:* *full section, 13.2 m for width of stone subbase layer, 1:1.5 slope of embankment; **average modulus in high temp.*

![Figure 2: Preparations for numerical modeling](image)

(a) Calculating model after meshing

(b) Simplified loading-time curve

**Fig 2. Preparations for numerical modeling**

3.3.2. Data extracting scheme

Six typical data extracting paths were used for the following responses analysis. Along wide direction, path 1, path 2, path 3, path 4, and path 5 were on the bottom of slab, CA mortar layer, base (HMA layer if RACS-2), HMA layer (crushed stone subbase if RACS-1) and crushed stone layer respectively. Path 6 was along the vertical direction passed through one of inner wheel loading points.
IV. RESULTS ANALYSIS

4.1. Horizontal dynamic stress

The horizontal dynamic stress-distance relationships along paths 1-5 are shown in fig 3.

4.1.1. Horizontal dynamic stresses at the bottom of slab

The transversal and longitudinal dynamic stresses along path 1 are shown in fig 3(a). The horizontal stress of RACS-2 was the largest among the three structures. The transverse tensile stresses were generated on both two sides about 0.4 m in length along wide direction both RACS-1 and SlabTrack structures. The longitudinal stresses at the bottom of the slab were all compressive, because the composite modulus of RACS-2 was the smallest among the three structures. The maximum horizontal tensile stress on the bottom of slab for RACS-1 was greater than that of SlabTrack, causing the flexibility of HMA reinforced subgrade to be improved. This is similar to increasing the stiffness of a spring.

4.1.2. Horizontal stresses at the bottom of CA sand mortar

The transversal and longitudinal dynamic stresses found along path 2 are shown in fig 3(b). The pattern of the horizontal dynamic stresses for all three structures along path 2 was similar. Stress magnitudes were greatest along the sides and smaller in the middle. The general stress level for RACS-2 is obviously larger than the others. On the ends of the layer, the transverse and longitudinal stresses of RACS-2 were about 10 kPa and 16 kPa respectively, whereas the horizontal stresses both of RACS-1 and SlabTrack were about 6 kPa. In the middle of the layer, the transverse and longitudinal stresses of RACS-2 were about 4 kPa and 9 kPa, respectively, but negligible for the other two structures. This meant that using an HMA base layer caused the stress level of the CA layer to increase. This layer did not function as a load carrying element. From this result, the structures beneath the base layer should maintain sufficient stiffness; otherwise, the distribution of stress in the upper structure would seriously be negatively affected. This will also shorten the life of the whole structure, but the stiffness only can be changed in a limited scope to improve the stress distribution, which is clear if compared RACS-1 to SlabTrack.

4.1.3. Horizontal stresses at the bottom of base

The transverse and longitudinal dynamic stresses were computed along path 3. At the base of the RACS-2 structure was the HMA layer; data curve is shown in fig 3(c). The graphs show that when HMA was used at the base to support the upper structure directly, the stress level was lower compared to the other two structures in either the transverse or longitudinal direction. The transverse and longitudinal stresses generated by RACS-2 were compressive in magnitude, and measured about 8 kPa and 15 kPa, respectively. They were also relatively uniform along the horizontal direction. This is because the HMA layer, as a flexible base layer, does not have sufficient bearing capacity to “resist” the train loading passed to the crushed stone layer. For RACS-1 and SlabTrack, the bottom of the bases generates a significant transverse tensile stress and longitudinal compressive stress. The difference between the results of the two structures is
that the transversal tensile stress of RACS-1 is greater than SlabTrack, while the longitudinal compressive stress of SlabTrack is greater than RACS-1. This shows that the base layer played an important role for bearing, and should have sufficient stiffness. In addition, the HMA reinforced subbase layer can positively affect the distribution of stress acting on the base, particularly the load-bearing function.

4.1.4. Horizontal stresses on the bottom of HMA layer in RACS-1

When HMA replaces the top part of the crushed stone layer in RACS-1, the path used to find the stresses acting on the three structures is along path 4. The horizontal stresses of three structures were different and are shown in fig 3(d).

(a) Horizontal dynamic stresses-distance relationships along path 1

(b) Horizontal dynamic stresses-distance relationships along path 2

(c) Horizontal dynamic stresses-distance relationships along path 3
From the figure, the stress tends to mutate at the edges of the base. This is due to the stress concentration caused by the cut effect of base edges. The transverse stresses under the wheel loadings was compressive in stress for RACS-1 and was measured to be about 7.5 kPa, which was distributed more evenly in the transverse direction. The transverse stress of RACS-2 showed characteristics with smaller values on both ends, but bigger in the middle (about 2.5 kPa maximum along the path). The overall compressive stress of SlabTrack was about 3.5 kPa, and was distributed more evenly in the transverse direction. For longitudinal stresses, however, the average compressive stresses were 10 kPa for RACS-1, 7.5 kPa for RACS-2, and 5 kPa for SlabTrack.

The analysis above shows that a subbase layer reinforced by HMA can positively improve the distribution of stresses on the track substructure regardless of transverse or longitudinal direction. Although there was a slight improvement of the stress in the longitudinal direction, the weakening capacity of the load resistance and the stress distribution was not stable for RACS-2. The HMA constructed at the top of the crushed stone layer can make the vertical modulus transition smoother, reducing the difference of modulus between layers of the PCC base and crushed stone subbase. By doing this, the distribution of the dynamic response of railway substructure will be more reasonable, lowering the stress level of subgrade surface, which has great significance to the sustained and stable long-term performance of the high-speed railway substructure.
4.2. Vertical deformation

The vertical deformation of three whole structures was generated at the maximum response value shown in fig 4. The results showed that RACS-2 generated the minimum dynamic tensile deformation and the maximum dynamic compressive deformation. As for the other two structures, the deformation of RACS-1 was slightly smaller than SlabTrack.

The maximum vertical dynamic deformation along path 6 from the loading central point on slab is shown in the fig 5(a). The central point load-time curves of the vertical deformation are drawn in fig 5(b). Fig. 5(a) shows the vertical deformation is decreasing rapidly with the depth. In the depth from 0 (at the top of slab) to about 2 m, the vertical deformation of RACS-2 was greater than the other two structures. The three would be very close if the depth was over than 2 m. The related curve of RACS-2 was more stable because HMA replaces the concrete as the base material, thus greatly weakening the upper composite modulus of the railway substructure.

![Graph showing maximum vertical deformations](image1)

**Fig 4. Maximum vertical deformations**

![Graph showing vertical deformation-depth curves](image2)

**(a) Maximum vertical deformation-depth curves**

![Graph showing deformation-time curves](image3)

**(b) Deformation-time curves**

**Fig 5. Vertical deformation relationship with depth & time**
Fig 5(b) shows the maximum vertical deformation of all three structures occurring at 0.081 s. As for RACS-2, the deformation with the time was always greater than the other two structures, and RACS-1 was similar but slightly smaller than SlabTrack. The curve also shows that the remnant of vertical deformation occurs over the end of one circle for the loading time. Thus, for traditional slab track, it is very difficult to recover the remnant deformation because of the rigid concrete material. However, HMA, being a flexible and visco-elastic material, has good capacity for recovery, which would be very important to the long-term performance of the track structure, especially for the high-speed railway substructure.

4.3. Vertical acceleration

The maximum vertical acceleration responses of the three structures are shown in fig 6. The value of RACS-1 was less than SlabTrack, while peak accelerations for RACS-2 were about 45% greater than SlabTrack and RACS-1.

Fig 7 shows the vertical acceleration relationship with depth and time. Along the depth direction from the center of the loading region, the maximum vertical accelerations of the three structures are shown in fig 7(a). The vertical acceleration-time relationship at the central load point on the slab is shown in fig 7(b).

Fig 7(a) shows vertical acceleration with depth in the subgrade. The highest accelerations are near the top of the subgrade and they decrease sharply in the first two meters which is consistent with field measurements. Fig 7(b) shows that the acceleration, especially the peak value of RACS-2, was significantly larger than the other two structures. Additionally, compared to SlabTrack the vertical acceleration of RACS-1 is slightly smaller.
V. CONCLUSIONS

This numerical analysis viewed horizontal stresses, vertical deformation and vertical acceleration under the same loading and interlayer contact conditions. At the depth of 1 m, the substructure of the slab track consists of PCC slab, CA sand mortar, PCC base and crushed stone layer. This was also the particularly effective depth to lower the vertical vibration by controlling the vertical acceleration. After analyzing and comparing the two HMA substructures to the traditional slab track, the following was found:

(1) The composite modulus of HMA reinforced subgrade can be improved relative to traditional slab track and HMA substructure has ability to recover the residual vertical deformation to a certain extent with circular loadings, which would be benefit for the long-term stability of the track structure.

(2) The comprehensive responses of RACS-1 were similar with that of SlabTrack, but the HMA substructure can positively affect the stress distribution. The stress distribution of RACS-1 at the stone subbase layer could be improved when compared to SlabTrack. The reasonable layer’s combination, especially the modulus match of the substructure, will enhance the stress distribution.

(3) At a depth of about 2 m, the vertical deformation of RACS-2 was always larger than RACS-1 and SlabTrack, and RACS-1 had a slightly smaller deformation and acceleration in vertical direction than SlabTrack. The deformation of three structures was very close over 2 m deep. Thus, the depth from 0 to 2 m, especially to 0.5 m, is the important depth for weakening dynamic loading.

(4) At the depth of 2 m, the vertical acceleration of the three structures were very close to 0. However, RACS-2 was larger from 0 to 2 m deep, and its peak value of vertical acceleration was also slightly larger than the other two structures. Thus, the depth from 0 to 2 m was still the effective depth for vibration control.

The results show that HMA is a very suitable material for railway substructure to enhance resilient performance, improve stress distribution, weaken the dynamic loading, and lower

\[
\text{(b) Acceleration-time curves}
\]

*Fig 7. Vertical acceleration relationship with depth & time*
vibration in the effective depth from 0 to 2 m especially to 0.5 m deep. The HMA constructed at the top of the stone subbase layer can make a smooth transition of the vertical modulus for each layer. From the comprehensive analysis of dynamic behavior, RACS-1 can be taken as a good option for practice, while RACS-2 needs further research before a conclusion made.

References
APPLICATION OF JERK IN ACTIVE SAFETY SYSTEMS

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Summary: In this article a new look at derivative of acceleration, known as jerk, is presented. First of all, it concerns application of jerk in transportation and automotive engineering, particularly, in active safety systems of road vehicles.

Keywords: Road safety; intelligent active safety systems; dynamic stabilization; jerk.

I. INTRODUCTION

Application of active safety systems is aimed at improvement of the global road safety situation. Such systems can be classified by control logic.

There are five types of control logic altogether that can be distinguished:

1. Hardwired (crisp) logic - algorithm with a single main path;
2. Fuzzy logic - algorithm with a few main paths;
3. Intelligent (adaptive) logic - ability to cumulate information;
4. Free logic - ability to react differently in equal situations;
5. Unpredictable logic - ability to react inadequately.

Obviously, free and unpredictable logics are peculiar strictly to the human being. An abiotic system with free logic would be nothing but artificial intelligence. Automated systems can be adaptive but will always tend as asymptotically to free logic.

Anti-lock braking system (ABS) is a typical representative of systems with hardwired logic. Recently, active safety systems with fuzzy logic have become widespread. However, low level of road safety still demands invention of new active safety systems with intellectual logic that is undoubtedly more progressive and modern.

An intelligent active safety system (IASS) is supposed to consist of such functions as distance regulation, adaptive braking forces regulation, wheelspin regulation, dynamic stabilization, active suspension, tire pressure regulation. General functional scheme of IASS should include following stages:
1. Determination - conversion of signals from sensors into information about current road traffic situation, driver's activity, technical state of vehicle control systems (first of all, braking and steering systems).

2. Estimation - analysis of entry data, comparison of current and critical meanings of entry parameters, identification of emergency situation.

3. Prediction of subsequent development of road traffic situation.

4. Optimization - refinement of motion parameters by means of specific influence on vehicle control systems, correction of control actions.

5. Accumulation of experience for its application in the sequel.

Algorithms of existing dynamic stabilization systems (for example, “Toyota”, “WABCO”, “Knorr-Bremse”) are usually based on registration of such parameters as yaw angle, lateral forces on wheels, lateral acceleration. The listed parameters help to determine a critical situation during motion of vehicle successfully, but they are not enough to prevent such a situation. Realization of IASS demands new approaches, one of which is application of first derivative of acceleration.

II. JERK AND ITS APPLICATION

It is very well known that the first derivative of position with respect to time is velocity, and the second one is acceleration, as well as that force is the time-derivative of momentum. It is much less well known that certain terms are used in physics to name higher derivatives of position and momentum. The third derivative of position, or the rate of change of acceleration, is called “jerk” or (mainly in British English) “jolt”. Other terms have appeared in individual cases for the third derivative, such as “pulse”, “impulse”, “bounce”, “surge”, “lurch”, “shock”, “super acceleration”, that are generally less appropriate, either because they are used in engineering to mean other things or because the common English use of the word does not fit the meaning so well. For example, “surge” is used by electricians to mean something like rate of change of current or voltage. In 1990 in ISO 2041 it was officially proclaimed: “1.5 jerk: A vector that specifies the time-derivative of acceleration.” By the way, the symbol “j” was not standardized for its notation. Terms “snap”, “crackle” and “pop” are used for the 4th, 5th and 6th derivatives of position respectively. Also, terms “yank”, “tug” “snatch” and “shake” have been suggested for the 2nd, 3rd, 4th and 5th derivatives of momentum. Needless to say, none of these terms have appeared in any standard yet.

Jerk is used in different engineering fields.

1. In the aerospace industry a “jerkmeter”, a special device for measuring jerk, is applied. In case of the Hubble Space Telescope, one of the greatest observatories in the world that was launched in April of 1990 and helped to derive a new cosmological model of the accelerating expansion of the Universe, the engineers have specified limits on the magnitude of snap, i.e. the
derivative of jerk.

2. Evaluation of g-overload on human in aerospace industry and when building roller coasters. For example, consciousness loss of pilots and cosmonauts depends not only on amplitude of acceleration, but also on the character of its change [1].

3. Estimation of discomfort caused to the driver and passengers in a vehicle. When designing a train, engineers are typically required to keep jerk within 2 m/s³. Consideration of maximum jerk is especially important in order to prevent passengers, who need time to sense stress changes and adjust their muscle tension, from suffering conditions such as whiplash.

4. Mechanical engineering. Evaluation of the destructive effect of motion and machining vibrations (chatter) on mechanisms in manufacturing. Movement of delicate instruments such as cutting tool needs to be kept within specified limits of jerk to avoid its damage and premature wear. Jerk is also considered in the cam profiles development because of tribological implications (ability of the actuated body to follow the cam profile without chatter).

5. Robotics and automation. In motion control, it is commonly necessary to move a system from one steady position to another (so called “point - to - point motion”). The fastest possible motion within an allowed maximum value for speed, acceleration, and jerk, results in a third - order motion profile that, in general, consists of 7 segments:

a) Acceleration build-up, with maximum positive jerk;

b) Constant acceleration (zero jerk);

c) Acceleration ramp-down, approaching the desired maximum velocity, with maximum negative jerk;

d) Constant velocity (zero jerk, zero acceleration);

e) Deceleration build-up, with maximum negative jerk;

f) Constant deceleration (zero jerk);

g) Deceleration ramp-down, approaching the desired position, with maximum positive jerk.

6. Jerk systems in microelectronics. A system whose behavior is described by a jerk equation is called “jerk system”. Certain simple electronic circuits may be described by a jerk equation. These are known as “jerk circuits”. One of the most interesting properties of jerk systems is the possibility of chaotic behavior. In fact, certain chaotic systems are conventionally described as a system of three first-order differential equations, but which may be combined into a single (although rather complicated) jerk equation.

7. Estimation of vertical vehicle motion smoothness [2, 3].

8. Estimation of longitudinal vehicle motion smoothness and its transmission performance quality [4]. The highest longitudinal jerk is excited during clutching and abrupt gas pedal release. It has been experimentally shown that the magnitude of jerk when shifting gears is
permissible within 3.4 g/s [5].

III. MATHEMATICAL MODEL

A research, carried out in MADI, has shown the possibility of application of jerk to improve lateral dynamic vehicle stability. It has been experimentally proved that lateral acceleration of the first axle noticeably advances yaw speed, but lags behind its own derivative, i.e. lateral jerk.

Another advantage of an algorithm based on lateral jerk is absence of parasite initial deviation. Such deviation of acceleration may excite for one of three reasons:

1) Installation inaccuracy of acceleration sensor;
2) Damaged geometry of body or frame due to an accident;
3) Parasite voltage excited by acceleration sensor.

During curvilinear motion kinematic parameters can be related by two linear dependences:

(A) Steering angle $\alpha$ - lateral acceleration $q$;
(B) Steering speed $\omega_{SW}$ - lateral jerk $j_Y$.

In the present article it is proposed to realize a qualitative leap from (A) to (B), in other words, to apply lateral jerk of front axle for prediction of emergency situation.

Positive jerk ($j > 0$) is being excited with transition from ($a = 0$) to ($a > 0$), or from ($a < 0$) to ($a = 0$), or from ($a_1$) to ($a_2$), where ($a_2 > a_1$). Negative jerk ($j < 0$) is being excited with transition from ($a = 0$) to ($a < 0$), or from ($a > 0$) to ($a = 0$), or from ($a_1$) to ($a_2$), where ($a_2 < a_1$).

Three types of jerk can be distinguished: instantaneous (elementary), optimized and intensive. Instantaneous jerk is equal to increment $da$ of acceleration in any elementary period $dt$ of time:

$$j = \frac{da}{dt} = \frac{a_2 - a_1}{t_2 - t_1}$$

Where $t_1$ and $t_2$ - initial and terminal moments of time respectively.

Preliminary analysis has shown that behavior and peak-to-peak amplitude of jerk depend significantly on differentiation step $\tau = t_n - t_{n-m} = m\tau_0$, where $\tau_0$ is sensitivity of chronometer.

Thus, for proper forecasting of emergency situations and in order to avoid misoperation, in mathematical models of IASS it is required to apply optimized jerk:

$$j_n = \frac{a_n - a_{n-m}}{\tau}.$$

Intensive jerk is calculated in period $T_{int}$ of stable acceleration growth from zero to
amplitude:

\[ j_{\text{int}} = \frac{a_{\text{max}}}{T_{\text{int}}} \]

Jerk is expressed by way of corner radius R:

\[ j = \frac{v_2^2 R_1 - v_1^2 R_2}{R_1 R_2 (t_2 - t_1)} \]

But in certain cases when regarding a transitional process it is convenient to use curvature \((K=1/R)\) of motion trajectory:

\[ j = \frac{v_2^2 K_2 - v_1^2 K_1}{t_2 - t_1} \]

The most common driver’s actions, leading to a vehicle stability loss, are following:

1. Abrupt bypass of a sudden obstacle due to driver’s inattention;
2. Hard braking due to nonobservance of frontal distance;
3. Braking during curvilinear motion.

Abruptness of steering and braking are inversely proportional to frontal distance. Excitation of lateral jerk in the case (2) and its absence in the case (3) tell unambiguously about rear and front axle skidding respectively. The case (1) is more complicate in respect to emergency situation prediction.

(Fig 1. Fig 2.)

Long-term observations have shown that the most unstable moments for passengers and cargo in a vehicle are short periods when frontal \((j_X)\) or lateral \((j_Y)\) jerk is nonzero \((j \neq 0)\). Jerk forces an uncontrollable displacement of mass centre of passenger or cargo (if it is improperly fixed). Magnitude of displacement depends on amplitude of intensive jerk. A humanlike system, consisted of many joints, cannot resist this displacement. When a passenger is rigidly fixed, the whole overload affects his pliable internal organs.

(Continuously see page 48)
SOME NEW RESULTS OF EXPERIMENT AND COMPUTING OF THE FATIGUE CRACK GROWTH IN THE BOGIE FRAME OF LOCOMOTIVE TYPE D19E RUNNING ON THE VIETNAM RAILWAY

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*Hung Yen University of Education and Technology*  
PROF. DR. DO DUC TUAN  
MS. PHAM LE TIEN  
*Hanoi University of Communication and Transport*

**Summary:** The following contents should be presented by author.

1. Research Phenomena of cracks at bogie frame of the locomotive of type D19E which have been running in the Vietnam railway in the Hanoi locomotive enterprise;
2. Computing the Fatigue Safety ratio with any crack length with help Maple;
3. Performance that safety ratio and Crack propagation crack properties;
4. Suggestions technology to stop cracks at the bogie frame of locomotive.

Process of computing and performance with help Maple basing on the measurable data of Test and Analysis Report on #903 and #907 locomotives of Chinese and Vietnamese scientific Organizations.

I. **INTRODUCE**

At the present time, in Vietnam there are 20 locomotives of type “Innovation” imported from China in 2003. After some running time on the railway, in the locomotive direction frame (LDF) the dangerous crack phenomena had been appearance before, have been appearance [7]. It is very hot situation which must be to research [3, 4, 5], fig.1.

![Crack on LDF](image)

For solving that crack problems, while locomotives (D19E 903 and D19E 907) had been
running on the railway, the NDT method and measurements on some dynamic parameters of FCG have been made by Chinese [3,4] and Vietnamese [5] scientific experts. The difference dangerous positions and points have been showed their results had been showed in some anti concepts [7].

However it still is not to solve the FCG. In this my work, the problem has set that what is the Fracture toughness (K_{IC}) and how to predict the lifetime of LDF by the FCG equation?

With above purpose, it must be to do some fatigue tests and some tests on the FCG by international standards [6] in the Vietnam National Laboratory (VILAB -185).

Firstly, it had made the test on fatigue with specimens for rotation bending to inspect Machine, specimen, fig 3, procedure and processing method had been obeyed by TCVN 257 -85 [2], fig 3. According to China TB/T1335-1996 “Strength Design and Test Appraisal Specification for Railroad cars”, the allowable safety stress of material Q345A under rotation bending is \( \sigma_s = 216\text{MPa} \) [3].

Secondly, it had made the test on FCG. The results of tests will show bellow.

![Fig 2. Cracks zone on LDF](image)

Some of researching results had been publicizing before [5, 6, and 7]. And now there are newest results of our experiments on fatigue and Fracture Mechanics of LDF.

**II. EXPERIMENTS ON FATIGUE**

2.1. Mechanical Chemical prosperities of Fatigue Specimens had been determine in laboratory of COMFA - Hanoi ...

2.2. Fatigue tests had been operating in the laboratory of University of Communication and Transports and with guide of TCVN showing in the literature [1].

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2.3. Data Processing of experimental data had been done by professional software. Results of Its should be seen, for example fig 4, fig 5.

**Fig 3. Fatigue Specimens for rotation bending tests**

**Fig 5. Probability distributions of Fatigue Experimental Function at 5-stress level**

### III. COMPUTING THE FATIGUE SAFETY FACTOR WITH HELP MAT-SOFTWARE MAPLE

Basing on the database of them, the Fatigue safety factor [1] had been computing with help of Mat- software Maple by us [1]. Some information of results has been showed in table 1, (with database of China), and table 2, (with database of Vietnam).

**Table 1. Results of the computing fatigue safety factor of LDF of D19E 903[1, 6]**

<table>
<thead>
<tr>
<th>At point</th>
<th>$\sigma_{\min}$, Map</th>
<th>$\sigma_{\max}$, Map</th>
<th>$\sigma_{a}$, Map</th>
<th>$\sigma_{m}$, Map</th>
<th>r</th>
<th>s</th>
</tr>
</thead>
<tbody>
<tr>
<td>P14, outside</td>
<td>2,5</td>
<td>57,5</td>
<td>27,5</td>
<td>30</td>
<td>0.043</td>
<td>2,55</td>
</tr>
<tr>
<td>P15, Inside</td>
<td>2,5</td>
<td>107,5</td>
<td>52,5</td>
<td>55</td>
<td>0.023</td>
<td>1,34</td>
</tr>
<tr>
<td>P14</td>
<td>2,5</td>
<td>62,5</td>
<td>30</td>
<td>32,5</td>
<td>0.040</td>
<td>2,34</td>
</tr>
<tr>
<td>P15</td>
<td>2,5</td>
<td>92,5</td>
<td>45</td>
<td>47,5</td>
<td>0.027</td>
<td>1,56</td>
</tr>
<tr>
<td>Number of Database</td>
<td>$\sigma_{a}$, kgf/cm$^2$</td>
<td>$\sigma_{m}$, kgf/cm$^2$</td>
<td>$r$</td>
<td>$s$</td>
<td>Road speed, $v$, km/h</td>
<td></td>
</tr>
<tr>
<td>-------------------</td>
<td>-------------------------</td>
<td>-------------------------</td>
<td>-----</td>
<td>-----</td>
<td>----------------------</td>
<td></td>
</tr>
<tr>
<td>133-140</td>
<td>172.2</td>
<td>168</td>
<td>0.044</td>
<td>1.10</td>
<td>KHz NET m. p. 15-30</td>
<td></td>
</tr>
<tr>
<td>232-239</td>
<td>140.7</td>
<td>239.4</td>
<td>0.260</td>
<td>0.55</td>
<td>North Hay van m. p 15-30</td>
<td></td>
</tr>
<tr>
<td>8-19</td>
<td>136.5</td>
<td>480.9</td>
<td>0.553</td>
<td>1.33</td>
<td>At top m. p. 15-30</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>121.8</td>
<td>42</td>
<td>-0.487</td>
<td>1.57</td>
<td>Smooth, flat 60</td>
<td></td>
</tr>
<tr>
<td>31-32</td>
<td>193.2</td>
<td>86.1</td>
<td>-0.374</td>
<td>0.99</td>
<td>Smooth, flat 90</td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>126</td>
<td>27.3</td>
<td>-0.648</td>
<td>1.55</td>
<td>bad 45</td>
<td></td>
</tr>
<tr>
<td>52</td>
<td>120.75</td>
<td>33.6</td>
<td>-0.564</td>
<td>0.57</td>
<td>bad 35</td>
<td></td>
</tr>
</tbody>
</table>

$\sigma_{\text{Min}}, \sigma_{\text{max}}$ had been taken from results of measurements in documents [3, 4, and 5].

$$
\sigma_a = (\sigma_{\text{max}} - \sigma_{\text{min}})/2;
\sigma_m = (\sigma_{\text{max}} + \sigma_{\text{min}})/2
$$

And the Fatigue Safety Factor can be computing by:

$$
K := \frac{\beta \kappa_1 \kappa_2 \eta}{\varepsilon \mu}
$$

Where some effluence factors:

$\beta$ - ratio of geometrical stress concentration;
$\kappa_1$ - Ratio evidential material;
$\kappa_2$ - ratio of residual stress;
$\eta$ - ratio of errors welding procedure;
$\varepsilon$ - ratio of effectors absolute dimensions;
$\mu$ - Ratio of the quality of surface technology.

**Computing the Fatigue Safety Factor (FSF) from database of China [3]:**

(At point P4-The most dangerous point on the reinforced Frame of Meter-Gauged Diesel Locomotive) Hanoi, Aug., 14th, 2005,

Applying Mat software Maple, the results of computing are showing as bellow:

```maple
> restart;
> sigmaghmoi:=2142 kg/cm$^2$; # reversed-stress fatigue limit of material made the reinforced Frame $\sigma_{\text{r}} = 216\text{MPa}$ #
```
> betak:=2.7; kappa1:=1.15; kappa2:=1.25; eta:=1.0; epsilon:=0.5; mu:=0.70;
> betak := 2.7
κ1 := 1.15
κ2 := 1.25
η := 1.0
ε := .5
μ := .70
> K:=betak*kappa1*kappa2*eta/(epsilon*mu); #According with CSRI MPS USSR#
K := 11.08928571

Study case 1: the smallest amplitude and main stresses

<table>
<thead>
<tr>
<th>s_o, kgf/cm^2</th>
<th>s_m, kgf/cm^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>172.2</td>
<td>168</td>
</tr>
<tr>
<td>140.7</td>
<td>239.4</td>
</tr>
<tr>
<td>136.5</td>
<td>480.9</td>
</tr>
<tr>
<td>121.8</td>
<td>42</td>
</tr>
<tr>
<td>193.2</td>
<td>86.1</td>
</tr>
<tr>
<td>126</td>
<td>27.3</td>
</tr>
<tr>
<td>120.75</td>
<td>33.6</td>
</tr>
</tbody>
</table>

> ssigmachophep:=2.0;
ssigmachophep := 2.0
> sigmaa:=172.2; sigmam:=168; psi:=0.2;
sigmaa := 172.2
sigmam := 168
ψ := .2
> b:=(K*sigmaa+psi*sigmam);
b := 1943.174999
> ssgma:=(sigmaghmoi/b);
ssgma := 1.102319658

Small consolations 1: It has not met allowable safety factor!

Study case 2: The biggest amplitude and main stresses

sigmaghmoi := 2142

> sigmaghmoi/b;
\input{ex2a.txt}

Small consolations 2: It has not met allowable safety factor!

**Computing the Fatigue Safety Factor (FSF) from database of Viet Naam[4]:**

(At point P4-The most dangerous point on the reinforced Frame of Meter -Gauged Diesel Locomotive, but the best case for Chinese manufacturer) Hanoi, Aug., 14th, 2005

> restart;

> sigmaghmoi:=2600;#gioi han moicua vat lieu nguyen thuy lam khung gia chuyen huong#
> sigmaghmoi := 2600

> Deltasigmagiammax:=(80/100)*sigmaghmoi;#muc suy giam lon nhat gioi han moi cua vat lieu khi han thanh khung va lam viec #
> Deltasigmagiammax := (80/100)*sigmaghmoi
Deltasigmagiammax := 2080

> Deltasigmagiamin := (20/100)*sigmaghmoi; # muc suy giam nho nhat gioi han moi cua vat
lieu khi han thanh khung va lam viec #

Deltasigmagiamin := 520

> a80 := (2600-2080); # gioi han thap nhat con lai#
a80 := 520

> a20 := (2600-520); # gioi han thap con lai#
a20 := 2080

> a100 := sigmaghmoi; # gioi han cao nhat con lai#
a100 := 2600

> b := (K*sigmaa + psi*sigmam); 
b := K sigmaa + psi sigmam

> K := betak*kappa1*kappa2*eta/(epsilon*mu); # According with CSRI MPS USSR #
K := \frac{betak \times kappa1 \times kappa2 \times \eta}{\epsilon \times \mu}

> betak := 1.0; kappa1 := 1.0; kappa2 := 1.0; eta := 1.0; epsilon := 0.5; mu := 0.70;
betak := 1.0

kappa1 := 1.0

kappa2 := 1.0

eta := 1.0

epsilon := .5

mu := .70

Study case 1: the smallest amplitude and main stresses

> ssigmachophep := 2.0;
ssigmachophep := 2.0

> sigmaa := 210; sigmam := 315; psi := 0.2;
sigmaa := 210

sigmam := 315

psi := .2
Small consolation 1: It has met allowable safety factor!

Study case 2: The biggest amplitude and main stresses

Small consolation 2. It has not met allowable safety factor! The some newest results of computing on safety factor has been showed in ref [9].
It is made us to take experiments on \( \frac{da}{dN} \). Machine (a), Specimens (b), fig 6, and standard test method … had been obeying by ASTM E 647-00 and ASTM E 399-90 [5] with INSTRON Teguments of England. One of results has been showed on fig 7.

III. PERFORMANCE THAT SAFETY FACTOR AND CRACK PROPAGATION CRACK PROPERTIES

\[
\frac{dl}{dN} = \frac{C_s (\Delta K)^n}{(1 - r) K^m}.
\]

Fig 7. Result of test on Fracture toughness (Criteria \( K_{IC} \))

Fig 8. Graphic load versus extension

- Influence of some parameters on \( \frac{da}{dN} \) may be presented by equation [1, 2, 5, and 8]:
Where: \( K_{c} = \frac{(\text{lgf} + 11.75)}{0.185} \quad (4) \)

- Frequency of acting loads on LDF; \( K = K_{\text{max}} - K_{\text{min}} \); \( r = \frac{\sigma_{\text{min}}}{\sigma_{\text{max}}}; C_{5}; \)

- \( n \) - constants of material manufactured LDF. The other parameters in equations (1) and (2) can see in the Ref [7, 8].

It can be found final cracked length at fracture moment, for example, showing on the table 3.

**Table 3. Computing fracture cracked length at the moment \( K_{\text{max}} = K_{IC} \)**

<table>
<thead>
<tr>
<th>Road situation</th>
<th>( K_{\text{max}} ) (MPa.m(^{1/2}))</th>
<th>( K_{\text{min}} ) (MPa.m(^{1/2}))</th>
<th>( \Delta K ) (MPa.m(^{1/2}))</th>
<th>( r )</th>
<th>( C_{5} )</th>
<th>( n )</th>
<th>( l_{c} ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth, flat</td>
<td>0.936</td>
<td>0.679</td>
<td>0.257</td>
<td>0.72</td>
<td>8.17(10^{-11}) 3.38</td>
<td>0.0181</td>
<td></td>
</tr>
<tr>
<td>Mountain, pass</td>
<td>1.261</td>
<td>0.679</td>
<td>0.582</td>
<td>0.54</td>
<td>8.17(10^{-11}) 3.38</td>
<td>0.01</td>
<td></td>
</tr>
</tbody>
</table>

The lifetime of LDF can be predicted by equation [2,8]:

\[
N_{p} = \int_{l_{0}}^{l_{c}} \left( \frac{dN}{dl} \right) dl \quad (5)
\]

Where \( l_{0} \) - beginning crack length (as technique defect); \( l_{c} \) - final crack length at fracture moment \( N = N_{p} \).

- Mathematical Software Maple had been applying to compute FCG, for example, [8] and pro-Software, for example, [9].

- It has been computing the lifetime and fatigue safety factor of FCG with two cases: Railway on smooth and flat road, and Railway on hills and mounds:

According to Hanoi Locomotive enterprise, the lifetime of locomotive D19E is averaged approximately \( N = 2.5 \times 10^{5} \) every year.

The fatigue safety factor (s) versus ratio (r) has been performance as below, fig 9.

![Fatigue safety factor s versus ratio r](image)

**Fig 9. Performance: Fatigue safety factor s versus ratio r**

**IV. SUGGESTIONS TECHNOLOGY TO STOP CRACKS AT THE BOGIE FRAME OF LOCOMOTIVE**

While applying welded technology for treatment all appearance cracks, it must be with
V. CONCLUSIONS

- The experimental tests on \( \frac{da}{dN} \) and Fracture toughness (\( K_{IC} \)) by ASTM E 647-00 and ASTM E 399-90 in the standard laboratory maybe had been made by us firstly in Vietnam;
- Results of tests had been used for prediction lifetime of LDF with computing in the Maple environment, they alloy us to make some useful suggestions about treatments cracks in LDF.
- Loads on LDF give up stresses which are variable with \( r \approx 0 \), the fatigue safety factor maybe get maximum values. In this case, it can be said LDF has met duration.
- From our received results, it is one more to say that LDF has not met fatigue safety and cracks will be appearance again.
- It has been focusing that the practical technical important problems have been solving by new concepts and new theory of strengths of structures and machine components.

References
[6]. Ngo Van Quyet, Do Duc Tuan, Pham Le Tien: “Nghienn cuu hien tuong nut trong khung gia chuyen huong loai dau may D19E van dung o xi nghiep dau may Ha noi”. -Tap chi:”Khoa hoc giao thong van tai”, So 12 thang 11 -2005, trg 165-170.
[7]. Ngo Van Quyet, Do Duc Tuan, Pham Le Tien: ”Ve phuong trinh lan truyen vet nut moi” - Tap chi “Khoa hoc giao thong van tai”, So 12 thang 11 -2005, trg 217-231.
I. INTRODUCTION

A part from fuel burning emissions in the form of roadway dust add to a high amount of particulates in transport. The discussed sources of the roadway dust include in particular formation of particles by mechanical roadway separation (bitumen particles, soil dust). Other sources include rubbing action of tyres and brake pads. Other discussed sources of roadway dust comprise particulates from chemical material (spreading salt) and inert spreading material (sand, cinder) used for winter maintenance, dirt dropping off from cars and losses from overloaded materials. Due to their size, all these particles quickly deposit on the surface of the roadway or

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close to their sources. They get back in the air by a resuspension process as a result of turbulent air flow initiated by preceding vehicles or whirls caused by blowing wind.

II. PARTICULATE MATTER MONITORING

Sites representing both non-urban and urban roads with various traffic volumes and various surface types were selected to monitor concentrations of specific fractions of the particulate matter at locations with different traffic volume.

Particulate matter was measured by two methods.

Some measurements were done using an ENVItech mobile laboratory (fig 3), where air samples for continuous measurement of flue dust were taken using an independent dust counter sampling probe. The monitoring station is equipped with analyzing sensors for measuring specific polluting agents (normative methods), telescopic meteorological mast, control and assessment software and GSM module for the remote transmission of measured data. The PM10 sampling head is located 1.8 m above the roof of the measuring station. Beta radiometry-based dust counter Eberline FH 62 I-R was used. Partial evaluation of measurements is shown in charts on fig 10.

Air samples were taken using medium-volume LECKEL MVS6 sampling pumps. The devices are intended for outdoor use at high or low temperatures. The flow of the air to be taken is controlled and basic physical parameters are maintained by means of a thermally compensated slot. PM capturing filters are inserted between Teflon holders into a sampling head forming an integral part of the device. Filters from nitrocellulose fibres were used to capture PM.

Measurements were done in compliance with the European standards EN 12341 [8] and EN14907 [9], where this methodology is used as a reference method.

Particle concentrations were determined gravimetrically from every exposed filter. Traffic volume was recorded continuously using an automatic traffic intensiveness detector SIERZEGA SR4. Detector - the microwave radar Works at the base of dopplers principle (fig 1).
Multiple measurements were taken at four sites between 2007 and 2009. Most measurements focused on PM$_{10}$ monitoring. Representation of different particles was monitored at two sites in July and October 2008, taking samples to monitor chemical composition of PM.

The positions were selected as follows:

1. D1 highway, Predmier - four-lane road, surface - stone mastic asphalt (SMA);  
2. II/507 Bytca, bypass road - two-lane road, surface - stone mastic asphalt (SMA);  
3. I/18 Zilina, city through road - four-lane road, surface - asphalt concrete (AC);  

Dependency between temperature and particulates concentration was shown at the monitored positions (fig 4 to 7), but no dependency between average values of particulates formed and total traffic volume was proven. This can be proven, for example, by the monitored section No. 4 in Dolný Hričov with the D1 highway opened early in 2008, so traffic volume at the I/18 road significantly decreased, but with no effect on PM$_{10}$ particulate concentration (Fig 7).
Continuous control measurements were made using mobile monitoring equipment in cooperation with ENVItech, s.r.o. Trenčín (Fig 8 and 9). However, traffic count was done only manually during the continuous measurement. This partial measurement, on the contrary, indicates an interim dependency between the currently increasing traffic volume and the amount of PM concentration.

**Fig 4.** Long-term observation at the measuring position 1, D1 highway, Predmier

**Fig 5.** Long-term observation at the measuring position 2, II/507 road, Bytca bypass

**Fig 6.** Long-term observation at the measuring position 3, I/18 Zilina city through road
Fig 7. Long-term observation at the measuring position 4, I/18 road Dolny Hricov

Fig 8. Course of dependency between particulate matter PM$_{10}$ formation and air temperature (position 4 – Dolny Hricov – 20.4.2008) [8]

Fig 9. Course of dependency between particulate matter PM$_{10}$ formation, traffic and air temperature (position 4 – Dolny Hricov – 19.4.2008)[8]
When the new pavement surface was laid down, formation of particulate matter was reduced, probably due to a better bond between the fine aggregate and the binding material.

III. CHEMICAL ANALYSIS OF SAMPLES TAKEN

Chemical analyses of samples taken at position 1 were done to determine content of selected metals (ICP/MS, Agilent 7500ce). The aim was to identify differences in PM composition resulting from operation of vehicles on roads with different surfaces. To evaluate metal content in PM$_{10}$ fraction we cooperated with Centrum dopravního výzkumu Brno (Transport Research Centre in Brno).

It is assumed that inorganic particles are formed only by abrasion of cement-concrete pavements. These particles therefore represent 90% of the particles resulting from abrasion of asphalt-concrete pavements [4] and they consist mostly of coarse fraction PM.

Contents of selected metals, representing sources related to mechanical abrasion of particles, such as Zn, Sb, Cu, Ba and other, ware identified in both fine and coarse PM fractions. Contents of selected metals in both PM fractions were higher at sites with asphalt-concrete pavement at both collection campaigns, except K and Pb in the second collection campaign in the autumn (fig 12).

Samples taken were analysed further to determine volume of selected metals (ICP/MS, Agilent 7500ce) and PAH - polyaromatic carbohydrates (GS/MS, Shimadzu QP2010) and to identify differences in PM composition due to vehicles operated on roads with different surfaces (fig 10, 11). The PM$_{2.5}$ fraction with the content of polyaromazic carbohydrates related to the amount of particulate matter was evaluated separately. Higher PAH concentration was identified during the autumn collection campaign at both sites; the PAH concentrations were higher in both campaigns also at sites with asphalt-concrete pavement. The lower PAH quantity during the summer campaign is likely to result from PAH occurrence mostly in its gasesous form due to higher temperatures, or from PAH degradation due to sunshine.
IV. DISCUSSION ABOUT RESULTS

According to foreign studies, abrasion of tyres, brake shoe lining and roadway contributes significantly to non-combusted traffic emissions. For example [6] or [7] suggest that the share of non-combusted PM$_{10}$ emissions in total traffic pollution is 50% at present and 50% comes from combustion processes. Considering the renewal of vehicle fleets and use of new fuel types it is likely that the share of non-combustion emissions will become higher.

Analysis of principal components (PCA) for emission source quantification in transport was done by [5]. A component analysis transforms initial variables into orthogonal quantities summarising variances of the initial variables. However, it is up to interpretation whether these new components represent artificial characteristics or whether they reflect real factors.

Thurston [5] specifies the substances that achieve the highest component values. For tyre and brake shoe lining abrasion, these are benzothiazole, zinc (Zn), copper (Cu), antimony (Sb), titanium (Ti), nickel (Ni); for roadway abrasion, these are nickel (Ni) and vanadium (V); and calcium (Ca), aluminium (Al), barium (Ba) in case of resuspendation.

Based on the chemical analysis done at measuring stations (fig 12) it can be assumed that the data obtained on higher Ca, Zn, and Cu values among the chemical substances monitored show that the particulates captured in flow pump filters can originate from tyre and brake shoe lining abrasion and resuspension. The share of road surface abrasion is negligible. Comparison of effects of roadway surface clearly shows that abrasion of components was higher on a roadways made from asphalt concrete than on roadways made with mastic asphalt surface.

These are just partial results of our research work. The monitoring should be complemented by other measurements to have sufficient number of samples for a relevant statistical evaluation. This research work is under way.

Decree on air quality in the SR [12] establishes an annual limit value for the protection of human health particulate - 40 µg.m$^3$. Observed on sections of roads have been measured values that exceed the hygienic established limit.

The road surface with the SMA achieved higher levels of abrasion particles (measured concentrations are reached in their area of 31 to 109 µg.m$^3$), but to exceed the limit there to
a lesser extent (2 of 6 measurements at the stand 1 and 5 of the 7 measurements at the stand 2).

The road surface with the AC achieved the values of abrasion particles slightly lower than the road surface with the SMA (measured concentrations are reached in their area of 31 to 85 μg.m\(^{-3}\)), but exceeded the hygienic limit to protect public health more frequently (5 of 5 measurements at the stand 3 and 6 of 8 measurements at the stand 4).

According to the decree on air quality \[12\] are currently set limits on air pollution for As, Cd, Ni, Pb.

There were monitored only Ni and Pb. The limit value for Ni is 20 μg.m\(^{-3}\), measured values for surface with AKM were 8 μg.m\(^{-3}\) and for surface with AC were 11 μg.m\(^{-3}\)

Similarly for lead (Pb) the decree set limit value is 0,5 μg.m\(^{-3}\), but the measured values for surface of SMA were 7 μg.m\(^{-3}\) and for surface of AC were 25 μg.m\(^{-3}\).

We can say that on the both surfaces of roads were monitored exceeded the permitted hygienic limits.

References

[12]. Decree of the Ministry of the Environment of the Slovak Republic No. 360/2010 Coll. of Laws on air quality
INTRODUCTION

Settlement control is one of key technologies in high-speed railway embankment projects. With the development of high-speed railway in China, more embankments were typically built on deep medium compressibility soil. Both in China and abroad, the studies\textsuperscript{[1-3]} on settlement deformation behaviors of soil through the geotechnical centrifuge were mainly conducted on soft soil. The Settlement deformation behaviors of soft soil have been well understood. When high-speed railway is built on deep medium compressibility soil, however, little is known about the deformation characteristics of embankment, such as the post-construction settlement and settlement velocity. Therefore, the study on the settlement deformation behavior of deep soil...
and the control technology is important for engineering practice.

Hainan east loop line, a passenger dedicated line in Hainan, China, has an embankment constructed on a deep, medium compressible, saturated, completely decomposed granite soil. In this paper, the settlement deformation behaviors of this soil are studied using the geotechnical centrifuge in Southwest Jiaotong University, Chengdu, China. The obtained conclusions may provide a guide to the embankment design and construction on the deep soil of the same kind.

I. ENGINEERING BACKGROUND

Engineering prototype of the centrifuge model test is the sections DK67+630 and DK67+666 on Hainan east loop line, with a hummocky topography. Section DK67+630 is three-layer foundation soil, including, from top down, soft soil (4) (Q4 dl+pl) with a thickness of about 0.6 m, completely decomposed granite (6-W4) with a thickness of about 25 m, and strongly weathered granite layer (6-W3). Section DK67+666 foundation soil is composed of five layers, from top down, soft soil (4) (Q4 dl+pl) with a thickness of about 3 m, completely decomposed granite (6-W4) with a thickness of about 12.6 m, and a strongly weathered granite layer (6-1-W4) with a thickness of about 7.2 m which is characterized by loose structure, high moisture content, stiff-plastic to soft-plastic range, completely decomposed granite (6-W4) with a thickness of about 10 m; strongly weathered granite layer (6-W3). The physical and mechanical parameters of the soil are shown in table 1.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Density ρ (g/cm³)</th>
<th>Natural moisture content w/%</th>
<th>Liquid limit wL/%</th>
<th>Plastic limit wP/%</th>
<th>Void ratio e</th>
<th>Natural quick-shear strength</th>
<th>Vertical compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>(4)</td>
<td>1.99</td>
<td>12.8</td>
<td>21.1</td>
<td>12.6</td>
<td>0.49</td>
<td>12.6</td>
<td>27.7</td>
</tr>
<tr>
<td>(6-w4)</td>
<td>1.90</td>
<td>49.0</td>
<td>64.7</td>
<td>32.1</td>
<td>1.26</td>
<td>33.8</td>
<td>17.4</td>
</tr>
<tr>
<td>(6-1-w4)</td>
<td>1.85</td>
<td>32.3</td>
<td>36.2</td>
<td>21.4</td>
<td>0.96</td>
<td>23.1</td>
<td>22.5</td>
</tr>
<tr>
<td>(6-w3)</td>
<td>2.30</td>
<td>/</td>
<td>/</td>
<td>/</td>
<td>/</td>
<td>/</td>
<td>35.0</td>
</tr>
</tbody>
</table>

II. TEST

2.1. Model design

A simulation was made for the single-track railway embankment and deep completely decomposed granite foundation soil in the test. The surface width of the subgrade bed is 8.2m; the center height of the embankment cross-section is 5 m for DK67+630, and 3.99 m for DK67+666; and the slope ratio is 1:1.5. The vertical distance from ground surface to the point where the ratio of additional stress to self-weight reaches 10%, is the thickness of compressive soil. The width of the influencing zone of the additional stress in the foundation is 46.6 m at the section DK67+630, and 40 m at the section DK67+666; the depth of the influencing zone is 20.4 m at the section DK67+630, and 20.1 m at the section DK67+666. According to the centrifuge model box size and boundary conditions, the test model scale n was determined to be 60, and the model size was converted from the prototype size by model scale n, as shown in fig 1.
Soil for test was collected from the construction site. After drying, crushing, and screening through 2 mm sieve, the foundation soil was filled into the model box layer by layer with moisture content and bulk density as control factors.

Embankment filling was layered in the model box and tamped in accordance with the optimum moisture content and the compaction coefficient 0.92. When the designated height was reached, the compacted foundation soil was molded by cutting technology. The track static loads and trains loads were replaced equivalently by steel plate, and the steel plate size was calculated according to the simulation scale.

Dynamic compaction soil, according to the relationship between its density and depth in [4-6], was also treated and filled layer by layer.

In construction site, cement-mixed piles were placed in square layout. The pile length is 4 m, diameter is 0.5 m, and pile spacing is 1 m. There is a 60 cm thick gravel cushion at the bottom of embankment. From top to bottom, the structure of the cushion is 15 cm thick cushion, geogrid, 30 cm thick cushion, geogrid, and 15 cm thick cushion. The piles were simulated by a mixture of P.O425 cement, water, and sand, and the mix ratio was determined by the strength of the mixture. In order to accelerate the solidification of piles, cement accelerator was used. A modified drill with a diameter of 8 mm and a length of 67 mm was used to drill holes in the model for laying out the piles. After drilling was completed, the mixture was injected into the holes. When the pile strength reached 80% of the design strength, it was time to fill the embankment. Gravel cushion was simulated using plastic gauze and fine sand. The sand was screened through a 0.6mm sieve. The strength of the plastic gauze was tested to meet the requirement that the geogrid design strength is no less than 80 kN/m.

2.2. Test scheme

The test purpose is to simulate the main consolidation of the foundation, and study the relationship between soil settlement and the time. Four groups of centrifuge model tests were carried out. Among them, models I and II were to simulate the untreated DK67+630 section, model III was to simulate the DK67+630 section which was treated by dynamic compaction,
and model IV was to simulate the DK67+666 section which was reinforced with cement-mixed piles. Model I was designed to test the pressure between embankment and foundation, and foundation settlement; models II, III, and IV were designed to test the laws of the foundation settlement.

By similarity theory\(^7\), the time scale for model was chosen: \(n^2=3 \times 10^6\). The model was loaded in three phases (i.e., the self-consolidation of foundation soil, the embankment construction, and long-term use of embankment) to simulate the characteristics of subgrade settlement and the pressure between embankment and foundation.

2.3. Test equipment

Test equipment is the TLJ-2 geotechnical centrifuge of Southwest Jiaotong University. Its main performance indicators are as follows: the maximum capacity is 100gt; the small model box size is 0.6 m×0.4 m×0.4 m, and the large model box size is 0.8 m×0.6 m×0.6 m; the centrifuge radius is 3.0 m, and the effective radius is 2.7 m; the maximum acceleration is 200g; the number of data acquisition channels is 70; the oil joint pressure is 10 MPa, and the oil flow is 10 L/min, supplied through 2 channels; the water connector pressure is 1 MPa, and the water flow is 10 L/min, supplied through 1 channel; the DC motor power is 185 kW; the gas connector pressure is 1 MPa, and the gas flow is 10 L/min, supplied through 2 channels; cameras and photographic equipment is equipped.

III. TEST RESULTS AND ANALYSIS

3.1. Main test results

The relationships between the ground settlement and time of the four models are shown in fig 2 to fig 8. In these figures, the time \(t\) is the one converted by model time scale \(n^2 (n^2=3600)\), \(s\) is settlement, \(h\) is filling height, and \(R\) is correlation coefficient. The settlement of the four models are provided in table 2.

<table>
<thead>
<tr>
<th>Time</th>
<th>Settlement /mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model I Model II Model III Model IV</td>
<td></td>
</tr>
<tr>
<td>Construction and holding period</td>
<td>135.4 158.6 116.8 129.4</td>
</tr>
<tr>
<td>Post-construction (3 year)</td>
<td>45.0 48.0 37.7 32.5</td>
</tr>
</tbody>
</table>

![Table 2. Settlement of ground]

Fig 2. Relationship between filling height, settlement and time of model I in the filling period
Fig 3. Relationship between total settlement and time of model I

Fig 4. Relationship of filling height, settlement and time of models II and III in the filling period

Fig 5. Relationship between total settlement and time of model II

Fig 6. Relationship between total settlement and time of model III

Fig 7. Relationship of filling height, settlement and time of model IV in the filling period
The tested pressure between embankment and foundation of model I was shown in table 3. For comparison, the pressure calculated by other two commonly used methods are also listed in table 3.

<table>
<thead>
<tr>
<th>Filling height</th>
<th>Pressure/ kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.2 m</td>
</tr>
<tr>
<td>γh method</td>
<td>24</td>
</tr>
<tr>
<td>Average method</td>
<td>22</td>
</tr>
<tr>
<td>Tested values</td>
<td>28</td>
</tr>
</tbody>
</table>

3.2. Analysis of test results

3.2.1. Ground settlement characteristics during filling and holding period

From figs 2, 4, and 7, one can see that the test soil was in the elastic state at the beginning of loading. As the load increases, the ground settlement increases approximately linearly. As the load continues to increase, the soil gradually enters the elastic-plastic state. In this process, the settlement rate keeps increasing until the load does not increase any more. In the holding period, because soil consolidation has not completed yet, the rheology and settlement continues increasing with time, with an ever smaller rate, until stabilized.

For models I to IV, the ratio of the settlement during the filling period to the total settlement during the filling and holding period was 82.9%, 87.5%, 88.3%, and 89.9%, respectively. At the end of the holding period, foundation settlement tends to be stable. For the completely decomposed granite foundation soil in Hainan, the settlement was nearly stable one year after filling of the soil.

3.2.2. Post-construction settlement characteristics

The relationship between total settlement and time for models 1 to 4 is shown in figs 3, 5, 6, and 8, respectively. The latter half of the curves are the relationship between settlement and time after the embankment was loaded by track and train. The post-construction settlement of model I to IV was 45.0, 48.0, 37.7, and 32.5mm, respectively. The post-construction settlement rate, as with the filling period, was larger in the initial stage. Settlement tended to stabilize in three years. After being loaded for 200 days, 67.9%, 64.0%, 71.5%, and 82.7% of the post-settlement finished in models 1 to 4, respectively, and the remaining settlement was 14.5, 17.3, 10.7, and 5.6 mm, respectively. By comparison, the post-construction settlement value and velocity of models I and II are basically the same, the post-construction settlement of model III
was 78.5% of that of model II, and the settlement of model IV developed most rapidly.

According to “Tentative standard for new building special railway line for passenger of 200-250 km/h”, the post-construction settlement and settlement velocity of the four models all meet the requirements.

3.2.3. Characteristics of the total settlement

Field tests show that the settlement accelerates in the early period, then slows down, and ultimately tends to a limit. Thus, the law of soil consolidation and settlement can be described by a S-curve. For fitting the relationship between settlement and time, Gomperte model, Pearl model, and Weibull model are frequently used. Fitting the relationship between settlement and time of the four centrifugal test models using the three methods indicates that the fitting correlation coefficient obtained by Weibull model is the greatest (above 0.99), so Weibull model is the best among the three fitting method. Weibull model is shown in formula (1), and fitting parameters of the four centrifuge models are shown in table 4.

\[ s = L - a e^{-bt} \]  

Where \( L, a, b, \) and \( r \) are fitting parameters.

<table>
<thead>
<tr>
<th>Model</th>
<th>L</th>
<th>( a )</th>
<th>b</th>
<th>( r )</th>
<th>Correlation coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>189.85</td>
<td>189.85</td>
<td>0.00877</td>
<td>0.98307</td>
<td>0.9942</td>
</tr>
<tr>
<td>II</td>
<td>205.46</td>
<td>199.44</td>
<td>0.0229</td>
<td>0.82375</td>
<td>0.9958</td>
</tr>
<tr>
<td>III</td>
<td>161.41</td>
<td>150.63</td>
<td>0.02559</td>
<td>0.78877</td>
<td>0.9962</td>
</tr>
<tr>
<td>IV</td>
<td>160.44</td>
<td>145.96</td>
<td>0.01435</td>
<td>0.96313</td>
<td>0.9976</td>
</tr>
</tbody>
</table>

3.2.4. Impact of filling simulation on settlement

Embarkment in model I was filled layer by layer. There were four layers in all. After each layer was filled, the centrifuge accelerated to 60g. Embarkment in models II and III was filled once to the design height. We simulated the stratified filling in the test through a gradual increasing in acceleration. From fig 2 and fig 4, one can see that the settlement during the filling period of model I is 23.2 mm less than that of model II, and the post-construction settlement is also 3 mm less. Compared with the field-measured settlement, the result obtained by filling simulation of model I is more reasonable.

3.2.5. Settlement comparison for different treatments

As described before, models II and III simulates the same section DK67+630; and the
difference is that the foundation in model II was not treated, but that in model III was dynamically compacted. During the filling period (fig 4), the settlement of the non-treated foundation was 158 mm, while that of the dynamic compaction foundation was 116 mm. Compared with the settlement of the non-treated foundation, the settlement of the dynamic compaction foundation was reduced by 26.6% during the filling and holding period, and was reduced by 21.5% during the post-construction period (see fig 6).

The embankment in model IV was 3.99 m high (fig 7). During 80 days after filling, the ground settlement in model IV was 116.6 mm, and the average settlement rate was 1.46 mm/d. The embankment in model III was 5 m high (fig 4). During 100 days after filling, the ground settlement in model III was 104.4 mm, and the average settlement rate was 1.04 mm/d. Compared with that of the dynamic compaction foundation, the settlement rate of cement-mixed piles was increased by about 40% during the filling period. For the settlement value at the end of holding period, model IV was 10.8% larger than model III. The post-construction settlement was 37.7 mm in model III, and 32.5 mm in model IV. In terms of the post-construction settlement per meter embankment height, model IV was 8% larger than model III. This indicates that dynamic compaction is a better treatment than cement-mixed piles for deep completely decomposed granite ground.

3.2.6. Pressure between embankment and foundation

Accurately determining the pressure between embankment and foundation is the prerequisite to calculate the additional stress in soil, and is also a crucial step for calculation of embankment settlement. In the calculation, the shear stress between embankment and foundation was often not considered [9]. The stress values and stress distribution in foundation, however, are obviously different to the calculated values under the usually assumption [10-11]. One can see from table 4 that the tested pressure values are 4% to 23% less than the calculated values obtained by γh method, but 12% to 27% larger than the calculated values obtained by average method. The higher the embankment is, the larger the difference is.

IV. CONCLUSIONS

(1) The centrifugal model tests show that after filling embankment on the decomposed granite soil in Hainan, the settlement rate was rapid in the early period: the settlement during the filling period was 87.2% of the total settlement before laying tracks, and was 67% of the total settlement after the embankment was loaded by tracks and train. After tracks were laid for 1 year, 95% of consolidation was reached. After tracks were laid for 3 years, the settlement stabilized completely.

(2) The relationship between settlement and time of all models can be appropriately fitted by Weibull model. The fitting correlation coefficients are greater than 0.99.

(3) For the settlement control of deep completely decomposed granite soil, dynamic compaction is more effective than the cement-mixed piles.
(4) Different filling simulation resulted in different test results. Comparison of the results obtained from the centrifugal model tests against the field-measured settlement show that the filling simulation of model I was more reasonable.

(5) The stress values and stress distribution in foundation are obviously different from the calculated values under the usually assumption. The higher the embankment is, the larger the difference is.

Note that differences between the remolded soil and the on-site soil, inhomogeneity of the centrifugal field, acceleration and deceleration history, etc. all can affect the test results. Taking into account these factors in the centrifugal model tests still need further study in our future work.

References


EXTENDED THE SINGLE MODEL STUDYING THE TRAJECTORY OF TRACTOR-SEMI TRAILER

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Summary: The paper focuses on the extended single model to study the trajectory of tractor-semitrailer. By building the motion equations of articulated vehicle and the survey might consider allowing the variation of kinetic and dynamics parameters of the vehicle to develop stable motion control research.

The movement safety will be mentioned by studying in various braking conditions of semi-trailer. Some achieved results can show the preliminary review of the variation of dynamics parameters related to the exploiting field of articulated vehicles. This paper uses the Matlab-Simulink software to solve the differential equations and define the trajectory of a truck tractor and semitrailer.

Keywords: Tractor - semitrailer; Vehicle trajectory; Vehicle dynamic model; Matlab-Simulink.

I. INTRODUCTION

It is necessary to take interest in studying the traffic safety of vehicle because of the increase in using many heavy vehicles such as articulated vehicles (tractor-trailer or tractor-semitrailer) which is promoting some theoretical and experimental investigations to that vehicle.

Applying the automatic control system to optimize the dynamics control is being investigated widely for much kind of vehicles. It is quite complicated to study the articulated vehicle dynamics while considering the full vehicle model with many changeable parameters.

For the dynamics of motion, a problem with single-trace model is relatively simple and considered in a limited impact parameters. In a recent article, the author was referring to trajectory movement determination of semi-tractor trailer with single-trace model, determination of kinetic and dynamics parameters of the vehicle. The extended model studies towards the full model to expand the survey and assess the variability of the parameters of the vehicle dynamics.

Within the framework of this article, the author mentions to the extension of single-trace model of articulated vehicle which considers the remarkable effect of the lateral and longitudinal parameters placed on the wheels.
II. TRACTOR-SEMITRAILER DYNAMICS MODEL

The dynamics model of tractor-semi-trailer with appropriate assumptions:
- Articulated Vehicle moves on a smooth and flat road,
- Tire side slip angles of front steering wheel are the same,
- Tractor and semitrailer body is absolutely rigid,
- Only the variability of kinematic and dynamic parameters on ground plane is considered.
- The atmosphere resistance is neglected,
- The deformation of tire is linear and the elastic moment is neglected,
- Joint is assumed as an ideal knuckle, with no interstice and no frictional joint moment.

Joint is located slightly ahead of the center of the tractor rear axle.

The space model of tractor vehicle and semitrailer shows on the fig 2 and fig 3.
Using the Dalambert rulers to setup the equations of motion with respect to the axles fixed to the tractor and semitrailer body. After reducing those equations, the set of differential equations is shown:

\[
\dot{V}_1 = \frac{1}{m_1} \left( (F_{xFL} + F_{xFR}) \cos \beta - (F_{yFL} + F_{yFR}) \sin \beta + F_{xRL} + F_{xRR} + F_{xK} \cos \alpha_1 + \right)
\]

\[
\dot{\alpha}_1 = \frac{1}{V_1 m_1} \left( -((F_{xFL} + F_{xFR}) \cos \beta - (F_{yFL} + F_{yFR}) \sin \beta + F_{xRL} + F_{xRR} + F_{xK} \sin \alpha_1 + \right)
\]

\[
\ddot{\epsilon}_1 = \frac{1}{J_{Z1}} \left( \frac{(F_{xFL} + F_{xFR}) \sin \beta + (F_{yFL} + F_{yFR}) \cos \beta \cos \phi_1 +}{(F_{xFR} - F_{xFL}) \cos \beta + (F_{yFL} - F_{yFR}) \sin \beta \sin \phi_1 + } \right)
\]

\[
\ddot{\phi}_1 = \frac{1}{J_{Z2}} \left( \frac{- (F_{xMR} - F_{xML}) \sin \phi + (F_{yML} + F_{yMR}) \sin \phi_1 +}{(F_{yK} \cos \phi + F_{xK} \sin \phi) \sin \phi_2 + } \right) + \ddot{\epsilon}_2 = 0
\]

Where:

- The dimension parameters of tractor and semitrailer.

\(\beta\): Front wheel steer angle of tractor; \(\alpha_1\): Tractor body side slip angle; \(\alpha_2\): Semi-trailer body side slip angle; \(\epsilon_1\): Tractor body yaw angle; \(\phi\): Semi-trailer rotational angle with respect to a tractor; \(V_1\): The velocity of the tractor in the center of gravity; \(V_2\): The velocity of the semi-trailer in the center of gravity.

- And longitudinal forces, lateral forces, inertial forces, yawing moment of tractor and semitrailer.

On moving, the tractor and semitrailer could roll with the roll axle which placed on each center axles.

![Fig 4. The roll angle of tractor and semitrailer](image)

In studying model, the roll angle of tractor and semitrailer are small so that it will be
defined:

\[ \psi_1 = \frac{m_1 h_1}{C_{gf} + C_{gr} - m_1 g h_1} a_{y1} \]

\[ \psi_2 = \frac{m_2 h_2}{C_{gM} - m_2 g h_2} a_{y2} \]

Where: Where relating to the stiffness of axles and later acclerations of tractor and semitrailer.

- \( C_{gf}, C_{gr}, C_{gM} \): The stiffness of tractor axles and semitrailer; - \( a_{y1}, a_{y2} \): The later acclerations of tractor and semitrailer.

- And other parameters show on the fig 4.

During moving, inertia forces will redistribute the load placed on the wheels of the vehicle. In figure 5 and 6 shows the main components loading on semi-trailers and tractors. To simplify the problem ignores the influence of wind resistance.

The normal loads on various axles can be expressed by the following.

**Fig 5. Forces acting on a semitrailer during moving**

The vertical load on the fifth wheel:

\[ F_{ek} = \frac{1}{2} \left( Z_k^T - \frac{m_1 h_2}{L_2} a_{x2} \right) \]

The vertical load on each wheels of semitrailer:

\[ F_{AML} = \frac{1}{2} \left( Z_k^T + \frac{m_1 h_2}{L_2} a_{x2} \right) - \frac{1}{b_{sl}} (m_{sl} a_{x2} h_{sl} + m_{sl} g h_{sl} - C_{sl} \psi_2) \]

\[ F_{AMR} = \frac{1}{2} \left( Z_k^T + \frac{m_1 h_2}{L_2} a_{x2} \right) + \frac{1}{b_{sl}} (m_{sl} a_{x2} h_{sl} + m_{sl} g h_{sl} - C_{sl} \psi_2) \]

**Fig 6. Forces acting on a truck tractor during moving**
The vertical load on each front wheels of tractor:

\[ F_{FL} = \frac{1}{2} \left( \frac{m_y g L_{f0} + F_{sk} (L_{f1} - L_k)}{L_1} - m_i h_1 a_{x_i} \right) - \frac{1}{b_f} \left( m_y a_{y_i} h_f + m_y g \psi_i h_f - C_{gy} \psi_i \right) \]

\[ F_{FR} = \frac{1}{2} \left( \frac{m_y g L_{f0} + F_{sk} (L_{f1} - L_k)}{L_1} - m_i h_1 a_{x_i} \right) + \frac{1}{b_f} \left( m_y a_{y_i} h_f + m_y g \psi_i h_f - C_{gy} \psi_i \right) \]

On each rear wheels:

\[ F_{RL} = \frac{1}{2} \left( \frac{m_y g L_{r0} + F_{sk} (L_{r1} + L_k)}{L_1} + m_i h_1 a_{x_i} \right) - \frac{1}{b_r} \left( m_y a_{y_i} h_r + m_y g \psi_i h_r - C_{gr} \psi_i \right) \]

\[ F_{RR} = \frac{1}{2} \left( \frac{m_y g L_{r0} + F_{sk} (L_{r1} + L_k)}{L_1} + m_i h_1 a_{x_i} \right) + \frac{1}{b_r} \left( m_y a_{y_i} h_r + m_y g \psi_i h_r - C_{gr} \psi_i \right) \]

On the other hand, when no lateral sliding takes place on the contact patch, the relationship between the cornering force and the slip angle of each wheels is expressed by:

\[ F_{yi} = 2C_{ai} \alpha_i \]

Where:
- \( C_{ai} \): the cornering stiffness of each wheels
- \( \alpha_i \): the slip angle of each wheels.

III. THE TRAJECTORY AND SWEPT PATH WIDTH OF TRACTOR-SEMITRAILER

During motional process, the center of gravity of tractor always changes and it is determined from instantaneous velocity \( V_1 \) making tangent line to curve of orbit considered in the fixed inertial system.

The motional trajectory of tractor-semitrailer is defined by 3 following points: the center of gravity of tractor, joint and center of gravity of semi-trailer.

The center of gravity location of tractor:

\[ X_{T1} = \Delta X_{to} = \int_{\alpha_i}^{\alpha_i + \psi_1} V_{1x} dt = \int_{\alpha_i}^{\alpha_i + \psi_1} V_1 \cos(\alpha_i + \varepsilon_1) dt \]

\[ Y_{T1} = \Delta Y_{to} = \int_{\alpha_i}^{\alpha_i + \psi_1} V_{1y} dt = \int_{\alpha_i}^{\alpha_i + \psi_1} V_1 \sin(\alpha_i + \varepsilon_1) dt \]

Joint location \( K \):

\[ X_K = X_{T1} - (L_{ef} - L_{ti}) \cos(\varepsilon_i) \]

\[ Y_K = Y_{T1} - (L_{ef} - L_{ti}) \sin(\varepsilon_i) \]

The center of gravity location of semi-trailer:
The swept path width of tractor-semitrailer during motional process is determined by the location of border rim point in proportion of limited dimension of tractor-semitrailer.

**Fig 7. Swept path width of tractor-semitrailer with the limited dimensions**

The location of these points described in that figure is determined:

\[
\begin{align*}
X_A &= X \tau t + T_1 A \cos(\epsilon_i + \theta) \\
Y_A &= Y \tau t + T_1 A \sin(\epsilon_i + \theta) \\
X_B &= X \tau t + T_1 B \cos(\epsilon_i - \theta) \\
Y_B &= Y \tau t + T_1 B \sin(\epsilon_i - \theta) \\
X_C &= X \tau t - T_1 C \cos(\epsilon_i + \theta) \\
Y_C &= Y \tau t - T_1 C \sin(\epsilon_i + \theta) \\
X_D &= X \tau t - T_1 D \cos(\epsilon_i - \theta) \\
Y_D &= Y \tau t - T_1 D \sin(\epsilon_i - \theta) \\
X_E &= X \tau t - K E \cos(\epsilon_i - \varphi - \theta_m) \\
Y_E &= Y \tau t - K E \sin(\epsilon_i - \varphi - \theta_m) \\
X_f &= X \tau t - K F \cos(\epsilon_i - \varphi - \theta_m) \\
Y_f &= Y \tau t - K F \sin(\epsilon_i - \varphi - \theta_m)
\end{align*}
\]

IV. STUDY THE TRAJECTORY OF TRACTOR-SEMITRAILER IN BRAKING REGULATIONS

4.1. Exciting functions

- The wheel turn angle function: The rule of wheel steering angle is shown in fig 8. With in two intial second, the wheel steering angle increases and reaches the maximum of 0.14rad (8°).

**Fig 8. Variation of wheel turn angle with time**
- The longitudinally force function: The longitudinally forces acting on each wheels of tractor and semitrailer are considered in differential conditions with some assumptions as following:

. In two first second, the vehicle moves stably and the total of tractive force acting on driven wheel of tractor equal to the total rolling resistance force of other wheels.

. To act braking force on each axle after 2s and the last wheel will be braked after 2.5s.

The maximum of braking force is equal to the maximum value $\mu p W$ ($\mu_p$ is the peak value of the coefficient of road adhesion and $W$ is the normal load).

4.2. The results

a. The first case

When braking, the braking force arising from wheels at different axles may variously alter but it all followes common rules. To carry out survey method, the braking force is controlled at any axle during cornering, and suppose that the brake system takes late effect at every axle. The order of controlling braking force can be seen in the fig 9 with the period of late effect between axles is 0.1s.

![Fig 9. The variation of the longitudinally forces acting on each axle with time](image)

After simulating, some results are shown in fig 10 to 13.

In the linear model, the yaw rate of tractor reaches the maximum value of approximately 0.45 (rad/s) and that of semi-trailer is approximately 0.26 (rad/s), as can be seen from fig 10.

![Fig 10. The variation of tractor and semitrailer yaw rate with time](image)

It can be seen from fig 11 that the lateral acceleration of the tractor and semitrailer predicted using the linear plane model. Based on the data shown in fig 11, it shows that the maximum of both tractor and semitrailer lateral acceleration is below 0.35g.
Fig 11. The variation of tractor and semitrailer lateral acceleration with time

Fig 12 shows the variation of articulation angle. After braking, the articulation angle increases steadily and reaches the maximum of 0.47 rad (27°) at the moment of 2s. It should be noted from the figures that the articulation angle increases and gets the big value because of braking forces acting on the different axles of tractor and semitrailer.

Fig 12. The variation of articulation angle between tractor and semitrailer with time

Fig. 13 expresses the points’ trajectory as following: the centre of gravity of tractor; Joint K and the centre of gravity of semitrailer.

- During one initial second to drive transitionally, (in the first 5 kilometers), trajectory of tractor-semitrailer is changed little.

- Finish driving transitionally, to maintain the steering wheel angle and act braking force, trajectory of tractor-semitrailer continues to change largely and it will stop after its braking time.

b. In second case

During braking, assuming that a certain wheels of the vehicle or wheels of the axle lost brakes. Here, assuming the front wheels of tractors lost brakes. Results achieved may consider
and evaluate the changes of kinetic and dynamics parameters of the vehicle.

**Fig 14. The variation of the longitudinally forces acting on each axle with time**

Some results can be shown in the fig 15 to fig 18.

**Fig 15. The variation of tractor and semitrailer yaw rate with time**

**Fig 16. The variation of tractor and semitrailer lateral acceleration with time**

**Fig 17. The variation of articulation angle between tractor and semitrailer with time**

**Fig 18. The trajectory of tractor and semitrailer**
V. CONCLUSIONS

Following these results, it can be noted that during braking control, the slow action of the braking system has certain influence to the orbital motion as well as variations in the acceleration of the vehicle. In some of the results of the author’s value the maximum lateral acceleration achieved in the linear model is less than 0.3g. In this paper, the acceleration values are within small limits. Consider special cases, the loss of braking could occur in many different states. Status are reviewed in the article is the loss of braking occurs in front of tractors. In this case, the acceleration of both cars and semi tractor trailers were significantly increased and relatively large value (in tractors is > 0.35gm/s²). Relative angle between tractors and semitrailers have relatively large values and fluctuations (approximately 0.48 rad or 280) as is evident in fig 18. Increased braking distances compared to plan a break in the case. This is also the factor of concern in terms of exploitation and use of union cars pulling trailers sale.

Due to the complexity of the dynamics of articulated vehicles, the trailer rules of the parameters are not clear, otherwise the problem is not addressed to the nonlinearity of the model. This is the direction of research, development model at higher level.

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INTEGRATION OF PRODUCTION PROCESS, LOGISTICS AND TRANSPORT - A THEORETICAL FRAMEWORK

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Summary: The purpose of this paper is a presentation of a state-of-the-art picture about the integration of production process, logistics and transport and its application. First of all, it reviews some key definitions about logistics, transport and production process; simultaneously this part provides the literature review on the integrative process and its current application level. The next section presents the current state-of-the-art picture about the integration between production and logistics, logistics and transport, transport and production. The contents of the second section are very important to give the initial proposal for a theoretical framework of integration of production process, logistics and transport which will be highlighted in the last section.

Key words: Integration, logistics, production, transport.

I. INTRODUCTION

In business environment today, the integration is one of the dominant themes in the development of logistics management. Most of enterprises, when supplying products or services that customers need, often try to pay much attention on the material flow, how to design and pack products, how to transport and store products. In other words, the present fierce competition in the global market, the introduction of products with the short life cycle, and high expectation of customers have forced business enterprises to pay attention to their supply chain. Therefore, enterprises often aim at building the effective and sustainable supply chain by integrated the production processes, logistics and transport. However, it cannot be denied that production, logistics as well as transport are facing many difficult problems. Because, on the one hand, freight carriers are usually expected to provide higher levels of service with the framework of Just-In-Time transport system with low costs whereas transport and traffic seem to reach their capacity limits. On the other hand, traffic and transport related decisions of the public authorities have to be considered. These are influenced by the similar factors and
have to mind the development of production and logistics. Therefore it is very necessary to find out integration model of production process, logistics, and transport to give feasible solution for sustainable development of enterprises in the future. The most important step to build the integrated model is to give a theoretical framework of the three factors: production process, logistics and transport.

II. SOME KEY DEFINITIONS

- **Logistics**

  A literature review in the field of logistics shows that the term logistics is presented in different ways. All of them try to express the same content that is one process including series of continuous, related and interactive activities. They are performed in a scientific and systematic manner. Logistics is also a process covering all factors that make product from the stage of input to final consumption.

- **Production**

  According to Business Dictionary, production can be defined as a processes and method employed in transforming tangible inputs (raw materials, semi-finished goods, or subassemblies) and intangible inputs (ideas, information, knowhow) into goods or services. Looking in the nature of production process shows that production refers to the economic process of converting from inputs into outputs.

- **Transport**

  As defined by Prof. Boltze (2006), transport can be understood as the spatial change of people, goods, service, information and energy. According to Gust Blauwens, Peter De Baere, Eddy Van de Voorde (Transport Economics, 2002) transport system can be generally regarded as an economic system that comprises three main ingredients, namely, the mean of transfer, the infrastructure and the load.

- **Integration and its levels of application**

  Integration process within and between organizations is one of the most fundamental trends on business these days (Attila Chikan, 2001). The basic concept of integration has been used widely to study various different organizational phenomena.

  In the boundary of organization, Mark Pagell (2004) defined integration as a process of interaction and collaboration in which manufacturing, purchasing and logistics work together in a cooperative manner arrive at mutually acceptable outcomes for their organization. Expanding out of the boundary of organization, the integration concept has been not mentioned many in recent studies. In its essence of the entire concept of supply chain management, the integration considers as a key business processes from end user through original suppliers that provides products, services, and information that add value for customers and other stakeholders (Lambert, Cooper and Pagh, 1998). Further expansion within an urban city or a region, the
integration concept becomes more complicated. When considering the integration process of transport and logistics within urban areas, Taniguchi (2007) defined as the process for totally optimizing the logistics and transport activities by private companies while considering the traffic environment, the traffic congestion and energy consumption within the framework of a market economy.

There are different ways to classify integration. The reviewed literature shows that integration has been generally studied at two different analysis levels. Internal integration examines integration across various parts of single organization. It seeks to eliminate the traditional functional “single approaches” and emphasize better coordination among functional areas (Cristina Gimenez, Eva Ventura, 2006). External integration, on the other hand, examines integration occurring among organizations. It is also considered as the integration of logistics activities across firm boundaries (Stock, Greis and Kasarda, 1998).

III. THE STATE OF THE ART OF THE INTEGRATION BETWEEN PRODUCTION AND LOGISTICS; TRANSPORT AND LOGISTICS; TRANSPORT AND PRODUCTION

A literature review in the field shows that there are a number of studies related to the integration between logistics and production, logistics and transport, transport and production. So far, there has not been any specific research on the integration of production process, logistics and transport. The aim of this part is providing a thorough review of the literature about the integration of each pair such as logistics and production; transport and logistics; production and transport before considering the integration of three factors is production process, logistics, and transport.

- The integration of production process and logistics

Basically, this relationship is considered within the scope of an organization. According to Attila Chikan (2001) before finding the integrated mechanism, it is necessary to point out the common and conflicting features between production and logistics. Cristina Gimenez and Eva Ventura (2006) have a bit different approach when considering the issue of the relationship between production and logistics. These authors analyze this relationship by considering the interaction between logistics and production. This interaction is examined by showing the duties of production and logistics functions, simultaneously indicating the intersection of two functions. The duties of production and logistics and the intersection of these two functions are as follows:

<table>
<thead>
<tr>
<th>Production</th>
<th>Intersection of activities</th>
<th>Logistics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plan’s activity</td>
<td>Product planning</td>
<td>Transport</td>
</tr>
<tr>
<td>Material manipulation</td>
<td>Plan location</td>
<td>Inventory</td>
</tr>
<tr>
<td>Quality control</td>
<td>Purchasing</td>
<td>management</td>
</tr>
<tr>
<td>Maintenance</td>
<td>Warehousing</td>
<td></td>
</tr>
</tbody>
</table>

No matter which approach is applied, the review of this impact process will be started by analyzing the common and conflicting features between logistics and production functions. Also according to Attila Chikan (2001), the common features between production and logistics are as follows:

- Both logistics and production focus on the “real” sphere. Both of them have activities which belong to or lead to the monetary and the information sphere of company operation.
- Both functions are often translated to actual day to day operation on the material sphere.
- Both have a fairly well - measurable contribution to the profitability of the company.
- For fulfilling their tasks, both functions must have a short-term feedback (and off-course a long-term one, as well).

Mentioned to the conflicting features, there are a lot of conflicting points of two functions which need to be identified when trying to establish an integrated connection. These conflicts stem from the nature of these functions and under the normal condition they are self-interest seeking of the parties involved. The most important conflicting features are the following:

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Production</th>
<th>Logistics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Add value to product</td>
<td>Add use-value</td>
<td>Add place and time value</td>
</tr>
<tr>
<td>Activities</td>
<td>Focus on operation which usually means manipulating the items at given place</td>
<td>Focus on moving items towards further steps of transformation or to the final customer</td>
</tr>
<tr>
<td>Evaluating the effectiveness of function</td>
<td>Usually advances quality</td>
<td>Emphasizes cost (or prices)</td>
</tr>
<tr>
<td>Attention in turns</td>
<td>Technology</td>
<td>Oriented more to the products it has to handle</td>
</tr>
<tr>
<td>Sub-function</td>
<td>Concentrated in the organization</td>
<td>Spread over</td>
</tr>
<tr>
<td>Organizational contact to other functions</td>
<td>Marketing</td>
<td>Usually production itself plays the most influencing role</td>
</tr>
</tbody>
</table>

*Table 1. The conflicting features between production and logistics*

Resource: Attila Chikan (2001)

Based on analyzing the common and conflicting features between logistics and production, it is a very important role of integration to provide a framework for handling these conflicts in a way which ensures the most effective connection between the functions, the fastest flow of goods towards the final customer, and fastest return of capital invested.

- **The integration of logistics and transport**

Recently, there are a number of studies on the integration of transport and logistics. Most
of these studies focus on examination of the relationship between transport and logistics through describing how transport sector influenced by logistical principles of distribution and production. However, the authors concede that it is very difficult to determine the actual relationship between logistical structures and logistics. On the other hand, to some extent, transport is perceived as an integrated part of logistical system, which make it difficult to isolate transport as an independent activity (Lise Drewes Nielsen et al, 2003). It means that the relationship between logistics and transport cannot be established straightforwardly. These authors have proposed the intermediate categories in an attempt to “translate” logistical principle into transport related concept:

- Transport distance - how far.
- Speed of transport - for how long.
- Frequency of transport - how often.
- Point in time of transport - when.

These above key indicators are considered as “intermediate indicators” and called the key indicators of transport logistics. Obviously, a change in logistics conditions caused by new production philosophy, a company merging or some new infrastructure will have impacts on the organization of transport logistics. This will be shown by a change of the key indicators of transport logistics: distance, speed, frequency and point of time. These indicators of transport logistics will have an influence on transport, measured by the indicators of transport. Lise Drewes Nielsen et al (2003) have proposed the one-way cause-and-effect relationship between the change of logistics organization, transport logistics, transport, and environment.
This is only an one-way relationship, so it can not reflect the full description of reality. This modeling, however, is considered as a basis foundation for analyzing the integration of logistics and transport, whereas in fact, the feedback mechanisms are often much slower than the cause-and-effect relationship shown in the above figure.

In order to examine the impact of change in logistical organization on transport, it is necessary to clarify the contents of each change in the above model. In fact, the decision of logistics can come from the corporate management’s decision to open or close down the production facilities, to the truck driver deciding which way to drive his lorry (Lise Drewes Nielsen et al, 2003). Based on that, McKinnon (1998) proposed the different level of decision within the field of logistics, namely:

- Logistics structure: Meaning that numbers, locations and capacity of factories, warehouses and terminals

- Pattern of trading links: made by commercial decisions on sourcing, sub-contracting and distribution, and manifest as freight network linking premises of company to those of its trading patterns.

- Scheduling of product flow: The programming of production and distribution operations translate trading relationship into separated goods flows.

- Management of transport: defined by the decision at the previous three levels.

• **The integration of production and transport**

In the theory of production economics, transport is a process of production as well as being a factor input in the production function of firms. Transport is produced from various services and is used in conjunction with other inputs to produce goods and services for enterprises. Transport is an intermediate good and therefore can be considered as a "derived demand".

More broadly, one has transport as an input into a production process. For example, the Gross National Product (GNP) of the economy is a measure of output and is produced with capital, labor, energy, materials and transportation as inputs. GNP = f (K, L, E, M, T) in which K means capital; L means labor; E means energy; M means material; T means transport.

In its essence of the concept of supply chain management, the coordination of production and transport is the key factor to ensure the smooth activity of supply chain. This coordination often focuses on the strategic and tactical planning or scheduling (Chen, 2004). Examining the contents of production planning and transportation planning problems can find some aspects of coordination between production and transport. Production planning in specific, on lot-size
models, considered the tradeoff between the inventory caring cost and setup costs, subject to capacity restriction of resources, whereas, the transportation planning studies the matters from the basis travelling salesman problem to the multi-vehicle pickup and delivery problem with time windows (Kadir Ertogal et al, 1998). Studying the main tradeoff between production and transportation decisions will give us the clear picture of combination of these two factors.

IV. PROPOSAL OF A THEORETICAL FRAMEWORK OF INTEGRATION OF PRODUCTION PROCESS, LOGISTICS AND TRANSPORT

Based on the analysis of the state of the art picture of the integration between production and logistics; transport and logistics; transport and production, the integration of production process, logistics and transport is proposed as below:

Theoretical, the integration of production process, logistics and transport can be shown in the following aspects: The change in condition of production will lead to the change of numbers, locations and capacity of factories, warehouses and terminals. This change will force it to consider the factors related to logistics such as delivery time, delivery speed, and delivery reliability in order to optimize the process of goods movement from the origin as suppliers to final customers. Among them, delivery time is considered as a variable describing demands to deliver goods at a designed period of time or time windows; delivery speed explains the requirement for (quicker) action in the supply and demand chain; delivery reliability is the variable expressing the ratio of the number of deliveries made without any error (regarding time, place, price, quantity or quality) to the total number of deliveries. All these factors are carried out through the support of transport shown by indicators such as transport mode,
transport route, transport efficiency. Transport mode is variable expressing the requirement for choosing the right transport mode for the deliveries. Transport route is the variable describing demand to make a route choice with time windows.

V. CONCLUSIONS

This paper has presented the contents of the state of the art of the integration of production process, logistics and transport. In the field of production process, transport is considered as the input factor in the production function of firms, cities, states and the country. Within the logistical field transport is often understood as an integrated part of the supply and demand chains. The integration of production process, logistics and transport, theoretically, can be understood as a process of interaction and collaboration in which manufacturing, freight carriers and logistics work together in a cooperative manner to arrive at mutually acceptable outcomes for their organization. The presented framework is still in its early stages of development. It is necessary to have further research to be done to evaluate the framework and to generalize the findings.

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DESIGN OF THE MINIMUM CURVE RADIUS OF HAINAN EAST LOOP LINE

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Summary: The minimum curve radius is one of the most important factors in railway route design. The minimum curve radius of Hainan east loop line, a newly built high-speed passenger dedicated line from Haikou to Sanya, Hainan, China, was determined through comparative analysis of the design codes both in China and German railways. The result shows that the design parameters of Hainan east loop line in terms of the minimum curve radius determined by China’s Temporary Regulations conforms to the ones by German code, and even preserve some capacity for speed raising up to 300 km/h in the long term. Additionally, the related countermeasures for speed raising in this line were proposed on the premise of safety and riding comfort of the trains.

Keyword: The minimum curve radius, Targeted speed, elevation

INTRODUCTION

With the entry of Chinese railway to the high-speed era, trains’ higher operation speed and passengers’ demand for higher riding comfort present a challenge to the design parameters and standards of line plan, line profile, cross-section\(^{[1-2]}\), etc. As one of main technical indexes in design of line plan, line profile, and cross section of the high-speed passenger dedicated line (PDL), the decision of the minimum curve radius directly affects the safety and comfort of the designed line, as well as the economic indexes such as project investment and operation management expenses\(^{[3-4]}\). Because a larger curve radius is difficult to adapt to the changes of ground and easy to go against bypassing the natural obstacles as well as man-built buildings, many earthwork and stonework projects, bridges and tunnels, and removal projects will be created inevitably, thus increasing civil works expenses a lot\(^{[5]}\). Therefore, reasonably choosing the minimum curve radius is important in the whole project budget on the premise of satisfying the safety and comfort of the trains. Conducting further research on the minimum curve radius is of great significance to the design of high-speed PDLs in China.

Hainan east loop line begins from Haikou city in the north to Sanya city in the south of
Hainan province, China, via Wenchang city, Qionghai city, Wanning city, and Lingshui county. It was designed as an inner-city passenger line with a trip of 308 km 14 stations totally, and a design speed of 250 km/h\[^6\]. Along the whole line mainly are plains and hills, so the line plan is mainly affected by the distribution of towns and the scenic spots along the line, public and railway transportation planning of the main cities, airport, express way, rivers along the line, unfavorable geologic conditions, etc. According to the principles of technical feasibility, economic rationality, and safe and reliable operation, to choose a reasonable minimum curve radius can avoid those natural obstacles and buildings well, decrease the above-mentioned earth-stone work and bridge-tunnel projects, thus reducing the whole project expenses, which is especially important in some cities and towns with complicated terrain.

Compared with the large-scale development of Chinese high-speed railway, the developed countries such as Japan, France and German have already come into the high-speed railway age for a long time. Take Germany as an example\[^7\]. In its 90’s of last century, after the great development of high-speed railway, Germany has gradually formed an integrated ICE (Inter City Express) system, in which all the links such as train, catenary, traction power supply, security system, lines (curves, bridges and tunnels), ballast bed, and detection system are connected and matched well. They also established the advanced, mature, economic, and applicable German Railway Design Standard\[^7\]., which was applied to survey and design of 200-300 km/h passenger railway lines. In 2007, the German Railway Designing Standard was used to examine and verify the design parameters of the line plan, line profile, and cross-section of Beijing-Shanghai high-speed railway, and its effectiveness was verified by a dynamic simulation for running trains. The German high-speed railway design specifications could provide a helpful reference for the construction of China's high-speed railway, especially the railway with 250km/h speed. China’s high-speed railway is still in its infancy, and the present design standard is only a temporary one which needs to be improved and perfected during the long term construction and operation, and meanwhile should learn the advanced and applicable techniques and experience from the advanced countries.

In this paper, we will check the calculation of the minimum curve radius of Hainan east loop line, using the German Railway Design Standard. The parameters of the Cologne-Frankfurt passenger line, which has a high similarity to Hainan east loop line, were used as a reference to compare with those determined by China’s “Temporary Regulations for the Design of the Newly Built Passenger Dedicated Railway with a Speed of 200 km/h to 250 km/h\[^8\]", to find the potentiality of further speed-up of this line.

I. SELECTION OF THE CALCULATION PARAMETERS OF THE MINIMUM CURVE RADIUS

1.1. Speed target value

According to the design of China Railway Eryuan Engineering Group Company\[^9\], Hainan east loop line, from Haikou to new Sanya station, is mainly for intercity passenger cars,
passenger cars around the island, some sea-crossing passenger cars, as well as light freight cars. The speed target value of the railway is 250 km/h for through trains to large stations, 200 km/h for every-stop trains, and 160 km/h for some cross-line trains and light freight trains.

1.2. Rating limits in "German Railway Design Standards 800.0110 - line"

To calculate the minimum curve radius of Hainan east loop line, some parameters need to be chosen by the German High-speed Railway Design Standard, so how to choose the rating limits of the German standard should be explained first. According to the German High-speed Railway Design Standard, the line type parameters should be chosen in comply with the limits specified by the Standard. The limits are graded according to their applicability. When choosing a parameter, its standard limit, estimated limit, and permissible limit should be determined at the same time (fig 1).

The standard value, which is similar to the recommended limit in our provisional specification for high-speed railway design in China, should be used as much as possible.

Estimated value is also known as reckoned limit, which is similar to the general limit in our provisional specification. It is considered a better way to leave a certain range of value to be used in choosing the design parameters of the line, when the project costs too much if using the standard value.

If the parameters are chosen between the standard value and the estimated value, it must be demonstrated and recorded according to the German standard.

Allowable value is also called agreed value, and it is like the difficult value in our Temporary Regulations, which should be explained and approved before use in confirming the parameter limit of the line design.

1.3. Selection of elevation parameters

The way of determining elevation parameters in “German Railway Design Standards

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**Fig 1. Estimated and allowable range specified in “German Rail Standards 800.0110-line”**
800.0110 - line” is quite different from that in China’s Temporary Regulations, which is decided depending on the curve radius and speed.

The selection of the upper limit of the actual elevation range is limited by balance elevation $u_0$ and maximum elevation $u_{\text{max}}$, while the selection of the lower limit by the lowest elevation and the possible minimum elevation $u_{\text{min}}=20$ mm. According to the German Standard 800.0110, the elevation standard value is $u = 6.5v^2 / r$, where $v$ is train speed, and $r$ is curve radius of line. This is equivalent to 55% of the curve equilibrium superelevation.

The deficient superelevation ($u_d$) is decided mainly according to the passenger comfort; at the same time, the amount of maintenance work is also another factor to be considered, which is usually caused by too much of the deficient superelevation.

Considering that there will be some cross line passenger trains and light freight trains of 160 km/h in Hainan east loop line, the excess elevation should be set up, in case the low speed trains are capsized when passing the curves. In German standard, it is unnecessary to verify the excess elevation value by test. Usually, if the train speed through the curve may be raised, a higher excess elevation is preferable. However, the excess elevation is not required in designing Hainan east loop line as this line is mainly for high-speed motor train units, with few cross-line and light freight trains.

Table 1 shows the limit values at different grades for various elevation parameters\(^7\).

<table>
<thead>
<tr>
<th>Actual elevation</th>
<th>Minimum value</th>
<th>Standard value</th>
<th>Estimated value</th>
<th>Allowable value</th>
<th>Exceptional value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deficient</td>
<td>20</td>
<td>100</td>
<td>160</td>
<td>160-180</td>
<td>&gt;180</td>
</tr>
<tr>
<td>superelevation</td>
<td>70</td>
<td>130</td>
<td>150-170</td>
<td>&gt;170</td>
<td></td>
</tr>
<tr>
<td>Excess elevation</td>
<td>70</td>
<td>130</td>
<td>150-170</td>
<td>&gt;170</td>
<td></td>
</tr>
</tbody>
</table>

\(\text{II. CALCULATION OF THE MINIMUM CURVE RADIUS}\)

The line curve radius $r$ is related to the train speed $v$, the deficient superelevation $u_d$, and the actual elevation $u$, and is given by:

$$r = 11.8v^2 (u + u_d)$$

(1)

According to formula (1), the minimum curve radius for trains operated at a single speed of 250 km/h is calculated, and the result is shown in table 2.

<table>
<thead>
<tr>
<th>Standard value</th>
<th>Estimated value</th>
<th>Allowable value</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 338</td>
<td>2 543</td>
<td>2 304</td>
</tr>
</tbody>
</table>

\(\text{III. RESULT COMPARISON WITH CHINA'S TEMPORARY REGULATIONS}\)

According to China’s Temporary Regulations, the elevation parameters of the curve were selected (see table 3).
Table 3. Elevation parameters designed by China’s Temporary Regulations, mm

<table>
<thead>
<tr>
<th></th>
<th>Recommended</th>
<th>General</th>
<th>Largest</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual elevation</td>
<td>170</td>
<td>170</td>
<td>170</td>
</tr>
<tr>
<td>Deficient superelevation + actual elevation</td>
<td>190</td>
<td>220</td>
<td>260</td>
</tr>
<tr>
<td>Excess elevation + deficient superelevation</td>
<td>90</td>
<td>110</td>
<td>140</td>
</tr>
</tbody>
</table>

The minimum curve radius for single-speed operation of high-speed trains and that for mixed-speed operation of high-speed and low-speed trains were both calculated by our Temporary Regulations, and the greater of the two was taken as the result, as shown in table 4.

Table 4. Minimum curve radius, m

<table>
<thead>
<tr>
<th></th>
<th>Recommended</th>
<th>General</th>
<th>Smallest</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4 838</td>
<td>3 958</td>
<td>3 110</td>
</tr>
</tbody>
</table>

Therefore, the minimum curve radius of this line was finally decided as Recommended 5 500-7 000 m, General 4 000 m, and Difficult 3500 m by the China Railway Eryuan Engineering Group Company[9].

Similar to Hainan east loop line, the German Cologne - Frankfurt passenger line has been opened since the beginning of 2001, with a total distance of 226 km, and the time needed for the whole trip is 58 min. There will be ten pairs of trains within every hour between two cities. Theoretically, the fastest speed of trains can reach 330 km/h, but the real operation speed is 300 km/h. Because the line mainly passes through mountain areas with complex terrains, the minimum curve radius of this line was fixed as 3 500 m for general areas, and 3 350 m for difficult mountain areas, and the slop limit was raised from 12.5‰ to 40‰. In this way, the line could adapt well to the different ground changes, and reduce a lot of bridge, tunnel earthwork and stonework projects. Finally 15% to 20% of the total investment was saved[3]. Even so, the train can move smoothly, safely and comfortably.

Comparison shows that the design parameters of Hainan east loop line determined by China’s Temporary Regulations, with a certain capacity reserved, are nearly consistent with those by German standard. In fact, the maximum allowable speed parameters of the line obtained using German deficient superelevation standard are more than 300 km/m, as shown in table 5.

Table 5. Maximum allowable speed, km / h

<table>
<thead>
<tr>
<th></th>
<th>Standard value</th>
<th>Estimated value</th>
<th>Permitted value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>308</td>
<td>313</td>
<td>317</td>
</tr>
</tbody>
</table>

IV. CONCLUSION

Through the comparison above, we can find that there is a big margin left for train’s riding comfort and safety in designing the minimum curve radius of the newly built high-speed...
passenger lines in China. This is mainly because the railway lines in China had been the ones for mixed passenger and freight traffic for long, and the PDL construction and operation, are just in its infancy stage, lacking the experience in balancing high-speed train operation organization and passengers’ comfort and safety. Although it seems more reasonable to decide the line design parameters on the basis of the smooth moving speed as in German standard, the designed deficient superelevation being lower than the German design value can not only ensure the security but also guarantee the riding comfort. As matter of fact, deciding the design parameters from the viewpoint of comfort is more strict and conservative than that from security. From the operation experience of the German mature high-speed railway system, Hainan east loop line has the potential of raising the speed up to 300 km/h in the long term. If the train is moving with this speed through the minimum curve radius, the unbalanced centrifugal acceleration is 0.93 m/s^2, which meets the requirement of German standard that the maximum centrifugal acceleration should be not more than 1.0 m/s^2. High-performance tilting train and electric multiple units (EMUs) with good riding comfort, however, should be developed. With the railway speed raising, a proper operation organization mode is to run passenger trains in the daytime and freight trains with higher speeds at night, and gradually reduce the speed gap between the passenger and freight trains to decrease the wear of the inner rail\textsuperscript{10}. If raising the operation speed of the line, the time from Haokou to Sanya will be shortened from 1.5-2 h to 1 h, which undoubtedly will make the railway transportation more competitive than civil aviation and highway transportation.

Note that the recommended minimum curve radius value presented in this paper is only obtained by taking the German standard for reference. However, all the technical and economic factors need to be synthetically optimized in practice of the minimum curve radius design.

References

THE HIGH-EFFICIENCY ROAD PAVEMENT SURFACE PROFILOMETER AND SOME MICROPROFILE PARAMETERS

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Summary: The basic inducing factor causing vertical displacements and vibrations of the travelling vehicle and its deviation from a desirable trajectory is the microprofile of a road surface. This article presents an overview of a profiling system called dynamic profile converter and its main technical characteristics. The longitudinal road profile data acquisition is of essential importance as its microprofile is primary data for calculating all other indexes which characterize road pavement surface condition such as IRI and power spectral density.

Key words: Dynamic profile converter (DPC), profilometer, road unevenness, road roughness, road microprofile, longitudinal profile, power spectral density (PSD), international roughness index (IRI).

While a vehicle travels along the road, even the best road, it’s sprung and unsprung masses perform various linear and angular displacements which at some intensity cause inconveniences for the driver and passengers, negatively affect on safety of transported cargoes, on operational reliability and durability of a vehicle and causes dynamic on-loading of the pavement. These displacements or oscillations are caused by unevenness - rather small falls and elevations randomly located on the road surface. Road roughness is mainly caused by the fact that existing road construction technologies and construction materials applied not allowed to reach “ideal” smoothness of road surface during the construction, and that in service influence of travelling vehicles and atmospheric factors lead to progressive deformation and destruction of a road pavement and the travel speed plays a prominent role.

That is why it is important for construction engineers and authorities managing road systems to have trustworthy information on a longitudinal profile of the road during its life cycle.
For measurement of the longitudinal profile it is traditionally used static method of rod and level - very accurate but extremely slow and costly method.

Therefore for past eighty years all over the world various devices and data gathering systems for measurement of the longitudinal profile have been invented. They are of different types and methods of operation, but worth saying are high speed profilometers, which meet the requirements of speed, accuracy and safety of data acquisition process.

In 1964 at MADI at the theoretical mechanics department the device for recording a longitudinal road pavement surface microprofile has been devised, makes it possible to collect data at travelling speed up to 80 km/h. It was called the Dynamic Profile Converter (DPC). Speed limits at that time were caused by possibilities of the towing car and bearing ability of the driving wheel of the device. It was the first such device in the world.

It is important to notice that DPC records a microprofile, instead of a longitudinal profile of road. Longitudinal profile consists of three main components, such as texture, microprofile and macroprofile. Texture relates to the coefficient of friction and to skid resistance, macroprofile show design grade and vertical curves, and microprofile is what we usually call roughness and unevenness. Microprofile is a longitudinal road pavement surface profile obtained along each wheel track contains a range of roughnesses wavelengths causing considerable vertical displacements of vehicle. Taking into account modern vehicle response characteristics and travelling speed accepted that microprofile includes roughnesses with wavelengths from 0.3 up to 100 m.

The high - efficiency profiling system includes two DPC units and a car equipped with data acquisition system. DPC unit is a single-wheel trailer with pneumatic tyre of feeler wheel towed by a car with at constant or variable speed that depends on traffic flow.

The dynamic scheme of the DPC is shown on fig 1.

![Fig 1.](image)

Where $M_S$ - sprung mass; $m_U$ - unsprung mass; $m_p$ - slow pendulum mass; $K_S$ - suspension spring stiffness coefficient; $C_S$ - suspension damping constant; $K_t$ - tyre stiffness coefficient; $K_p$, $C_p$ - inertial pendulum suspension stiffness coefficient and damping constant respectively; $Z_S$, $Z_U$ and $Z_P$ - vertical coordinates of sprung mass, unsprung mass and inertial pendulum
respectively. $h$ - Road longitudinal profile ordinates (unevenness).

The trailer of DPC is designed in a way that vertical displacements of towing vehicle are not transferred to data-acquisition equipment, therefore only vertical oscillation of a feeler wheel caused by road surface roughness are recorded.

The trailer through a suspension and damping system is loaded by heavy ballast, which makes a feeler wheel being kept in permanent contact with the surface. The gauge of relative displacements of the wheel - inertial pendulum is installed on the trailer also.

A ballasted external frame of a trailer supports an oscillating internal frame with a feeler wheel. Vertical displacement of the wheel result in angular travel of the internal frame, measured with respect to the horizontal arm of an inertial pendulum. The measurement is made by an inductive transducer associated with the pendulum. Electrical signal produced by the gauge is amplified and recorded.

The structural scheme of DPC trailer is shown on fig 2.

The DPC is linked to the towing vehicle 1 by a drawbar 2 which is placed on the forward end of an external frame 12. On the rear end of an external frame heavy ballast 13 is placed. The external frame is supported by a spring 10 leaning on an internal frame 11. The forward end of an internal frame is connected with an external frame with an axis 5, the rear end of an internal frame leans on an axis of a wheel with the pneumatic tyre 14. The hydraulic shock-absorber 9 (damper) is used for sprung and unsprung masses oscillation damping. Precision part of DPC is inertial pendulum 7 is placed on the front end of an internal frame and the axis of an additional weight of a pendulum is combined with an axis 5. Inertial pendulum is supported by a spring 4, pendulum oscillations are damped by a magnetic damper 8. The gauge of relative movements of a wheel 6 is inductive type (coil). The casing 3 protects a pendulum from turbulent air flows, and also from dust and dirt.

![Fig 2. The structural scheme of DPC trailer](image)
The electronic part of DPC and data acquisition system consists of the recording module of analog signal, sampling device ADC and the personal computer the notebook type. As a target signal is angular moving of a internal frame holding a wheel relative to a pendulum, it is not a scaled copy of a longitudinal profile, but a conversion of this profile by corresponding transfer function of device DPC. To obtain a microprofile as time function an inverse conversion should be made. Using encoder data (traveled distance and speed data) microprofile is recalculated to be a function of traveled distance. Encoder is installed on the towing vehicle wheel.

![Fig 3. Outline of road surface unevenness to longitudinal microprofile conversion section of DPC unit](image)

Later in France rather like profiling system APL (Analyseur de Profil en Long) and in Spain profiling system ARS (Analizador de la Regularidad Superficial) has been made. Both of them copy DPC operation principle and are as well a single-wheel trailer towed by a car and inertial pendulum as main measuring unit. Unfortunately these systems still can obtain profile data only at constant speed.

In the year 2003 a new measuring profiling system using accelerometers (acceleration sensor) (fig 2,15) has been mounted on the DPC trailer in addition to the existed one. And in the year 2006 in addition to two existed systems another one (fig 2,16) has been added. It is combination of triangulation laser sensor for measuring distance to the road surface and acceleration sensor for measuring acceleration normal to the road surface. This type of profiling system is widely spread all over the world today, because it is rather low cost and easy to mount system. It can be mounted almost on any vehicle.

At present, three independent profiling systems are installed on the DPC trailer. Two of them are contact type systems - road surface roughness is measured using the feeler wheel and the latest is noncontact one. This guarantee high reliability and measurement accuracy of longitudinal microprofile of road surface.
Main technical characteristics of DPC profiling system are the following: operation speed can be variable during the measurement, but should be within the range from 15 to 110 km/h. Measured wavelengths of road roughness are within the range from 0.1 up to 250 m, and are subject to the operating speed. This allows to measure longitudinal profiles of runways as well, where long waves are taking into consideration. Microprofile ordinates measurement accuracy is 0.1 mm.

Contact type profiling systems have doubtless advantages over optical profiling systems. Optical systems are very sensitive to dust and reflection capacity of road surface during the rain or when the surface is wet or covered with water layer. All this vitally influences on the measurements accuracy. Contact type systems using a feeler wheel are all-weather systems more over with the wheel due to its smoothing ability it is possible to measure profile of layers made of crashed stones and materials with very rough texture.

The longitudinal road pavement surface microprofile is the primary and absolute characteristic of road longitudinal smoothness.

Profilometers can record microprofile data of for section of any length. Most of indexes used in different countries nowadays and in the past for road roughness rating can be calculated out of microprofile. Thus microprofile data bases make it possible to compare and analyze measurement results of road condition regardless of time they were obtained and/or profilometer type they were recorded with, using this or that particular index.

At present, the international roughness index (IRI) defined by an algorithm by Sayers, is the most popular single-number unevenness indicator. IRI is a single-parameter integral index; it contains roughness magnitude information in the range of wavelengths from 1 to 30 meters. Using IRI is convenient if we talking about road system monitoring or diagnosis. But IRI does not represent frequency characteristics of unevenness. IRI is calculated by a mathematical algorithm from microprofile data array obtained by profilometer. Calculation of IRI from some response type measuring device obtained signal data transformation is not correct approach, because response type device characteristics are not stable at time.

Longitudinal microprofile is completely described by its power spectral density (PSD) as it is realization of a random function which is centered, Gaussian and homogeneous. The spectral density captures the frequency content of a microprofile and helps to identify periodicities.

Information about dominant roughness of certain wavelength allows PSD to be used during construction of road pavement layers for operation control.
For example, on fig 4 microprofile elevation PSD of a highway section is shown. A, B and C are road roughness classes due to ISO 8608, a classification of road unevenness is proposed for the waviness \( w = 2 \). This classification is absolute and does not reflect the fundamental effect of traffic speed on the assessment of the road performance ability. It is evident that microprofile elevation PSD is lying under upper limit of A class in the whole interval of effectively acting wavelengths, a dominating roughness with the wavelength of 5 m is clearly viewed as it corresponds to the wave number of 0.2 cycle/m and it PSD magnitude exceed class B upper limit. This periodical roughness inducts perceptible vehicle vibration while IRI value was quite acceptable.

The survey has found out that due to violation of pavement construction technology the fixed stringline of reference system was not properly drawn and grade control sensors made incorrect measurements between rods with fixed height points. This has resulted in a wave of roughness emerged on the pavement surface with wavelength equal to the distance as anchored fixed height points were located along the lane.

References
ELASTIC MODULUS LIMIT OF SUPPORT LAYER IN TWIN-BLOCK BALLASTLESS TRACK BASED ON TRAIN LOAD EFFECT

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Summary: Using the beam-plate finite element model on elastic foundation, the bending stress and deformation of track slab and support layer due to train load was studied when different types and modulus of support layer were used. The results show the support layer type have great impact on the deformation of support layer and the stress on subgrade, but have little impact on the bending stress of track slab and support layer. The continuous support layer and articulated support layer with shear transfer device at their ends are recommended. In order to restrict the stress in support layer to be less than that in track slab, the modulus of the continuous, the unit, the articulated support layer in unit twin-block ballastless track and the support layer in continuous twin-block ballastless track should not be larger than 15GPa, 22GPa, 20.5GPa and 5GPa, respectively. In addition, the modulus of unit support layer should not be more than 20GPa to ensure the step in support layer less than 1 mm.

Key words: Ballastless track; Beam-plate model on elastic foundation; Support layer; Elastic modulus.

I. INTRODUCTION

Support layer (SL) is an important component of twin-block ballastless track. It locates between track slab (TS) and surface layer of subgrade as shown in fig 1. Its primary role is to transfer the stress from TS to surface layer of subgrade and as stiffness transition between TS and surface layer of subgrade.
The modulus of SL is an important design parameter and should be properly solved, if not it will not only cause an increase in investment, but also may affect the performance and service life of the twin-block ballastless track. In German, the twin-block ballastless track is laid continuously by using the hydraulic bounded layer (HBL), whose elastic modulus is 5~10GPa \(^{[1-3]}\). The twin-block ballastless track was laid separately in ballastless track comprehensive experimental section of Chongqing-Suining line in China, where the SL was made from C20 concrete \(^{[4]}\) and its elastic modulus was much larger than that of German HBL. The difference is due to the recognition to the mechanical character of the support layer. In this paper a ballastless track beam-plate finite element model on elastic foundation was established. The bending stress in TS and SL and the SL deformation with different SL elastic modulus was calculated using this model and the elastic modulus limit of SL was proposed from train load effect viewpoint.

II. CALCULATION MODEL AND PARAMETERS

A ballastless track beam-plate model on elastic foundation \(^{[5]}\) was used to analyze the train load effect as shown in Fig 2. In the model, rails were simulated as discretely supported beam. Fasteners have two elastic layers: rail pads and plate pads. Steel plates located between these two elastic layers. Steel plates were simulated as plate with relatively high rigidity. Rail pads were simulated as single linear spring and plate pads as several parallel linear springs. TS and SL were simulated using plate element because their thicknesses were far less than their lengths and widths which are in line with the structural characteristics of elastic thin plate. Because it was difficult to maintain good connection between TS and SL in unit twin-block ballastless track under long-term environmental and train load effects, they were simulated as two separate layers. Stiffness between the two layers was calculated using the ratio of elastic modulus to thickness of SL. For continuous twin-block ballastless track, because of the deformation consistency of the TS and SL under the train load, they were simulated as a combined slab\(^{[6]}\). The subgrade was simulated as Winkler foundation.
In the model, the CHN60 rail was used, the fasteners were configured with space $a=0.625\text{m}$ whose stiffness were $K_z=50\text{kN/mm}$. The TS was made of C40 concrete, with width $w_1=2.8\text{m}$, thickness $h_1=0.26\text{m}$, and length $l_1=5\text{m}$. SL has width $w_2=3.4\text{m}$ and thickness $h_2=0.3\text{m}$, and elastic modulus ($E_2$) was taken as $2, 5, 8, 10, 15, 20, 25, 30$ and $32.5\text{GPa}$ respectively. The stiffness of subgrade is $k_2=76\text{MPa/m}$. Three different types of SL structures in unit twin-block ballastless track were considered as shown in fig 1: (a) the continuous SL, (b) the unit SL with the same length as TS, and (c) the SL with the same length as TS but with dowel bars at their joints, in order to facilitate, described as the continuous SL, the unit SL and the articulated SL, respectively. To eliminate the border effect, three slabs were considered, but only the middle one needs to be studied in detail. The TS and SL were free boundary in horizontal directions. For the articulated SL, vertical deformations of two adjacent SLs were assumed to be equal to simulate the shear force transfer function of the dowel bars. For continuous twin-block ballastless track (fig 1 (d)), only one combined slab was used, the model length was taken as the same as the unit one.

**III. BENDING STRESS IN SL AND TS VS DIFFERENT SL ELASTIC MODULUS**

Based on the beam-plate model on elastic foundation, when $300\text{kN}$ wheel loads were applied on the rails, the bending stress ($\sigma$) in TS and SL of the unit twin-block ballastless track under different SL elastic modulus was calculated by FEM. The results were shown in figs 3 - 5.

The following can be seen from the calculation results:

1. The bending stress in TS decreased while that in SL increased with SL elastic modulus. Different types of SL structure affected little on the bending stress.

2. For the continuous SL, when the elastic modulus was larger than $15\text{GPa}$, the longitudinal stress in SL surpassed the lateral stress in TS; when the elastic modulus was larger than $27\text{GPa}$, the longitudinal stress in SL surpassed that in TS.

3. For the unit SL, when the elastic modulus was larger than $22\text{GPa}$, the longitudinal stress in SL surpassed the lateral stress in TS.

4. For the articulated SL, when the elastic modulus was larger than $20.5\text{GPa}$, the longitudinal stress in SL exceeded the lateral stress in TS.

To control the crack width and ensure effective load transfer function, reinforcement
should be configured in TS in longitudinal and lateral directions while no reinforcement in SL. Therefore, the bending stress in SL should not be greater than that in TS, which is a basic design principle of the twin-block ballastless track. From this point of view, for the continuous SL, the elastic modulus should not be larger than 15GPa, for the unit SL, the value is 22GPa, and for the articulated SL, the value is 20.5GPa.

Fig 3. The TS and SL stress vs. different SL modulus (continuous SL)

Fig 4. The TS and SL stress vs. different SL modulus (unit SL)

Fig 5. The TS and SL stress vs. different SL modulus (articulated SL)

Fig 6 shows the bending stress in TS and SL of the continuous twin-block ballastless track changing with different SL elastic modulus. As can be seen, the stress in TS reduced while
stress in SL augmented with the increase of SL elastic modulus, because the TS and SL bent around the same neutral axis in continuous twin-block ballastless track. When the SL elastic modulus increased, the neutral axis moved down. When SL elastic modulus was lower than 10GPa, the stress in TS and SL changed greatly, and with the increase of elastic modulus the change trend becomes slow. The SL stress has already surpassed the TS stress when SL elastic modulus was 5GPa. From the basic design principle, the SL elastic modulus in continuous twin-block ballastless track should not be larger than 5GPa.

**Fig 6. The TS and SL stress vs. different SL modulus (continuous TS)**

**IV. DEFORMATION IN SL VS DIFFERENT SL ELASTIC MODULUS**

For different types of SL structure, when the elastic modulus was 5GPa, wheel loads were applied on different locations of the track system, the deformation comparisons of SL were shown in fig 7.

**Fig 7. The SL deformation distribution vs. different load location**
As can be seen from fig 7, different types of SL structure have great impact on SL deformation. When wheel loads were applied on the middle part of the slab (fig.7 (d)), there was little difference in SL deformation distribution among three types; while wheel loads were applied on the edge of the slab (fig. 7(a)), there was great difference.

For the unit SL, when wheel loads were applied on the first fastener and second fastener, large deformation occurred in SL, which will cause large pressure on subgrade, especially great vertical deformation difference in adjacent SLs, namely step. The relationship between the step and SL elastic modulus were listed in table 1. When wheel loads were applied on the second fastener, the greatest step occurred. The step decreased with the increase of SL elastic modulus when wheel loads were applied on the first fastener; while the step increased when wheel loads were applied on the other parts of the track system. For the unit SL, the step in SL caused by train loads should not be greater than 1mm [7]. Thereby the unit SL elastic modulus should not be greater than 20GPa, and otherwise it will cause large force in fastener system [9].

<table>
<thead>
<tr>
<th>SL elastic modulus /GPa</th>
<th>load location</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No.1 fastener</td>
</tr>
<tr>
<td>2.0</td>
<td>0.645</td>
</tr>
<tr>
<td>5.0</td>
<td>0.640</td>
</tr>
<tr>
<td>8.0</td>
<td>0.636</td>
</tr>
<tr>
<td>10.0</td>
<td>0.634</td>
</tr>
<tr>
<td>15.0</td>
<td>0.628</td>
</tr>
<tr>
<td>20.0</td>
<td>0.624</td>
</tr>
<tr>
<td>25.0</td>
<td>0.620</td>
</tr>
<tr>
<td>30.0</td>
<td>0.616</td>
</tr>
<tr>
<td>32.5</td>
<td>0.614</td>
</tr>
</tbody>
</table>

The deformation in articulated SL have been greatly reduced than that in unit SL due to the dowel bars setting at the joints between the two adjacent SLs, the maximum deformation located between that in continuous and unit SL. When wheel loads were applied on the first fastener, a large angle will be formed between the joint of two adjacent SLs. The angle decreased with the increase of SL elastic modulus, and when the SL elastic modulus was 5GPa a maximum angle 0.76‰ was got and within 1 ‰ limit [7] range.

V. CONCLUSIONS

In this paper, the SL type and elastic modulus in unit twin-block ballastless track were studied using the beam-plate finite element model on elastic foundation. The bending stress in
TS and SL and the deformation in SL under different SL elastic modulus were analyzed. The following conclusions were drawn:

(1) The SL structure type has little impact on the bending stress in SL and TS, but has great impact on the SL deformation and subgrade stress. The continuous SL or articulated SL in which dowel bars were set at their ends should be used to improve the subgrade mechanical condition.

(2) From the basic design principle that bending stress in SL should not be greater than that in TS, the elastic modulus of continuous SL, unit SL and articulated SL in unit twin-block ballastless track and SL in continuous ballastless track should not be more than 15GPa, 22GPa, 20.5GPa and 5GPa, respectively.

(3) For the unit SL, when wheel loads were applied on the second fastener, the step is the largest, and the unit SL modulus should not be greater than 20GPa to guarantee the 1mm step limit.

References


I. INTRODUCTION

Non-linear dynamic analysis of civil engineering structures requires large scale calculations. The necessity to perform parametrical studies led us to adopt a simplified approach in order to reduce the computational cost. In this paper the performance of simplified modelling strategies to simulate the non linear behaviour of reinforced concrete structure is presented. The structure is modelled using multifibre beam elements and so the number of degrees of freedom of the problem is reduced. The equations of a multifibre Euler-Bernoulli beam element and its use for modelling the non linear behaviour of reinforced concrete structures are detailed. Comparison with experimental results of a reinforced concrete column submitted to cyclic loading shows the performance of the approach.

However, constitutive models for concrete under cyclic loading have to be able to take into account some complex phenomena such as decrease in material stiffness due to cracking, stiffness recovery which occurs at crack closure and inelastic strains concomitant to damage. An optimum idealization is then needed i.e. one that is sufficiently fine and yet not too costly. To simulate the behaviour of concrete under cyclic loading, a model based on damage mechanics with scalar damage variables (one for tension and one for compression) is used. Unilateral effect, stiffness recovery (damage deactivation) and inelastic strains are also included.

The choice of using a multifibre finite elements configuration combines the advantage of using beam type finite elements with simplicity of uniaxial behaviour. For the following simulations, the Euler-Bernoulli multifibre beam is used. Comparison with experimental results of a reinforced concrete column submitted to cyclic loading shows the performance of the approach.
II. MULTIFIBRE EULER - BERNOULLI BEAM ELEMENT

Different multifibre beam elements exist in the literature. Timoshenko multifibre beam elements are able to simulate non linear shear but 3D appropriate constitutive relations are needed (Guedes et al. 1994, Kotronis et al. 2005, Petrangeli et al. 1999…). Finally, for cases where non linear shear or torsion can be neglected, the Euler-Bernoulli multifibre beam elements are appropriate (Nguyen 2006, Nguyen et al. 2006, Spacone 1996). The Euler-Bernoulli multifibre beam implemented in the finite element code Aster is presented hereafter (Ghavamian et al. 2002, Nguyen 2006).

Consider the beam of figure 1. X the axis of the beam, 1 and 2 being the two nodes of the beam, S the subscripts defining “section variable”. This beam is subjected to a lateral distributed loading q_y, q_z.

![Figure 1: Euler-Bernoulli beam](image)

With Euler-Bernoulli beam theory, the displacement field can be expressed in terms of any section, that is the displacement $u_s, v_s, w_s$ of x and the rotations $\theta_{sx}, \theta_{sy}, \theta_{sz}$ of the plane:

$$u(x, y, z) = u_s(x) - y \theta_{sx}(x) + z \theta_{sy}(x)$$

$$v(x, y, z) = v_s(x)$$

$$w(x, y, z) = w_s(x)$$

$$\varepsilon_{xx} = u_s'(x) - y \theta_{sz}'(x) + z \theta_{sy}'(x)$$

$$\varepsilon_{xy} = \varepsilon_{xz} = 0$$

The virtual work takes the expression:

$$\int_0^L \left( N \delta u_s(x) + M_y \delta \theta_{sy}(x) + M_z \delta \theta_{sz}(x) \right) dx = \int_0^L \left( \delta v_s(x)q_y + \delta w_s(x)q_z \right) dx$$

Where:

$$N = \int_S \sigma_{xx} dS;$$

$$M_y = \int_S z \sigma_{xx} dS;$$

$$M_z = \int_S y \sigma_{xx} dS;$$

$$M_x = 0.$$
\[ T_y = T_z = 0 \]

### 2.1. Interpolations functions

The interpolations functions take the following form:

\[
\{U \}_s = [N][U]
\]

\[
\{U\}_s^T = \{u_1, v_1, w_1, \theta_{x_1}, \theta_{y_1}, \theta_{z_1}, u_2, v_2, w_2, \theta_{x_2}, \theta_{y_2}, \theta_{z_2}\}
\]

\([N]\) is the matrix containing the interpolation functions:

\[
[N] =
\begin{bmatrix}
N_1 & 0 & 0 & 0 & 0 & N_2 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & N_1 & 0 & 0 & 0 & N_4 & 0 & N_5 & 0 & 0 & 0 & N_6 \\
0 & 0 & N_1 & 0 & -N_4 & 0 & 0 & 0 & N_5 & 0 & -N_6 & 0 \\
0 & 0 & 0 & N_1 & 0 & 0 & 0 & 0 & -N_4 & 0 & N_5 & 0 \\
0 & 0 & 0 & 0 & N_1 & 0 & 0 & 0 & 0 & N_5 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & N_4 & 0 & 0 & 0 & 0 & 0 & N_6
\end{bmatrix}
\]

\[
N_1 = \frac{1}{L} \frac{d}{dx}; \quad N_1' = -\frac{1}{L}
\]

\[
N_2 = \frac{x}{L}; \quad N_2' = \frac{1}{L}
\]

\[
N_3 = 1 - 3 \frac{x^2}{L^2} + 2 \frac{x^3}{L^3}; \quad N_3' = -\frac{6x}{L^2} + \frac{6x^2}{L^3};
\]

\[
N_4 = \frac{x}{L} - 2 \frac{x^2}{L} + \frac{x^3}{L^2}; \quad N_4' = 1 - 4 \frac{x}{L} + \frac{3x^2}{L^2};
\]

\[
N_5 = 3 \frac{x^2}{L^2} - 2 \frac{x^3}{L^3}; \quad N_5' = \frac{6x}{L^2} - \frac{6x^2}{L^3};
\]

\[
N_6 = \frac{x}{L} + \frac{x^2}{L^2}; \quad N_6' = -\frac{x}{L} + \frac{3x^2}{L^3}
\]

### 2.2. Stiffness matrix evaluation

The interpolations functions depend on the material properties and they are calculated only once, for the first increment. If \(\{F\}\) and \(\{D\}\) are the section “generalized” stress and strains respectively, the section stiffness matrix \([K_s]\) is calculated as (Guedes et al. 1994):

\[
\{F\} = [K_s]\{D\}
\]

\[
\{F\}_s^T = \{N \quad M_y \quad M_z \quad M_x\}
\]
\[ \{ \mathbf{D} \}^T = \{ \mathbf{u}'(x) \quad \mathbf{\theta}'_x(x) \quad \mathbf{\theta}'_x(x) \quad \mathbf{\theta}'_x(x) \} \]

\[ \mathbf{K}_s = \begin{bmatrix} K_{s11} & K_{s12} & K_{s13} & 0 \\ K_{s22} & K_{s23} & 0 & \\ K_{s33} & 0 & \end{bmatrix}_{\text{sym}} \]

\[ K_{s11} = \int S Eds; \quad K_{s12} = \int S Ezds; \quad K_{s13} = \int S Eyds; \quad K_{s22} = \int S Ez^2ds; \]

\[ K_{s23} = \int S Eyzds; \quad K_{s33} = \int S Ey^2ds; \quad K_{s44} = GJ_k \]

The equation that gives the “generalised” strains as a function of the nodal displacement takes the following form: \[ \{ \mathbf{D} \} = [\mathbf{B}] \{ \mathbf{U} \} \]

\[ [\mathbf{B}] = \begin{bmatrix} N_1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & N'_2 & 0 & 0 & 0 \\ 0 & 0 & 0 & N'_3 & N'_4 & 0 \\ 0 & N'_3 & 0 & 0 & N'_4 & 0 \\ 0 & 0 & 0 & 0 & 0 & N'_5 \\ 0 & 0 & 0 & 0 & 0 & N'_6 \end{bmatrix} \]

\[ N'_3 = \frac{6}{L^2} + 12 \frac{x}{L^2}; \quad N'_4 = \frac{4}{L^2} + 6 \frac{x}{L^2} \]

\[ N'_5 = \frac{6}{L^2} - 12 \frac{x}{L^2}; \quad N'_6 = \frac{2}{L^2} + 6 \frac{x}{L^2} \]

Finally, the stiffness matrix of the element given by: \[ [\mathbf{K}_{\text{elem}}] = \int_0^L [\mathbf{B}]^T \left[ \mathbf{K}_s \right] [\mathbf{B}] dx \]

### 2.3. The mass matrix

The mass matrix of the section and element takes the form:

\[ [\mathbf{M}_s] = \begin{bmatrix} M_{s11} & 0 & 0 & M_{s12} & M_{s13} & 0 \\ 0 & 0 & 0 & \end{bmatrix}_{\text{sym}} \]

\[ M_{s11} = \int_S \rho ds; \quad M_{s12} = \int_S \rho ydz; \quad M_{s13} = \int_S \rho xdz; \]

\[ M_{s22} = \int_S \rho z^2ds; \quad M_{s23} = \int_S \rho zdydz; \quad M_{s33} = \int_S \rho y^2ds; \]

\[ [\mathbf{M}_{\text{elem}}] = \int_0^L \left[ \mathbf{N} \right]^T [\mathbf{M}_s] [\mathbf{N}] dx \]
III. CONSTITUTIVE MODELS OF MATERIALS

The reinforcement steel is modelled with an isotropic cinematic hardening law. Constitutive model for concrete under cyclic loading ought to take into account some observed phenomena, such as decrease in material stiffness due to cracking, stiffness recovery which occurs at crack closure and inelastic strains concomitant to damage.

To simulate this behaviour we use a damage model with two scalar damage variables one for damage in tension and one for damage in compression (La Borderie 1991). Unilateral effect and stiffness recovery (damage deactivation) are also included. Inelastic strains are taken into account thanks to an isotropic tensor. The total strain in formulation of the law is given by:

\[
\varepsilon = \varepsilon^e + \varepsilon^p
\]

\[
\varepsilon^e = \frac{\sigma^+}{E_0(1-D_1)} + \frac{\sigma^-}{E_0(1-D_2)} + \frac{\nu}{E_0}(\sigma - \text{Tr}\sigma)
\]

\[
\varepsilon^p = \frac{\beta_1D_1}{E_0(1-D_1)}f'(\sigma) + \frac{\beta_2D_2}{E_0(1-D_2)}L
\]

The two variables are the damage indicators in tension ($D_1$) and in compression ($D_2$). $f(\sigma)$ is the crack closure function. $\sigma^+, \sigma^-$ denote the positive and the negative parts of the stress tensor respectively. $E$ is the initial Young's modulus and $\nu$ the Poisson’s ratio. $\beta_1$, and $\beta_2$ are material constants. $\varepsilon^e$ is the elastic strains tensor and $\varepsilon^p$, the inelastic strains tensor.

IV. APPLICATIONS

In order to validate the performance of the proposed numerical the 3D multifibre Euler-Bernoulli element is used hereafter to simulate the inelastic behaviour of a column under a general three dimensional load history (Bousias et al. 1995).

\[\text{Fig 2. Reinforced concrete column: description of the specimen}\]
The specimen has a 0.25 m square cross section, a free length of 1.5 m and is considered at the base. Longitudinal reinforcement consisted of eight 16 mm diameter bars, uniformly distributed around the perimeter of the stirrups are 15 mm think (fig 2). Reinforcement bars showed yield stress and ultimate strength of 460 MPa and 710 MPa respectively, the latter at a uniform elongation of 11%.

The test S5 of the experimental campaign is simulated: uniaxial displacement cycles in pairs of linearly increasing amplitude are alternately applied in the two transverse directions at the top of the column (fig 3).

Twenty two multifibres Euler-Bernoulli beam elements having 2 Gauss points are used to simulate the column. Each section has 36 fibres for concrete and 8 for steel. Base slab is not simulated and the specimen is considered fixed at the base. Comparison of the model and experimental results is represented in fig 4. The models simulate correctly the global behaviour of the mock-up in terms of displacement and forces in both directions.

Fig 3. Reinforced concrete column: Displacement load histories

Fig 4. Reinforced concrete column: Model and experimental results
V. CONCLUSIONS

This work investigates the simplified modelling strategy to simulate non linear behaviour of reinforced concrete structures. The formulation of Euler - Bernoulli multifibre beam element is elaborated. The constitutive behaviour used for concrete is based on damage mechanics and is suitable for cyclic loadings. A classical plasticity model is chosen to reproduce the behaviour of reinforcement bars. The simulation is performed using Euler- Bernoulli multifibre beam elements that provide a compromise between numerical cost, quality of results and facility of modelling. The use of simplified modelling brings rapidity in the meshing and reduces the computation time.

As demonstrated by the results presented in the paper, the model was able to reproduce with good approximation the response of the structures. This confirms that the level of the discretization and the type of numerical elements adopted in model are sufficient to describe the non linear behaviour of the reinforced concrete structures.

References


[10]. ASTER finite element code, official web site: http://www.code-aster.org/
OPTIMIZATION AND ALGORITHM FOR THROUGH TRAIN CONNECTIONS AT DISTRICT STATIONS

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Summary: In the optimization of train diagrams, selecting the arrival and departure paths of the through trains has a great impact on the dwell time at district stations. In this paper, on the basis of train paths and the through train connection time standard at district stations, we built a mathematical model aiming at minimizing dwell time of through trains at two adjacent district stations, and then converted this into a network flow model to which is added a source and a sink node. We propose a new algorithm for solving the network flow model based on the minimum-cost flow algorithm. A case study for through trains from the Guiyang south railway station to the Chongqing west railway station shows that the algorithm is reliable and efficient for the problem of through train connection, and there is a reduction in the total dwell time that the through trains spend at two adjacent district stations.

Key words: Train diagram; Through train connection; Minimum-cost flow algorithm; District station

INTRODUCTION

The optimization of train diagrams plays an important role in the organization of train operations. This has attracted much attention, especially in China. Sun and Li\[1\] proposed a method of optimizing the wagon diagram for a single-track line. Cao\[2\] studied an optimization model and its algorithm for adjusting a train diagram on single-track rail lines. Peng\[3\] put forward an integer programming model and a solution for drawing a double-track train diagram with a computer. Shi\[4\] presented a timing-cycle iterative optimizing method for drawing single-track railway train diagrams. Zhao\[5\] presented a 0-1 linear model and an optimal algorithm for adjusting the train diagram on a single-track railway. Carey and Crawford\[6\] considered railway networks consisting of complex stations linked by multiple one-way lines in each direction, and developed heuristic algorithms to assist in the task of finding and resolving conflicts in draft train schedules. Zhou and Zhong\[7\] dealt with a double-track train scheduling problem for planning applications with multiple objectives. Goverde\[8\] described a stability theory to analyze timetables on sensitivity and robustness to delays based on a linear system description of a
railway timetable in max-plus algebra. D’Ariano\[^9\] considered the creation of a conflict-free timetable after train operations were perturbed. Rodriguez\[^10\] considered the routing and scheduling of trains through a junction and proposed a constraint programming model. In the optimization of train diagrams, selecting the arrival and departure paths of the through trains has a great impact on the dwell time at district stations. However, very few reference studies exist regarding this problem. Although Zhu\[^11\] optimized the through freight train connection and the equilibrium of the departing train at a technical station, the model and solution strategy were not tested on real-life data. In this paper, we contribute to the problem by focusing on the minimum dwell time of through trains at two adjacent district stations. Firstly, we construct a mathematical model of the problem and a network flow model. Then we propose an algorithm to solve the problem of through train connection. Finally, we verify the algorithm on the through trains from the Guiyang south railway station to the Chongqing west railway station.

I. MATHEMATICAL MODEL

The problem of through train connection can be modeled by a constrained assignment problem\[^12\] as follows. As shown in fig 1, the through trains from marshalling stations A to B go through two adjacent district stations C and D.

\[ \text{Fig 1. Location of stations A, B, C and D} \]

Let \( I = \{1, 2, L, \ldots , l\} \) denote the set of arrival paths to district station C, \( J = \{1, 2, L, \ldots , m\} \) denote the set of departure paths from district station C and the set of arrival paths to district station D, \( K = \{1, 2, L, \ldots , n\} \) denote the set of departure paths from district station D, and \( c_{ij} \) represent the dwell time at district station C when the through train chooses the arrival path \( i \) ( \( i \in I \) ) and departure path \( j \) ( \( j \in J \) ).

Let \( x_{ij} \) be a binary variable. It is equal to 1 if the through train arrives at path \( i \in I \) and departs at path \( j \in J \), and 0 otherwise. Let \( p \) be the number of through trains through two adjacent district stations C and D, \( t_{ai} \) be arrival time of train path \( i \) to district station C, \( t_{ei} \) be departure time of train path \( j \) from the district station C, and \( t_{ci} \) be the dwell time at the district station C.

The meanings of \( z_{jk}, d_{jk}, t_{dj}, t_{aj} \) and \( t_{ci} \) at the district station D are similar to those of \( x_{ij}, c_{ij}, t_{dj}, t_{aj} \) and \( t_{ci} \) at the district station C, respectively.

Then the formulation of the problem can be provided as follows:

\[
\min \sum_{i=1}^{l} \sum_{j=1}^{m} \sum_{k=1}^{n} c_{ij} x_{ij} + d_{jk} z_{jk}
\]  

(1)
Subject to $\sum_{j=1}^{m} x_{ij} \leq 1 \quad (i = 1, 2, \ldots, l)$ \hfill (2)

$\sum_{i=1}^{k} x_{ij} \leq 1 \quad (j = 1, 2, \ldots, m)$ \hfill (3)

$\sum_{k=1}^{n} z_{jk} \leq 1 \quad (j = 1, 2, \ldots, m)$ \hfill (4)

$\sum_{j=1}^{m} z_{jk} \leq 1 \quad (k = 1, 2, \ldots, n)$ \hfill (5)

$\sum_{i=1}^{l} \sum_{j=1}^{m} x_{ij} = p \quad (i = 1, 2, \ldots, l; j = 1, 2, \ldots, m)$ \hfill (6)

$\sum_{j=1}^{m} \sum_{k=1}^{n} z_{jk} = p \quad (j = 1, 2, \ldots, m; k = 1, 2, \ldots, n)$ \hfill (7)

$x_{ij} \in \{0, 1\} \quad (i = 1, 2, \ldots, l; j = 1, 2, \ldots, m)$ \hfill (8)

$z_{jk} \in \{0, 1\} \quad (j = 1, 2, \ldots, m; k = 1, 2, \ldots, n)$ \hfill (9)

$$
c_{ij} = \begin{cases} 
  t_{f_i} - t_{d_i} & t_{f_i} - t_{d_i} \geq t_C \\
  t_{f_i} - t_{d_i} + 1440 & t_C - 1440 < t_{f_i} - t_{d_i} < t_C \\
  t_{f_i} - t_{d_i} + 2880 & t_{f_i} - t_{d_i} < t_C - 1440 
\end{cases} \quad (10)
$$

$$
d_{jk} = \begin{cases} 
  t_{f_k} - t_{d_j} & t_{f_k} - t_{d_j} \geq t_D \\
  t_{f_k} - t_{d_j} + 1440 & t_D - 1440 < t_{f_k} - t_{d_j} < t_D \\
  t_{f_k} - t_{d_j} + 2880 & t_{f_k} - t_{d_j} < t_D - 1440 
\end{cases} \quad (11)
$$

In this formulation, constraint (2) indicates that each arrival path $i$ is assigned to no more than one departure path $j$, constraint (3) indicates that each departure path $j$ is assigned to no more than one arrival path $i$, constraints (6) and (7) mean that all through trains have one arrival path and one departure path. Constraints (4) and (5) at district station C have the identical meanings as constraints (2) and (3) at district station D, respectively.

II. NETWORK MODEL

In order to solve the above model, the corresponding topological network is constructed through dummy source node $V_s$ and dummy sink node $V_t$. The network graph $G$ is shown in
The node set of network $G$ is $\{V_s, D_1, D_2, \ldots, D_l, E_1, E_2, \ldots, E_m, F_1, F_2, \ldots, F_m, G_1, G_2, \ldots, G_n, V_t\}$ and the arc set is
\[
\{ (V_s, D_i) \mid i = 1, 2, \ldots, l \} \cup \{ (D_i, E_j) \mid i = 1, 2, \ldots, l; j = 1, 2, \ldots, m \} \cup \\
\{ (E_j, F_k) \mid j = 1, 2, \ldots, m; k = 1, 2, \ldots, n \} \cup \{ (G_k, V_t) \mid k = 1, 2, \ldots, n \}
\]
Each arc corresponds to the value $(e_{ij}, b_{ij})$, where the value $e_{ij}$ is the capacity of the arc, and $b_{ij}$ is the unit flow cost of the arc.

Because the source node $V_s$ and the sink node $V_t$ are dummy, the cost of one unit flow of arc $(V_s, D_i)$, $(E_j, F_k)$ and $(G_k, V_t)$ is zero, that of $(D_i, E_j)$ is $c_{ij}$, and that of $(F_k, G_k)$ is $d_{jk}$. The capacity of all arcs is 1.

**Fig 2.** The network graph $G$ of through-train connection at two adjacent district stations

### III. ALGORITHM

#### 3.1. Preliminaries

Definition 1\(^{[13]}\) The residual network $G(x)$ corresponding to a feasible flow $x$ is defined as follows: We replace each arc $(i, j) \in A$ by two arcs $(i, j)$ and $(j, i)$. The arc $(i, j)$ has cost $b_{ij}$ and a residual capacity $r_{ij} = e_{ij} - x_{ij}$, and the arc $(j, i)$ has cost $b_{ji} = -b_{ij}$ and a residual capacity $r_{ji} = x_{ij}$.

Lemma 1\(^{[14]}\) A feasible solution $x^*$ is an optimal solution of the minimum cost flow problem if and only if the residual network $G(x^*)$ contains no negative -cost directed cycle.

Lemma 1 provides a simple approach for solving the minimum cost flow problem and finding the optimal scheme of through train connection at two adjacent district stations.
Theorem 1\cite{15} The optimal scheme of through train connection at two adjacent district stations can be obtained by solving the minimum cost flow of network $G$.

Proof Because the cost of one unit of flow of arc $(D_i, E_j)$ and $(F_j, G_k)$ is the dwell time of through train at district stations $C$ and $D$, respectively, the minimum cost flow of network $G$, whose volume is the number of through trains between marshalling stations $A$ and $B$, is the minimum dwell time of through train connection at two adjacent district stations.

The algorithm first establishes a feasible flow $x$ in the network $G$. Then it searches negative-cost directed cycles through iteration in the residual network and augments flows in these cycles. The algorithm terminates when the residual network contains no negative-cost directed cycle.

3.2. Steps for through train connection problem

Step 1: According to the arrival time and departure time of train at two adjacent district stations, calculate $c_{ij}(i = 1, 2, \ldots, l; j = 1, 2, \ldots, m)$ and $d_{jk}(j = 1, 2, \ldots, m; k = 1, 2, \ldots, n)$, and construct the matrices $[c_{ij}]_{l \times m}$ and $[d_{jk}]_{m \times n}$.

Step 2: Construct the network $G$ for the problem of through train connection at two adjacent district stations by the matrices $[c_{ij}]_{l \times m}$ and $[d_{jk}]_{m \times n}$.

Step 3: Find the flow $x$ whose value is $p$ in network $G$.

Step 4: Form the residual network $G(x)$.

Step 5: Test for the existence of a negative-cost cycle.

If there are no negative-cost directed cycles in $G(x)$, the flow $x$ is optimal. Then turn to step 6.

If there is a negative-cost directed cycle $Q$, a new flow $x'$ can be determined according to:

$$x'_{ij} = \begin{cases} x_{ij} + \delta & \text{if } (i, j) \in Q \\ x_{ij} - \delta & \text{if } (i, j) \notin Q \\ 0 & \text{otherwise} \end{cases}$$

Where $\delta = \min_{(i,j) \in Q} \{r_{ij}\}$. Return to step 4.

Step 6: The minimum cost flow of $G$ is the optimal solution of through train connection at two adjacent district stations.

IV. CASE STUDY

We used the real-life data of through trains from the Guiyang south railway station to the
Chongqing west railway station to verify the validity of the model. These through trains go through two adjacent district stations: Nangongshan station and Ganshui north station. We assume that the connection time of through trains at both Nangongshan station and Ganshui north station is 25 min. The time of all possible arrival paths and departure paths at Nangongshan and Ganshan north station is shown in Table 1. We attempted to find the minimum dwell time of seven through trains from Table 1. The algorithm was implemented with the C++ language and run on a 1.73GHz Pentium PC with 512MB RAM. Computational results are listed in Table 2. The total dwell time at two adjacent district stations is 712 min. The dwell time of trains obtained by our algorithm is less than that by the method of Zhu [11]. With an increase in the number of district stations, the performance of our algorithm will be more efficient and reliable.

**Table 1. Selection set of arrival time and departure time at Nangongshan station and Ganshui north station**

<table>
<thead>
<tr>
<th>Nangongshan station</th>
<th>Ganshui north station</th>
</tr>
</thead>
<tbody>
<tr>
<td>arrival time</td>
<td>departure time</td>
</tr>
<tr>
<td>18:41</td>
<td>18:31</td>
</tr>
<tr>
<td>19:15</td>
<td>19:36</td>
</tr>
<tr>
<td>20:30</td>
<td>21:40</td>
</tr>
<tr>
<td>22:16</td>
<td>22:57</td>
</tr>
<tr>
<td>1:59</td>
<td>0:11</td>
</tr>
<tr>
<td>2:59</td>
<td>3:55</td>
</tr>
<tr>
<td>3:09</td>
<td>6:52</td>
</tr>
<tr>
<td>5:32</td>
<td>8:04</td>
</tr>
<tr>
<td>6:27</td>
<td>9:43</td>
</tr>
<tr>
<td>7:33</td>
<td>10:12</td>
</tr>
<tr>
<td>9:00</td>
<td>13:55</td>
</tr>
<tr>
<td>9:24</td>
<td>16:03</td>
</tr>
<tr>
<td>9:37</td>
<td>17:06</td>
</tr>
<tr>
<td>16:35</td>
<td></td>
</tr>
</tbody>
</table>

**Table 2. The arrival time and departure time of seven through trains**

<table>
<thead>
<tr>
<th>No. of trains</th>
<th>Nangongshan station</th>
<th>Ganshui north station</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>arrival time</td>
<td>departure time</td>
</tr>
<tr>
<td>1</td>
<td>18:41</td>
<td>19:36</td>
</tr>
<tr>
<td>2</td>
<td>20:30</td>
<td>21:40</td>
</tr>
<tr>
<td>3</td>
<td>3:09</td>
<td>3:55</td>
</tr>
<tr>
<td>4</td>
<td>6:27</td>
<td>6:52</td>
</tr>
<tr>
<td>5</td>
<td>7:33</td>
<td>8:04</td>
</tr>
<tr>
<td>6</td>
<td>9:37</td>
<td>10:12</td>
</tr>
<tr>
<td>7</td>
<td>16:35</td>
<td>17:06</td>
</tr>
</tbody>
</table>

V. CONCLUSIONS

We constructed the mathematical model and flow network of the problem of through train connection at two district stations. We designed an algorithm based on the well-known...
minimum cost flow algorithm to efficiently solve the mathematical model. A case study on through trains from the Guiyang south railway station to the Chongqing west railway station shows that our algorithm is efficient and reliable. Furthermore, this algorithm is also effectively applied to more district stations.

References
ESTIMATION OF ROAD HARM
FROM THE HEAVY VEHICLES RIDE DURING SPRING TIME

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Summary: The technique for definition of harm from movement of heavy vehicles on highways in spring, adverse on conditions of humidifying the periods of year is offered. Relative shares of harm from decrease in bearing ability of road designs and because of the raised deterioration of road coverings are considered.

The admissible error is proved at definition of harm and insignificant influence of deterioration of coverings on a combined value of harm from movement of a heavy vehicle is shown. Correlation dependence for harm definition is offered at different bearing ability of road designs and excess degree by axial loadings of a vehicle of admissible sizes.

Keywords: A technique, the heavy vehicle, harm from movement, a highway bearing ability, deterioration of pavements, a harm share, an error, correlation dependence.

In Russia at insufficient bearing ability of road designs during the spring periods of year either forbid movement of heavy vehicles [1], or suppose on a paid basis [2] for the purpose of indemnification of arising harm. To the heavy carry any vehicle, axial loadings which exceed the established admissible values, causing the raised deterioration of road designs and reduction of reserve maintenance periods of their service.

Harm or fare count proceeding from actual bearing ability of the road design characterized by corresponding admissible axial loading [3] which is defined depending on factor of durability of the road clothes, calculated by results of diagnostics of highways (tab. 1). The durability factor is equal to a parity actual (Eф) and demanded (Eтр) modules of elasticity of a design for flexible road clothes or to a parity of actual and demanded thickness of a covering for roads with rigid pavements.

Besides it is considered that the harm size is proportional to destroying influence of a heavy vehicle which, as it is known, is estimated by the corresponding factor of reduction of a vehicle to settlement loading.
For definition of harm by a technique [4] the assumption as a part of a transport stream of additional intensity of movement of the heavy car which reduces a reserve maintenance period of service of road pavements to size ≈ 0,7-0,8 from standard (settlement) value of “Тn” is provided:

\[ N_{1x} = x N_1 \]  

(1)

Where: \( N_{1x} \) - the intensity of movement of a transport stream led to settlement loading with Heavy vehicles in the first considered year; \( x \) - factor of increase in settlement intensity of movement at the expense of inclusion in structure of a transport stream of \( j \) heavy vehicles, led to settlement loading 100 kN; \( N_1 \) - actual intensity of movement of a transport stream in the first year. The operation, led to settlement loading, a unit/days.

<table>
<thead>
<tr>
<th>Factor of durability(^1)</th>
<th>The Safe load [Q] on each axis of a vehicle at:</th>
</tr>
</thead>
<tbody>
<tr>
<td>( K_{тп} )</td>
<td>Single axis, (tf)</td>
</tr>
<tr>
<td>1,09 - 1,14</td>
<td>12</td>
</tr>
<tr>
<td>1,05 - 1,08</td>
<td>11</td>
</tr>
<tr>
<td>1,00 - 1,04</td>
<td>10</td>
</tr>
<tr>
<td>0,94 - 0,99</td>
<td>9</td>
</tr>
<tr>
<td>0,88 - 0,93</td>
<td>8</td>
</tr>
<tr>
<td>0,81 - 0,87</td>
<td>7</td>
</tr>
<tr>
<td>0,71 - 0,80</td>
<td>6</td>
</tr>
<tr>
<td>0,60 - 0,70</td>
<td>5</td>
</tr>
<tr>
<td>0,50 - 0,59</td>
<td>4</td>
</tr>
</tbody>
</table>

**Table 1**

**The note:** 1 tf = 10 kN

\(*\) Distance between axes less than 2 m.

\(**\) Distance between extreme axes less than 3,5 m

Generally the size of harm from movement of the heavy vehicle includes expenses for premature major repairs as a result of decrease in bearing ability of road designs and an expense for premature repair because of the raised deterioration of road pavements:

\[ H_j = \frac{C_{сп} \times \alpha_j}{\Delta N_{сп} \times (1 + E_{пп})^{\text{сп}}} + \frac{C_{п} \times \alpha_{тп}}{\Delta N_{п} \times \Delta C_{II}} \]  

(2)

where:

\[ \Delta C_{II} = \sum_{i=1}^{m} \frac{1}{(1 + E_{пп})^{\text{сп}}} - \sum_{i=1}^{m} \frac{1}{(1 + E_{пп})^{\text{п}}} \]  

(3)

\( C_{сп} \) and \( C_{п} \) - accordingly expenses for major repairs because of decrease in the bearing

\(^1\) The durability factor corresponds to results of test of a road design the vertical loading equal 50 kH (axial loading 10 tf)
abilities of a design and repair as a result of deterioration of a road pavements; $\alpha_j$ and $\alpha_{cj}$ - reduction factors $j$ - that heavy vehicle to settlement loading (100 kN) accordingly on bearing ability to design and deterioration of a road pavement; $\Delta N_{ep}$ and $\Delta N_p$ - quantity проездов heavy vehicles during the period to the major repairs, led to settlement loading, accordingly on bearing ability of a design and deterioration of a pavement; $t_p$ - actual service life of a road design, years; $t_{фI}$ and $t_{фII}$ - accordingly actual and standard service life of a road pavement; $E_{ШP}$ - the specification for reduction of expenses occurring at different times (discount); m - actual quantity of repairs of a road pavement in limits actual service life of road design $t_ф$; $\mu$ - standard quantity of repairs of a road pavement in limits actual service life of road design $t_ф$.

Quantity heavy vehicles for any period of time $t_i$ define, having presented movement volume at different initial the intensity movements in the form of the sum of members of a geometrical progression:

$$\Delta N = n \times \frac{q^{\frac{1}{2}} - 1}{q - 1} \times (N_{1x} - N_1)$$  \hspace{1cm} (4)

Where: $n$ - settlement number of days in a year [5].

For calculation under the formula (3) shares of expenses occurring at different times $\Delta C_n$ the correlation dependence defining service life of a road pavement by criterion of deterioration and received on the basis of norms of reserve maintenance periods [6, 7] is used:

$$t_П = K_{ШМА} \cdot K_{ДКЗ} \cdot \frac{42474}{N_p + 4734}$$  \hspace{1cm} (5)

Where: $K_{ШМА}$ - the factor considering work the pavements from шебеноchno-mastichnyj asphalt ($K_{ШМА} = 1,4$); $K_{ДКЗ}$ - the factor considering influence of a road-climatic zone. Factor is accepted accordingly: 1,24; 1,14 and 1 for I-II, III and IY-Y RCZ; $N_p$ - the intensity of movement led to settlement loading, on most to the loaded lane which is calculated with the account проездов each axis vehicle.

The range of values of factors of the durability defining admissible axial loading of a vehicle (tab 1 see), allows to estimate an admissible error in size of harm which should not exceed 20 % as have shown the calculations spent in the environment of Microsoft Excel for each axial loading at the maximum and minimum factor of durability [8]. Considering it, additional calculations also have shown that the share of harm from deterioration of a covering is, as a rule, within an admissible error (6 - 18 %) and can not be considered at harm definition in practice from journey of the heavy vehicle.

Results of calculations also testify to some law of change of harm with change of admissible axial loading $[Q]$ and degrees of its excess $\Delta Q$ axial loadings of a heavy vehicle. The processed data as an example are taken out on fig. 1 in the form of relative change of rates of harm $\Delta H_j$ depending on change of admissible axial loading $[Q]$ and sizes of its excess $\Delta Q$. 

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Fig 1. Relative change of harm depending on change of admissible axial loading (2) and degrees of its excess (1)

On the basis of the data fig 1 corresponding dependences are picked up by a method of the least squares at which the greatest deviation of settlement values does not exceed 12 - 16 % that is in limits of a supposed error.

Calculations are executed for different categories of the roads located in different road-climatic zones. Universal dependence for definition of the rate of harm (rbl./km) on roads IY of a road-climatic zone is as a result offered:

\[ I_j = I_{1k} \times (1 + 0.2 \times \Delta Q^{1.92} \times \left( \frac{a}{[Q]} - b \right)) \] (6)

Where: \( I_{1k} \) - Harm reference value (at \( \Delta Q = 0.5 \) tf) in the prices 2009 for roads different categories and the solidity of road designs defined by \( E_{TP} \pm \) of 10 %; a and b the empirical parameters depending on a category of roads (tab 2).

<table>
<thead>
<tr>
<th>Road category</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IY</th>
</tr>
</thead>
<tbody>
<tr>
<td>( I_{1k}, \text{rbl.}/\text{km} )</td>
<td>2</td>
<td>8,3</td>
<td>22</td>
<td>37</td>
</tr>
<tr>
<td>a</td>
<td>99</td>
<td>37,5</td>
<td>34,3</td>
<td>10,3</td>
</tr>
<tr>
<td>b</td>
<td>8,9</td>
<td>2,75</td>
<td>2,43</td>
<td>0,03</td>
</tr>
<tr>
<td>( E_{TP}, \text{MPa} )</td>
<td>337</td>
<td>274</td>
<td>239</td>
<td>223</td>
</tr>
</tbody>
</table>

For highways IY-Y of categories with gravel road (\( E_{TP} = 114 \) MPa) and initial size of the rate of harm \( N_{1k} = 85 \) rbl./km:

\[ I_j = N_{1k} \times (1 + 0.14 \times \Delta Q^{1.24} \times \left( \frac{7.3}{[Q]} + 0.27 \right)) \] (7)
Formulas (6) and (7) are fair at $\Delta Q > 0$ and $[Q]$ from 5 to 10 tf.

Under identical traffic conditions the average size of harm on highways in II and III road-climatic zones in 2,14 and 1,6 times accordingly exceeds harm from heavy vehicles in the conditions of IY a road-climatic zone.

The basic conclusions:

- The technique of definition of size of harm (rbl./ km) from journey of the heavy vehicle is offered;
- The error in definition of size of harm from journey of the heavy vehicle is proved admissible 20 %;
- It is established that the share of harm from deterioration of a road pavements is in limits of an admissible error and can not be considered at definition of size of harm from movement of heavy vehicles in practice;
- The universal decision for definition of specific harm (rbl./ km) depending on admissible axial loading of a vehicle and size of its excess taking into account solidity of a road design is offered at a considered category of a highway.

References


A METHOD FOR EVALUATING THE GAS PERMEABILITY OF DAMAGED CONCRETE

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CHARLIER ROBERT
University of Liege, Belgium

Summary: This research concerns the establishment and the validation of a mechanical law for gas permeability of concretes. Based on the obtained relationships between permeability coefficient $K$ and applied stress in the pre-peak phase and the correlation between diffuse damage and the pre-peak stress of concrete, we proposed a phenomenological model, which can describe not only the evolution of gas permeability with diffuse damage, but also with localized damage. A verification of the permeability value when the damage becomes fracture (cracked state) in comparison with a reference value given by Poiseuille law was also realized to evaluate the precision of the proposed model. An example of simulation to determine the structural values of gas permeability through a concrete disk in a splitting test was carried out to validate this model.

Key words: Concrete, gas, permeability, stress, diffuse damage, localized damage, crack.

I. INTRODUCTION

Evaluation of the gas permeability of concrete under loading is the topic of some studies in literature in recent years [1], [2], [3], [4], [5], [6], [10], [15], [17]. A number of interactions laws for permeability prediction have been proposed in literature [4], [8], [14]; in which, permeability evolution is represented as a function of the damage state of concrete. However, these laws are only applicable when the damage remains homogeneous (diffuse damage), a state observed in pre-peak phase of concrete. In the post-peak behavior of concrete, when the damage becomes localized due to the strain localization in fracture process zone, the application of these laws becomes delicate. The evolution of permeability according to these laws are either too fast or too slow [19], in comparison with the real values. The effort of Picandet [14] to propose a general law also make the application of his law for different types of concrete becomes difficult. The method to establish such interaction laws by measuring the evolution of permeability in post-peak phase of concrete proposes some problems concerning the accuracy of the measurement of permeability values when flux flow becomes turbulent and the difficulty in determining damage variable $D$ from experimental curves.
In this research, we based on the experimental results obtained in laboratory when we measured the gas permeability of an ordinary concrete with compression resistance \( f_c = 30 \text{MPa} \) [20] to develop a phenomenological law for gas permeability of damaged concrete. From the evolution curve of permeability value with pre-peak stress level and the correlation of diffuse damage level and pre-peak stress, we first propose an evolution law of permeability with diffuse damage; then the proposed law is extended when damage becomes localized. The critical value of permeability corresponding to the totally damage state is compared with a reference value given by the Poiseuille law in order to estimate the precision of the extended law. A simple example of simulation of structural permeability is carried out to validate the proposed law.

II. EXPERIMENTAL RESULTS

Fig 1 shows the obtained experimental results of the evolution of permeability with pre-peak relative applied stress [20]. We found that the obtained values of gas permeability \( K \) of considered concrete vary from \( 2.10^{-17} \) to \( 4.10^{-16} \text{ (m}^2\text{)} \) with the applied stress; in which a slight decrease of \( K \) in comparison with the initial value \( K_o \) of intact concrete is recorded when applied stress \( \sigma \leq 0.7\sigma_{\text{max}} \) (\( \sigma_{\text{max}} \) the maximal value of applied stress), then the value of \( K \) increases strongly from this threshold. The form of permeability curve with the applied stress level is relatively similar to the results of other authors in the literature [5], [15], [17].

![Fig 1. Variation of \( K \text{ (m}^2\text{)} \) according to applied stress](image)

III. PROPOSITION OF AN INTERACTION LAW: DAMAGE-PERMEABILITY

Recalling the mechanical behavior of concrete from the intact state to the totally damaged state [13], [19], we represent the complete relation between applied stress and strain in concrete in fig 2. The different phases of behavior including:

+ Phase OA corresponding to an elastic or quasi-elastic behavior: concrete remains intact and behaviors as a linear elastic material. The porous matrix in concrete is nearly unchanged. The main principles of the elastic theory are totally appropriate to apply in the modeling of the behavior of concrete.
+ Phase AB corresponding to the pre-peak non-elastic behavior where concrete begins to be damaged. A type of damage called homogenous damage or diffuse damage is recorded in concrete. The porosity and the connectivity of concrete porous matrix increase strongly bring about the increase of permeability. Gas flowing through concrete can be considered as laminar. Damage theory can be perfectly used to simulate the behavior of concrete.

+ Phase BC corresponding to the localized damage of concrete where micro-cracks localize strongly to create macro-cracks, which can be observed by the naked eye. Concrete becomes weakly discontinuous, porosity and connectivity of concrete increase rapidly bringing about a strong decrease of anti-seepage capacity. Damage theory of concrete can be weakly applied to simulate the behavior of concrete.

+ Phase CD called cracked phase just after the localized damage of concrete, where the concrete medium is totally discontinuous. Fracture theory of concrete should be applied in this phase.

\[
\sigma = (1-D)\sigma_0
\]

![Diagram of Different phases of behavior of concrete](image)

Fig 2. Different phases of behavior of concrete

We base on correlations between the evolution of permeability as a function of the pre-peak stress and that according to the diffuse damage to establish the relationship between permeability and diffuse damage (fig 3).

From our results shown in fig 1 and the experimental results of several authors in the literature (Picandet (2001) [15], Choinska (2006) [5] ...), we found that there is an interval of pre-peak stress \([0.7 \sigma_{\text{max}} - \sigma_{\text{max}}]\) where the variation of stress from \(0.7\sigma_{\text{max}}\) to \(\sigma_{\text{max}}\) corresponds with a variation of the diffuse damage \(D\) from 0 to 0.25. Thus, the diffuse damage in the concrete starts from an applied stress \(\sigma = 0.7\sigma_{\text{max}}\) and reaches the max value of 0.25 when \(\sigma = \sigma_{\text{max}}\). This justifies the observation of a sharp increase of the permeability of concrete in the interval \([0.7\sigma_{\text{max}} - \sigma_{\text{max}}]\) due to the effects of diffuse damage. The threshold \(D = 0.25\) of the diffuse damage is a given phenomenological value found in the literature (fig 3) [5], [15]. It is noteworthy that a more accurate value of the threshold of diffuse damage may be obtained by benchmarking audits through numerical simulations for different damage models of concrete.
However, this interval of variation of diffuse damage obtained by experimental measurement has also been confirmed by the numerical results of Chatzigeorgiou (2005) [2].

The equivalence between the pre-peak stress and diffuse damage as well as their effects on permeability of concrete in pre-peak phase and post-peak phase are shown in fig 3. Indeed, during three phases of behavior of concrete including the elastic phase, the diffuse damage phase and the localized damage phase, the evolution of permeability of concrete are different:

+ When concrete is elastic, the Young modulus $E_o$ is unchanged, the damage doesn’t occur in concrete, the permeability of concrete is constant or lightly decreases in comparison with the initial value $K_o$. To simplify the simulation, we supposed $K = K_o$ in this phase.

+ When the diffuse damage initiates in concrete rendered by a lightly decrease of the elastic modulus or a nonlinear behavior in pre-peak phase, the permeability of concrete increases sharply. The experimental results show an exponential trend of permeability with the applied stress. Accordingly, we can get an evolution of the permeability with the diffuse damage like an exponential function.

+ When damage becomes localized, concrete permeability increases strongly but the evolution law in this phase is not easy observed from experimental results.

During the diffuse damage phase of concrete, basing on the experimental results (fig 1) and following the spirit of some authors in the literature like Picandet (2001), Choisnka (2006) [4], [5], [14], [15] we proposed the formula of this exponential function as follow:

$$K = K_o \alpha \exp(\beta D)$$  \hspace{1cm} (1)

Where $K$ is the permeability of damaged concrete (m$^2$), $K_o$ is the initial permeability, $D$ is the diffuse damage, $\alpha$ and $\beta$ are material parameters.

![Fig 3. Correlation between the permeability evolution with applied stress and with damage state in concrete](image-url)
To calibrate the values of material parameters $\alpha$ and $\beta$ in (1) for the experimental results on fig 1, we use the hypothesis $K = K_0$ when $D = 0$ (when concrete remains elastic). Then, two boundary conditions of (1) are:

$$K_{D=0} = K_0$$  \hspace{1cm} (2)

$$K_{D=0.25} = K_{\sigma = \sigma_{\text{max}}}$$  \hspace{1cm} (3)

By replacing (2) and (3) in (1), we obtained a couple of values of $\alpha$ and $\beta$ for the considered concrete M30 as follows: $\alpha = 1$ and $\beta = 15.529$.

After the applied stress reaches the maximal value and decreases, the permeability of concrete continues to increase. In this case, relationship permeability - post-peak stress is not clear and very difficult to obtain by experimental work. So, instead of finding such relation, we always proposed to represent the evolution of permeability as a function of the localized damage by the formula (1). Two boundary conditions of this function are:

$$K_{D=0.25} = K_{\sigma = \sigma_{\text{max}}}$$  \hspace{1cm} (4)

$$K_{D=1} = K_f$$  \hspace{1cm} (5)

Where $K_f$ is the permeability of cracked concrete calculated using the Poiseuille law [4], [14].

This proposed law is well validated for diffuse damage of concrete when we compare it with the proposed laws in literature of Choinska (2006) and Picandet (2001) (fig 4.a). In addition, the extension of this law provides a critical value of permeability (when $D \approx 1$) nearly equal to the value calculated with Poiseuille law (see annex) (fig 4.b).

Fig 4.a shows the increases of the permeability $K$ with diffuse damage obtained by the proposed law and by the laws of Picandet (2001) and Choinska (2006). We found that the evolution of the permeability of our proposed law is the fastest for $D \leq 0.15 - 0.16$. For the values of $D = 0.16 - 0.25$, our law lies between the Picandet’s one and the Choinska’s one.

Fig 4.b shows that the critical value of permeability calculated by the proposed law (1) is very close to the reference value calculated according to Poiseuille law when $D \approx 1$. While the increase of permeability by law Picandet (2001) or Choinska (2006) is too fast; the critical values of permeability calculated according to these laws are both greater than the reference value. The slow increase of the permeability according to the law of Gawin (2003) [8] gives a critical value significantly smaller than the reference value, so this law is not very precise either.

**Fig 4.** Evolution of permeability with diffuse damage (a) and with localized damage (b)
From the proposed model for predicting the increase of permeability with damage state of concrete as above, we propose a process to calculate the structural permeability of concrete structures as follows:

+ For intact zones of concrete structures: \( K = K_o \) (permeability of intact concrete or initial permeability).

+ For damaged zones of concrete structures: \( K = K_o \alpha \exp(\beta D) \).

The values of \( \alpha \) and \( \beta \) of different types of concrete are easily obtained from the experimental results measured in laboratory as the process applied with concrete M30 as above. We only need the results measured in pre-peak phases of concrete; all the difficulties in the measurement of concrete permeability in post-peak phase are hence avoided.

IV. CALCULATION OF STRUCTURAL PERMEABILITY

The different values of fluid permeability migrating through concrete can be summarized as follows:

+ For intact concrete: \( K = K_o \).

+ For damaged concrete: \( K = K_o \alpha \exp(\beta D) \)

Using the non-local damage model Mazars [13], [16] the average value of permeability migrating through a damaged concrete structure can be calculated in the following formula:

\[
K_m = \frac{K_o S_o + K_{Di} S_{Di}}{S_T}
\]  

(6)

Where \( K_m \) is the mean value of permeability, \( K_o \) is initial permeability, \( S_o \) is the area of intact zones, \( K_{Di} \) is permeability penetrating through damaged zone, \( S_{Di} \) is the area of damaged zones, \( S_T \) is total area of concrete structure.

Then, the absolute value of structural permeability migrating through concrete structure is:

\[
K = K_m S_T
\]  

(7)

V. EXAMPLE OF SIMULATION

Considering a concrete disk in splitting test (fig 5) with diameter \( d = 110 \text{mm} \), two wedges of width \( b = 11 \text{mm} \) are placed on the top and the bottom of the concrete disk to avoid boundary localized failure. Mazars non-local damage law for concrete is assigned for all elements of the disk; linear elastic law for steel is assigned for all elements of two wedges. Damage parameters according to concrete with \( f'_{c} = 30 \text{ MPa} \) include [18]: damage threshold \( \varepsilon_{Do} = 0.00007 \); compressive coefficients \( A_c = 1.4 \); \( B_c = 2000 \); tensile coefficients \( A_t = 0.8 \); \( B_t = 20000 \); shear factor \( \beta = 1 \) and internal length \( l_c = 0.02 \text{m} \). Steel wedges with Young modulus \( E = 2.1E5 \text{ MPa} \) and yield stress \( f_y = 370 \text{ MPa} \). The initial value of permeability \( K_o = 10^{-17} (\text{m}^2) \) (fig 1).
The structural permeability values migrating through the concrete disk have been evaluated as the area weighted average values of all permeability values through different part of the disk according to damage level during loading process (formula (6)). The total structural value is then calculated according to (7).

**Fig 5. Definition of splitting test (a); Damage zones (b, Distribution of permeability on the disk ($D_{max} = 0.75$) (c)**

The distribution of permeability on the disk is represented in fig 5.c for a maximal damage $D_{max} = 0.75$. We observe an increase of five orders of the permeability from initial value $K_o = 10^{-17}$ (m$^2$) to a maximal value $K_{max} = 8 \times 10^{-13}$ (m$^2$) and this increase has well verified the evolution law of permeability according to (1) (fig 4).

In fig 6, the convergence of numerical results according to non-local damage model is verified for four different numbers of finite elements (FEs) (1625, 2745, 3413 and 4965).

Using of different numbers of finite elements is also necessary in verification of the convergence of structural permeability when we make an area weighted average of this value. In fig 6.a and 6.b, we found also a good convergence of permeability evolution according to applied load and maximal damage.

**Fig 6. Relation between applied load $P$ and maximal lateral displacement $D_{x_{max}}$ and with the mean value of structural permeability $K_m$**

Fig 6.b shows that the structural permeability of concrete remains almost unchanged when applied load is smaller than a critical value equal to 270 kN. And only after this threshold, the
structural permeability begins to increase strongly. This critical value of applied load is determined equal to 0.65$P_{\text{max}}$ and has been confirmed with a good agreement with experimental results in the literature [5], [15].

In fig 7.a, we found that the evolution of average structural permeability value with maximal damage is fairly similar to the observation in fig 4. However, a difference of six orders of maximal value of average structural permeability in comparison with the initial value $K_o$ is smaller than the difference of seven or more orders of permeability values calculated according to (2).

If we consider the COD (Crack Opening Displacement) as the sum of two times of the maximal lateral displacement $D_x$, we can represent the evolution of total value of structural permeability calculated according to (5) with the COD in fig 7.b. We found that the form of this curve is entirely similar to the adjustment curve of experimental results of Choisnka [4] or Pincandet [14], [15]. However, our values of total structural permeability are large than these experimental values due to two reasons: (i) the diameter of measurement of experimental disk is 7.7 cm which is smaller than the diameter of numerical disk and equals to 11cm. (ii) the experimental concrete is high strength concrete M75, which gives smaller permeability values in comparison with those of ordinary concrete M30.

In fig 7.b, we found also that structural permeability values through the concrete disk begin to increase remarkably when COD reaches a critical value approximately equal to 0.01mm. This threshold is verified with the same value as the threshold of applied load $P$, from which, the response of the disk begins to be nonlinear (fig 6).

VI. CONCLUSIONS

The permeability of concrete remains nearly constant and equals to initial values $K_o$ when applied stress is smaller than a critical value found in the interval [0.6 - 0.8]$\sigma_{\text{max}}$, this value depends on concrete type. For the considered concrete M30, this threshold $\sigma \approx 0.7\sigma_{\text{max}}$. After reaching this threshold, permeability of concrete begins to increase strongly according to
applied stress. Basing on the correlation of the permeability evolution with applied stress and with diffuse damage in pre-peak phase of concrete, we have proposed a phenomenological law for evaluating the gas permeability evolution of damaged concrete. This law has been validated initially by a simple example modeling the structural permeability through a concrete disk in a splitting test. The obtained results show that the structural permeability evolution according to the applied stress and according to the crack opening displacement is quite similar to the experimental observations in the literature. The proposed model can describe the evolution of permeability not only with the diffuse damage but also with the localized damage due to strain localization in damage zones of concrete. The method to calibrate material parameters is simple and only carried out in pre-peak phase of concrete behavior, the supposition of a laminar flow is always satisfied. This method hence can overcome the difficulties that we may have in the estimation of permeability values of concrete in post-peak phase by experimental measurement.

ANNEXE

The reference value of permeability according to Poiseuille law is calculated as follows:

From the basic formula of Poiseuille law for permeability in cracked media:

\[ K_f = \frac{[u]^2}{12} \]  \hspace{1cm} (8)

We replaced the crack opening displacement (COD) \([u]\) of a macro-crack by a band of micro-cracks so that the permeability migrating through the former equal to the latter [4], [5] (fig 8).

\[ \lambda l_c \]

Fig 8. Approximation of \([u]\) by a band of micro-cracks \(\lambda l_c\)

We have:

\[ [u] = \int_0^{\lambda l_c} (\bar{\varepsilon} - \varepsilon_{Do}) \, dx = (\bar{\varepsilon} - \varepsilon_{Do}) \lambda l_c \]  \hspace{1cm} (9)

And (9) becomes:

\[ K_f = \frac{(\lambda l_c)^3}{12} (\bar{\varepsilon} - \varepsilon_{Do})^2 \]  \hspace{1cm} (10)

In which, \(\varepsilon_{Do}\) is the initial damage threshold of concrete, \(\bar{\varepsilon}\) is the non-local equivalent strain, \(l_c\) is the internal length, \(\lambda \approx 3\). [4], [5].
The reference value of permeability of concrete can be calculated as a function of damage variables according to (10).

References

PLANNING FOR COASTAL ROAD USING TIDE RESISTANCE CEMENT & CONCRETE

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Summary: Currently, Vietnam as well as other countries around the world are faced with changes of climate phenomena, such as global warming, melting ice, rising sea levels,...; the climate change is much bad causes to the general activities of man. The characteristics of Vietnam's long coastline, the coastal road system has an important role in the economic and social development, national defense - national security. Ensuring the sustainable development of coastal road system before the change of climate, sea level rise is an urgent problem and deserve attention.

Key words: The coastal road system, the impact, Cement concrete, High-Volume Fly Ash Concrete (HVFAC), Sand Concrete (SC), Fly Ash.

I. VIETNAM IN GENERAL VIEW

1.1. Geography Condition

Vietnam located in the Indochina Peninsula, belongs to South East Asia. Vietnam's territory runs along the east - coast of the peninsula. Vietnam has borders with China (1281 km), Laos (2130 km) and Cambodia (1228 km) and 3444 km long coastline bordering the Bac Bo gulf, South Sea and Thailand Gulf.

Vietnam has an area of 331,212 km², including approximately 327,480 km² land and more than 4200 km² sea inland, with more than 2800 islands, reefs - big and small, near and...
Truong Sa and Hoang Sa, which belongs to Vietnam, is internal waters, territorial sea, exclusive economic zone and continental shelf of the Vietnam’s Government is determined almost three times the interior area of over 1 million square kilometers.

Vietnam’s terrain is very diverse as natural areas like the North West, North East, Highlands is full of hills and mountain forests, while the flat land covering about les than 20%. About 40% of mountains, 40% of hill, and its covering is about 75%. The plains as the Red River Delta, the Mekong River delta and coastal areas such as coastal North Central and South Central Coast. Overall, there are three domain areas in Vietnam: northern highlands and the Red River delta, central part with coastal lowlands, the highlands run along the Truong Son mountain, and south is the Cuu Long river Delta. the highest point of Vietnam is 3143 meters, at the top of Fansipan, belongs to the Hoang Lien Son mountain range.

Vietnam has a tropical climate in the South with two seasons (rainy season, from mid May to mid September, and the dry season, from mid October to mid April) and monsoon climate in the north with four seasons (spring, summer, autumn and winter). Lying along the coast, the climate of Vietnam is regulated by ocean currents and having climatic factors of sea. The average relative humidity is 84-100% all year. Annual rainfall from 1200 to 3000 mm in some places and could cause more flooding, nearly 90% of rain falling in summer. The average annual temperature in the plain generally slightly higher than the mountains and plateaus. Temperature varies from a lowest. of 5°C from December to January, the coldest month, for more than 37°C in April, the hottest month. The division of the season in the north more clearly than in the southern, where the only exception of the highlands, seasonal temperature difference is only a few degrees, usually around 21-28 C. Every year, Vietnam always have to against to 5 to 10 storms and flooding per year.

1.2. The impact on the coastal road works

The impact of sea level rising is extremely serious when Vietnam has the coastline of 3260km, over 1 million km² territorial waters, over 3000 islands near the shore and two offshore islands, many low-lying coastal areas have to suffered from heavy flooding during the rainy season and drought, stalinization, dry seasons.ect. Climate change and sea level rising will exacerbate this situation, increasing flooded, drainage, increased coastal erosion and salinization of water sources, affecting agricultural production and refresh water, risk for coastal construction projects like roads, ports, factories, cities and residential areas.

The ports include docks, storage yards, warehouses are designed in the present sea level will must be renovated again, even have to move to another place. The railway north - south roads and near the seaside of road systems will be affected. The rising level of the sea water
II. OVERALL PLANNING OF THE COASTAL ROAD AND THE ACTUAL USING OF CEMENT CONCRETE MATERIALS

2.1. Overall planning of the coastal road

Coastal road for economic-social development, tourism, contributing to relief and disaster prevention; national security.

As planned was approved by the Prime Minister, the coastal route starts at Port Red Mountain, Mui Ngoc Quang Ninh province to the border gate in Ha Tien, Kien Giang province, about 3041 km.

The coastal route is planned to be near the sea, formed on the basis of connecting many existing roads (including national highways, provincial roads, rural roads) is combined with investment in new works, connections with national and regional planning, regional. In particular, the coastal route can be combined with sea dyke systems and systems of coastal defense in order to facilitate the handling of situations dealing with natural disasters and strengthen national defense and security area.

The general size:

Minimum size of the coastal route as follows:

- North: from Quang Ninh to Ninh Binh: 3rd level
- North Central near the sea (các tỉnh từ Thanh Hoá tới Quảng Trị): 3rd level
- Mid central part (from Thua Thien Hue to Binh Dinh): 3rd level;
- South Central part (from Phu Yen to Binh Thuan): 4th level;
- East South (from Ba Ria Vung Tau to Ho Chi Minh city): 4th level;
- West South (from Tien Giang to Kien Giang): 4th level.

2.2. The use of cement concrete roads in Vietnam

Actual using of cement concrete materials in road construction mainly focused on the rural road network from the commune level or lower units (roads, village roads). According to statistics, the total number of km of cement concrete is 22,227 kilometers (approximately 9%,...
not to mention urban roads and special roads), in which the ratio of cement concrete road for rural roads peak areas (18,898 km, covering over 85%), the lowest provincial roads (211 km, covering 0.95%), the highway is 626 km, accounting for 2.82% (mainly Ho Chi Minh Trail and some of the highways were flooded in flood season).

If we involve the using the cement concrete according to areas, it is showed that: the North Central region has had the highest percentage (over 78%), while other areas as cement concrete road rate is very low (below 5%), this shows us (especially rural roads) using cement concrete is not popular in some areas.

### 2.3. The issues for building roads made of concrete cement coastal

#### 2.3.1. Capacity to applying materials

As planned in development of cement industry was approved by the government, the demand for cement in Vietnam in 2005 was 29 million tons, 2010 is 46 million tons and 2015 is 62 million tons in 2020 is 68 [5]. At the moment, domestic cement production is still excess production capacity, specific statistics in 2010, the quantity to provide more than the factual demand to use is 5 million tons.

Cement yield to produce cement concrete is a lot, but type of cement is not abundant, especially cement used in construction for roads in coastal areas, this type of cement should have the characteristics of drag bending strength and ability to sulfate. Moreover, to facilitate the using of cement in construction works in general, and in particular the coastal road will require construction of norms and legal provisions enabling the contractors to use Blended Cement Portland (PCB) is currently popular on the market.

In road building with materials cement concrete, to minimize the using of more cement that polluted environment, there may be many kinds of mineral additives used in combination with cement like fly ash, blast furnace slag, rice husk ash, silica... It should use the three component binder (portland cement-slag-fly ash shrine) to make cement concrete to withstand the impact of sea water.

Currently, the type mineral additives can be manufactured in Vietnam, because it is an indispensable component when producing cement concrete with special requirements, moreover, using mineral additives to reduce the amount of cement in and increase using the industrial solid wastes are intended to reduce CO2 emissions in the environment.

Fly ash has an important role in producing high performance concrete. C. Muller, and Peter R. Hardtl Schiellbl [6] when they researched on X ray concrete has amount cement not to be hydrated (without fly ash) had showed that a large amount of hydraulic cement after months not
be hydrated. Results of research also showed that fly ash increases the strength of concrete is not as it reacts with water, but because it reacts with the chemical products of hydraulic cement to create durable products and make profiles sound concrete structures and can withstand the impact of the salt in sea water.

D. Stephen H. Lane and Celik Ozyildirim [7] had studied the effects of fly ash, slag and alkali - Aggregate reactivity (ASR) and they had concluded that the concrete and mortar containing fly ash, slag and are more durable for silicafume ASR.

According to the planned development of the electricity industry from 2006 to 2015 was approved by Prime Minister, is expected to put to use many power plants run on coal capacity expected in table 2.1

<table>
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<th>No</th>
<th>Capacity, MW</th>
<th>Consume coal, million ton</th>
<th>Fly and slag produce, million ton</th>
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<td>1</td>
<td>35.090</td>
<td>95.9</td>
<td>27.34</td>
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However, to use effectively a large mount of slag ash from the thermal power plants, we need a plan for the construction of the plant ash team to ensure quality when used in combination with cement.

**Fine Aggregate (sand)**

Ability to provide sand for cement concrete in sustainable construction at coastal Vietnam, currently the source of sand for cement concrete is utilize from rivers, sand resources depleted day by day as well as the rapid growth of the construction industry. Therefore the using of alternative materials for large sand grains are necessary, first of all that is taking advantage of pour sand in some areas as coast to replace a small portion of aggregates for concrete cement that some countries over the world have been using.

Sand has large reserves and are mined from the rivers, according to data [8] of dust, mud, clay of these sand all most are permitted. Also present in all Vietnam, there are many basic materials, building stone, the average amounts of mine rock put out environment the tailings of 20% ÷ 35%, This is very large and if We use it, We shall protect the environment effectively [9].

The Rivers in Northern of Vietnam has a density distribution, sand is now mainly from these rivers. Characteristics of sand in this area is relatively clean, its composition are suitable for making cement concrete according to current standards, a large fineness modules $M_k > 2.2$.

Sand in Vietnam's central provinces from Nghe An to Binh Thuan is mainly white with
very small fineness module, currently not be used for cement concrete. But according to the research recently, this type can be used for cement concrete of coastal areas.

At South Central, from Quang Ngai to Binh Thuan has “huge sands reserves”, and mostly fine sand smooth.

South eastern region, sand is mined at Bien Hoa Dong Nai. Projects in Ho Chi Minh City and Mekong Delta mainly mine on the Dong Nai River. Its module \( M_k \) is from 1:50 to 2.5.

The area of the Mekong Delta provinces, sand are mined mainly from Tien and Hau rivers, but sand here have module of smooth very low and a large amount of impurities, the sand used for concrete mainly from Tan An, An Giang, Vinh Long with the module \( M_k \) magnitude 1.0 to 2.0. So when used for cement concrete is composed of sands also be able to meet supply.

There are some difficulties in using cement concrete for coastal road project in Vietnam. It is current standards are not prescribed as fine sand used to replace part coarse sand.

**Coarse Aggregate (crushed stone or gravel)**

Coarse aggregate used for cement concrete must be focussed on mineral composition, chemical and basic formation of rocks. Because the strength of the material was much interested, durability of concrete materials by time, the reaction was resistant to alkali aggregate, sulfates resistant, anti-shrinkage drying, scaling and abrasion road surface.

The amount of macadam and gravel used for construction of cement concrete roads are great, to build 1km cement concrete road surface width of 7m must be used about 3000 m³ to nail macadam and concrete.

The distribution of reserve of macadam are not equally, in northern Vietnam, Central Vietnam, mainly along the borders of Laos, there is little madacam in South Vietnam. This fact is difficulty for build roads in this area.

Solutions to ensure for construction in areas there is rare macadam, especially when the concrete construction of coastal roads, it is possible to use sand and solid waste materials using binders combined ash slag cement to manufacture of cement concrete or concrete slabs inserted as a high quality road paving.

The outstanding difficulties in the process provide macadam for construction are regulations on inter-system standards in the industry is not unified. Criteria for concrete aggregates in road construction requirements in accordance with AASHTO standards, defined at the construction industry in general is Russian standards, it causes difficulties in production and evaluate grading.
Chemical additives

Additives are used to make cement concrete are very abundant and be produced domestically. Therefore, cement concrete can suffer affective from sea water, must consider the compatibility with the type of cement and adhesive instead.

Water

Water can be used for cement concrete need be fresh and ensure cleanliness.

2.3.2. Suitable Cement concrete for the construction of high way

According to scenario of rising sea water due to climate change, Vietnam is one of the countries that be heavy impact. The question is: which materials be used?.

We should use cement concrete to build coastal roads system to ensure for sustainable development of road system. Cement concrete should be high performance. It have both strength and endurance over time, not corroded by the action of salt water. We can reduce using cement by using high fly ash content (High-Volume Fly Ash HVFA) as research in using fly ash cement up to 50% [12]. HVFA concrete have properties to withstand the impact of seawater as a corrosion resistant sulfate highly; limited types of corrosion that occurring in concrete; waterproofing capable of Cl-ions to reduce the high food worn in reinforced concrete, plus an additional concrete has high waterproofing capabilities.

For rich sand region but lack of stones which along the coast of Vietnam, It can be used sand concrete (Sand Concrete-SC). Sand concrete can utilize the available sand source for aggregates, fly ash can be used to improve the structure and reduce the cost of the concrete. Sand concrete, concrete without stone is one of advances technology on concrete structures, contribute to "greenlization" of the concrete industry.

We can also use cement concrete to build coastal road. We can utilize solid material, fly ash for layers of concrete pavement.

2.3.3. The situation of technical and technology in Vietnam

Currently, the technology in building cement road can satisfy the fabrication and construction of concrete material that presented above. However, We also need a long-term strategy in using clean technology.

Currently, construction technology cement concrete is generally not popular, from manufacturing to construction and mining, there are no standards for construction and checking and takingover the work. The standard design of cement concrete according to document named 22 TCN, 223-95, however this standards need be reviewed and added to complete.

Vietnam has all conditions and the ability to develop cement concrete road in large-scale.
To implement this policy, the first it is necessity to supplement and complete standards, procedures and norms from design to construction and checking and takingover the work. Now we have only a few forein technology of construction but it is very expensive. The suitable technology for large scale of using concrete, It need be noted that: need having the slots to ensure technical as well as sence, against early cracking, shrinkage, and creep of concrete.

III. CONCLUSION

Potential use of cement concrete road that using new technology which adaption with for Vietnam coastal road.

In applying the progressing of cement concrete roads, it is necessary to conduct the research to maximize the advantages and disadvantages of this type of road, this type of cement concrete is releval for the flat terrain, geological stability, uniform in terms of intensity and ability to resist the ravages of high water; therefore, it is better use this type of road in Vietnam.

To apply the construction of cement concrete pavement for high-level roads (highways, national highways important) effectively, in early stages we need taking application in standards and advance construction technologies, then, we should develop the technical standards as the standard for design, construction, testing ect… to enhace the efficiency of construction, mining as well as road management.

There are mechanisms to mobilize capital to build cement concrete roads, in which, the amount from the Government in the poor community is always higher, with rule “Both Government and citizen make the changes”.

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