# Development of a Static Back-calculation Software and Its Application to Maintenance of Flexible Pavement Structures

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Higher costs and technological advancement in road construction and evaluation demand correspondingly accurate decision making processes for maintenance and repair works. In Japan, pavement surface conditions in terms of pavement surface cracks, rutting, and longitudinal roughness are the main factors used to establish criteria for pavement maintenance and repair. Structural adequacy factor of the pavement is neglected. In this paper, as a way to introduce structural evaluation during decision making, development of a static back-calculation computer program called BALM (back analysis for layer moduli) and its application to pavement maintenance is introduced. Pavement structural capacity is widely being estimated by the use of pavement layer moduli obtained by back-calculation analysis using deflections from a falling weight deflectometer (FWD) test. BALM uses axisymmetric multi-layered elastic program (AAMES) for forward analysis and modified Gauss-Newton method for back-calculation of layer moduli. Truncated singular value decomposition is used to stabilize computations. BALM can back-calculate pavement layer moduli and provide confidence intervals of the results. It is possible to fix one or more layer moduli or consider interface slip condition during back-calculation. Back-calculated modulus of asphalt concrete is modified considering an average temperature. Equivalent layer coefficients would then be determined to compute effective full-depth asphalt concrete thickness ($T_E$). $T_E$ is obtained as sum of products of pavement layer thicknesses and equivalent layer coefficients. $T_E$ values will enable pavement engineers evaluate structural adequacy of the pavement system before making final decision on maintenance or repair of the deteriorated road surface.

## INTRODUCTION

In Japan, the key road network constitutes of the national expressways, which form main part of the automobile transportation network throughout the country, and the general national expressways, which connect major cities, principal harbors, airports, interchanges of national expressways etc. The other roadways are prefectural and municipal roads, depending on the control authority. Most of these roads are subjected to severe loading conditions due to higher rates of heavy vehicles as well as higher temperature during summer season where average pavement surface temperature is about $60°C$.

Two types of pavement design procedures exist: one is the design procedure for a new pavement and the other is rehabilitation design. According to the Japanese pavement design and construction code (*1*), pavement thickness required if the entire depth is constructed of hot asphalt mixtures ($T_A$ method) is determined based on the design CBR of subgrade soil and the road classification by one-way daily traffic volume. As for the rehabilitation design, maintenance control index (MCI) has been adopted in Japan as an index to evaluate surface conditions of national expressways and the need for maintenance and rehabilitation. The index is determined from three main parameters: cracking ratio, rutting depth and longitudinal roughness.

As mentioned above, the overall concept of pavement performance relies on functional performance indicators while neglecting structural performance factor. The first part of this paper introduces the general pavement design and rehabilitation procedure performed in Japan. And the second part of this paper introduces the use of a multi-layered elastic analysis, where layer moduli are back-calculated from FWD test results.

Whenever necessary, back-calculated asphalt concrete modulus would be converted to equivalent modulus at standard temperature, 20°C. Inoue (*2*) provided the relationship between layer moduli and equivalent layer coefficients in $T_A$ method for pavement materials by conducting extensive laboratory tests, FWD tests and elastic multi-layer analyses. By using Inoue's method, which is incorporated in this back-calculation procedure, equivalent layer coefficients are evaluated to find effective structural capacity of the pavement system in terms of effective full depth asphalt concrete thickness, $T_E$. This procedure is expected to complement existing design and rehabilitation methods, which were developed in large part based on past experience, by introducing structural performance factor into the decision making process.

## STRUCTURAL DESIGN AND MAINTENANCE METHODS FOR ASPHALT PAVEMENT IN JAPAN

The Japanese design method for asphalt pavement was developed based on CBR method, AASHO Road Test results and test pavement results in Japan. The CBR method was derived from the relationship between CBR of the subgrade soil and total pavement thickness as originally established by the American Corps of Engineers while in AASHO Road Test results, the relationship between thickness indexes and repeated traffic loads was derived. Based on the Japanese test pavement results as well as past experience, the above mentioned relationships were revised and modified to suit various Japanese road conditions. Furthermore, flexible pavement design method based on structural number (SN), which is found in AASHTO guide for design of pavement structures (3), was also taken into consideration when developing Japanese design method. Finally, $T_A$ method was adopted and is included in the Japanese pavement design and construction code.

### Formulation of Japanese Structural Design Procedure

The following equation is used for structural design of asphalt pavement (considering 90 % confidence level of traffic volume):

$$T_A = 3.84 \times N^{0.16} / CBR^{0.3} \quad (1)$$

where $T_A$ = design full-depth hot mix asphalt concrete thickness (cm)
$N$ = cumulative equivalent 49 kN wheel load for the design period (i.e. required 49 kN wheel load passes to fatigue failure)
$CBR$ = design CBR of the subgrade soil

The number of equivalent 49 kN wheel load passes per day, $N_{49}$, over the measured section is given using the following equation:

$$N_{49} = \sum_{j=1}^{m} \left[ (P_j / 49)^4 \times N_j \right] \quad (2)$$

By using Equation (2), the value of N can be obtained as:

$$N = \sum_{i=1}^{n} \left( N_{49} \times 365 \times r_i \right) \quad (3)$$

where $P_j$ = representative of the $j^{th}$ wheel load range (kN)
$m$ = number of wheel load range $j = 1 \sim m$
$N_j$ = number of $P_j$ passes
$n$ = design period
$r_i$ = $N_{49}$ equivalent traffic growth at the $i^{th}$ year ($r_i \geq 1.0$)

The general design process is as follows:

1) Selection of design CBR of subgrade soil based on test results
2) Selection of classified traffic category as shown in TABLE 1
3) Selection of target value of $T_A$ as shown in TABLE 2

$T_A$ is defined by the following equation:

$$T_A = a_1 T_1 + a_2 T_2 + ... + a_n T_n \tag{4}$$

where $a_n$ = coefficient shown in TABLE 3 for $T_A$ calculation
$T_n$ = thickness of pavement layer (pavement thickness equals to total sum of $T_n$)

4) When structural design results meet requirements for $T_A$, cost effectiveness of various alternatives shall be evaluated.

**Maintenance and Rehabilitation Procedures for Asphalt Pavement**

*Evaluation of Pavement Surface*

Periodic inspection is usually carried out over two to four years on the whole network section. The purpose of this inspection is to measure fundamental surface characteristics (crack, rut depth, longitudinal roughness) and to evaluate whether the pavement maintains a good level of performance or not and also to identify pavement sections with low level of performance. Skid resistance, water permeability, and tire rolling noise are also checked at the same time or on another day to be arranged. Automated measuring devices mounted on a vehicle, thus causing minimal traffic disruption, perform all these inspections except water permeability.

Since new pavement design concepts were introduced from, among others, AASHO Road Test results, the concept of PSI was also applied for evaluation of pavement surface. However, as survey results in Japan have shown that sometimes PSI does not match actual pavement failure trends, maintenance condition index (MCI) was developed by the then Japanese Ministry of Construction following national wide pavement condition surveys. MCI is used to evaluate pavement surface conditions indicating functional performance levels of national roads. Crack, rut depth and longitudinal roughness are measured in order to obtain this index.

Crack is evaluated in the index of crack ratio, which is calculated from cracked area (mesh method) divided by monitored area. Rut depth is evaluated using rut depths on the wheel paths, which are measured by either mean method or peak method. Span length in the longitudinal direction is usually 20m. Longitudinal roughness is evaluated using standard deviation that is calculated from longitudinal difference at mid point of 3 m long straight beam at every 1.5 m measuring interval along a 100 m length section.

These three fundamental characteristics are directly adopted in MCI. In general, MCI is evaluated using four equations shown below, the minimum of which is considered to be MCI value for the measured section:

$$MCI = 10 - 1.48C^{0.3} - 0.29D^{0.7} - 0.47\sigma^{0.2} \tag{5a}$$

$$MCI_0 = 10 - 1.51C^{0.3} - 0.30D^{0.7} \tag{5b}$$

$$MCI_1 = 10 - 2.23C^{0.3} \tag{5c}$$

$$MCI_2 = 10 - 0.54D^{0.7} \tag{5d}$$

where $MCI$, $MCI_0$, $MCI_1$, $MCI_2$ = maintenance control indexes
C = cracking ratio (%)
D = rutting depth (mm)
$\sigma$ = longitudinal roughness (mm)

MCI is a useful index for overall evaluation of the road surface and planning of the pavement maintenance, including priority and selection of the procedure and method of maintenance or repair. TABLE 4 shows an approximate relationship between the values of MCI and level of maintenance.

## DESIGN METHODS FOR ASPHALT OVERLAY THICKNESS

### Empirical Overlay Design Method

The most common asphalt overlay thickness method use in Japan is the design of overlay thickness using CBR value of the subgrade soil to obtain effective (remaining) $T_E$ value of the existing pavement. This method first evaluates the equivalent thickness of the asphalt concrete of the existing pavement ($T_E$) according to its damage condition as shown in TABLE 4 and TABLE 5. Then it calculates the bearing capacity of the subgrade soil (design CBR value) and the equivalent thickness of asphalt concrete necessary to carry the future traffic volume of heavy vehicles ($T_A$) from TABLE 2, thus obtaining the thickness of asphalt mixture to be overlaid using the following equation:

$$\text{Overlay thickness (cm)} = T_A - T_E \tag{6}$$

### Mechanistic-Empirical Overlay Design Method

The wide use of non-destructive testing of highway and airfield pavements is expected to encourage expressways agencies to use mechanistic methods in the design of asphalt concrete overlay thicknesses to complement empirical methods as the one shown above. Falling weight deflectometer (FWD) is used in most of these non-destructive tests. Deflection results obtained from these tests are used in numerical back-calculation procedures to obtain pavement layer moduli. Inoue provided the relationships between layer moduli and the equivalent layer coefficients in the $T_A$ method for pavement materials following extensive laboratory tests, FWD tests and elastic multi-layer analysis as indicated in his dissertation. By using Inoue's method, equivalent layer coefficients are evaluated to find effective structural capacity of the pavement system in terms of full depth asphalt concrete thickness, $T_E$.

#### *Back-calculation Procedure*

Various evaluation methods have been developed, many of which are based on static approach. There are two commonly used back-calculation programs in Japan, which are LMBS and BALM (*4*, *5*). The latter program will be described here because the authors have developed it.

Back-calculation is more commonly called inverse analysis in other disciplines. Various methods have been proposed such as extended Kalman filter, dynamic programming, various optimization algorithms, Gauss-Newton method and more recently Monte Carlo filter. FWD users group in Japan has tested many of these and found Gauss-Newton based method more suitable than others as long as back-calculation of pavement layer stiffness is concerned. As an optional function, BALM can provide a confidence region of layer modulus and important strains for estimating design life with the use of Monte Carlo simulation (*6*).

A problem of numerical instability is inherent in inverse analysis and the Gauss-Newton method for pavement problems can not avoid it either. The effect of instability is amplified due to errors in measurement, modeling and model parameters (*7*, *8*). Measured deflections cannot be free from errors. Modeling error refers to the error that is produced by modeling pavement as an elastic multi-layered system. Model parameter errors are generated by errors contained in a layer thickness and Poisson's ratio. Some regularization techniques must be introduced to control the growth of errors. The regularization is well presented in Hansen and O'leary (*9*). In order to achieve computational stability, truncated singular value decomposition is implemented in BALM.

Yamanokuchi and Inoue (*10*) presented a method of overlay design for asphalt pavement. The basic concept presented in this paper is adopted in the current Japanese pavement design and construction code.

#### *Truncated Singular Value Decomposition Approach*

Let $\mathbf{u}(\mathbf{X})$and $\mathbf{u}^*$ be $N \times 1$ vectors of computed and measured responses. Using Taylor series expansion, $\mathbf{u}(\mathbf{X}+d\mathbf{X})$ can be written as:

$$\mathbf{u}(\mathbf{X}+d\mathbf{X}) = \mathbf{u}(\mathbf{X}) + \frac{\partial \mathbf{u}}{\partial \mathbf{X}} d\mathbf{X} \tag{7}$$

in which **X** is an $M \times 1$vector of unknown parameters. Equating Equation (7) to measured response, $\mathbf{u}^*$ one can write as:

$$\mathbf{A}\, d\mathbf{X} = \mathbf{b} \tag{8}$$

where

$$\mathbf{A} = \left[ \frac{\partial u_i}{\partial X_j} \right] \tag{9}$$

$$\mathbf{b} = \left\{ u_i^* - u_i \right\} \tag{10}$$

$$d\mathbf{X} = \left\{ dX_j \right\} \tag{11}$$

**A** is an $N \times M$ matrix, **b** is an $N \times 1$ vector and d**X** is an $M \times 1$ vector. Equation (8) is called a normal equation. **A** refers to sensitivity of surface deflections with respect to unknown parameters. Assuming an initial value of, **X** Equation (8) can be solved for d**X** with the use of singular value decomposition. It has to be solved with care because of its unstable nature. **A** can be decomposed and written as:

$$\mathbf{A} = \mathbf{U}\mathbf{D}\mathbf{V}^T \tag{12}$$

where **D** is a diagonal matrix composed of diagonal elements called singular values. Since;

$$\mathbf{U}^T\mathbf{U} = \mathbf{V}\mathbf{V}^T = \mathbf{I} \tag{13}$$

then, the solution **dX** can be written as:

$$d\mathbf{X} = \mathbf{V}\mathbf{D}^{-1}\,\mathbf{U}^T\,\mathbf{b} = \sum_{j=1}^{M} a_j\, v_j \tag{14}$$

in which $v_j$ is a j-th column vector of $\mathbf{V}$, $a_j = \sum_{i=1}^{N} U_{ij} b_i / d_{jj}$, $\mathbf{U} = \lfloor U_{ij} \rfloor$, $\mathbf{b} = \{b_i\}$ and d**X** refers to a vector of modification and is called upgrade formula for unknown parameters **X**. The surface deflections are computed by AAMES (11) and their sensitivities are obtained by using AAMES with a finite difference method.

*Devices and Functions in BALM*

If a diagonal element $d_{jj}$ in Equation (14) is smaller than a threshold value, then $1/d_{jj}$ is taken as zero in the computation of d**X**. This operation is called regularization. Since back-calculation is iterative and unstable in nature, it is not possible to achieve convergence without some form of regularization. Various regularization techniques have been proposed for back-calculation problem in the past. Truncated singular value method is adopted in BALM. A key point of this method is the selection of the threshold value, which has to be determined carefully.

Scaling of unknown parameters is introduced in order to improve convergence. All unknown parameters are set to an order of one such as $X_j = E_j/E_{j0}$, where $E_j$ is an estimate of j-th layer modulus and $E_{j0}$ is its corresponding seed value. Another device introduced in BALM to reduce computational instability is fixing of parameter value during back-calculation. In case of M layer model, up to M-1 parameters can be fixed and the remaining parameter values can be identified.

Layer interface is assumed fully bonded in most flexible pavement when back-calculation is carried out. However, in case of concrete pavement, interface between concrete and base course is not fully bonded. It is also known that surface deflections increase considerably, if the interface is smooth. Thus, interface slip is implemented in BALM following successful development of AAMES.

*Reliability Analysis*

FWD tests were conducted 50 times at a site arranged by Japan FWD users group. Mean, standard deviation as well as coefficient of variation of the measured deflection data were determined as shown in TABLE 6, while correlation matrix of the deflection data is given in TABLE 7. Pavement deflections, although provide valuable information regarding structural strength of the pavement, are highly dependent on loading mode and magnitude as well as the precision of the FWD device (12, 13, 14). This means that coefficients of variation of data measured by different FWD devices may be quite different. FWD deflection data are sometimes well correlated as shown in TABLE 7.

If mean values, standard deviation and/or correlation matrix of the measured data are known, one can generate a number of deflection data using Monte Carlo simulation. In this research, 1,000 deflection data sets with similar correlation to the above FWD data were generated. 200 mean deflection data sets were obtained by taking averages of 5 deflection data. Back-calculation analysis of these 200 data sets yielded 200 sets of layer moduli. These results are summarized in TABLE 8. Based on mean and standard deviation and taking into consideration normal distribution pattern of these results, confidence intervals were computed and are given in the same table.

After layer moduli were determined, horizontal strains at the bottom of surface layer and vertical strains at the top of bottom layer were be computed. Since strains were computed from back-calculated layer moduli, they were also normally distributed. This implies that their confidence regions (15) could be given by the following equation:

$$(\varepsilon-\overline{\varepsilon})^{T}\Sigma^{-1}(\varepsilon-\overline{\varepsilon})\leq\chi^{2}_{\alpha,2} \tag{15}$$

where $\varepsilon$ = a vector composed of strains $\varepsilon_x$ and $\varepsilon_z$
$\overline{\varepsilon}$ = a vector composed of mean values of $\varepsilon_x$ and $\varepsilon_z$
$\Sigma^{-1}$ = inverse of covariance matrix
$\chi^{2}_{\alpha,2}$ = chi-squared distribution of confidence limit of 100(1-$\alpha$)% and a number of degrees of freedom equal to the number of variables (=2)

These results are plotted in FIGURE 2 showing 50% and 95% confidence regions.

Furthermore, in the Japanese pavement design and construction code, the model for asphalt concrete pavement that relates number of load repetitions to failure, $N_f$, to the horizontal tensile strain, $\varepsilon_x$, at the bottom of the asphalt concrete layer that was developed by Asphalt Institute (AI) after extensive laboratory testing and correlations with field observations (16) is given as:

$$N_f = S_A\left\{18.4*(10^{M})\times(6.167\times10^{-5}\times\varepsilon_x^{-3.291}\times E_1^{-0.854}\right\} \tag{16}$$

$$M = 4.84\times\left(\frac{V_b}{V_v+V_b}-0.69\right) \tag{17}$$

where $S_A$ = coefficient for a defined percent fatigue cracking (e.g. $S_A$ = 1 when percentage cracking is 20%)
$E_1$ = modulus of elasticity of asphalt concrete (MPa)
$V_v$ = percentage volume of air voids
$V_b$ = percentage volume of asphalt
$\varepsilon_x$ = magnitude of tensile strain at bottom of AC layer

For typical values of $V_v$ = 4.4 percent and $V_b$ = 12.5 percent, then M = 1.74.

Similarly, AI's permanent deformation model assumes that rutting occurs in the subgrade while neglecting rutting attributed to other pavement layers. In this regard, the model relates the vertical compressive strain at the subgrade surface, $\varepsilon_z$, to the number of load repetitions to failure due to permanent deformation, Np, according to the following equation:

$$N_p = 11.365 \times 10^{-9} \times \varepsilon_z^{-4.477} \tag{18}$$

These two models have been included in the Japanese new pavement design and construction code. FIGURE 3 shows a plot of the computed number of load repetitions to failure using the above strain values. This plot was obtained in the manner similar to the plot for strains at the bottom of asphalt concrete layer and on the surface of the subgrade layer. FIGURE 2 and FIGURE 3 give a very good indication of the effectiveness of Monte Carlo simulation in reducing variation of backcalculation results. Furthermore, strains and load repetitions that were obtained by the direct use of back-calculated layer moduli are appear to be very reliable.

*Computing $T_E$ using BALM*

After Back-calculating layer moduli, effective full-depth asphalt concrete thickness shown in TABLE 9 were computed by using equations, which were developed by Inoue. These equations were developed following an extensive laboratory tests, FWD tests as well as elastic multilayer analysis. Layer coefficient for asphalt concrete:

$$a_1 = 0.99 \times \log E_{ac} - 2.36 \tag{19}$$

where: $a_1 \leq 1.2$

Layer coefficients for the base and sub-base courses (bituminous treated base, granular base, and sub-base):

$$a_{2,3} = 0.35 \times \log E_{2,3} - 0.384 \tag{20}$$

Whenever necessary, modulus of asphalt concrete at T°C should be converted to obtain equivalent modulus at 20°C using the following relationship that was derived from Inoue's work:

$$E(20°C) = E(T°C) \times 10^{-0.0434(20-T)} \tag{21}$$

In this research, however, no temperature conversion was performed on the back-calculated asphalt concrete modulus results, because only one measurement point was evaluated and the temperature was considered constant.

## DISCUSSION OF RESULTS AND CONCLUSIONS

Successful development of AAMES computer program for multi-layered elastic system encouraged the authors to extend it to back-calculation analysis and overlay design. Both the back-calculation method developed and its application to overlay design were described above. However this method is still under evaluation to link results from back-calculation to overlay design of asphalt pavement.

The following conclusions can be made from our work in Japan.

1. The Gauss-Newton method is found to be useful for back-calculation analysis. However, since back-calculation is highly unstable, regularization must be implemented in the software.
2. Monte Carlo simulation can be used to improve accuracy of the measured data by producing a bigger number of deflection data. This will, however, increase computation time.
3. Reliability analysis can be performed on the back-calculated layer moduli, the corresponding strains, $\varepsilon_x$ and $\varepsilon_z$, as well as the number of load repetitions to failure.

4. Back-calculated layer moduli can be used to obtain effective full depth asphalt concrete thickness, $T_E$. This is a measure of structural capacity of the pavement system and could complement the existing empirical method during maintenance and rehabilitation decision making process.

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## REFERENCES

1. Japan Road Association. *Pavement Design and Construction Code*. 2001. (in Japanese).
2. Inoue, T. *Study on Evaluation of Bearing Capacity and Rehabilitation Design for the Asphalt Pavement*. Ph.D. Dissertation. Waseda University. Japan. 1994. (in Japanese)
3. AASHTO. *Guide for Design of Pavement Structures*. Washington, D.C. 1993.
4. Matsui, K., Inoue, T. and Sanpei, T. *Development of Analytical Method for Stiffness Estimation Pavement System*. JSCE. No.420/V-13. pp.17-25. 1990. (in Japanese)
5. Matsui, K., Kurobayashi, I. and Nishiyama, T. *Effort for Improving Accuracy of Pavement Layer Moduli Estimated from FWD Test Data*. Journal of Pavement Engineering. JSCE. Vol. 3. pp.39-47. 1998. (in Japanese)
6. Maina, J.W., Yokota, H., Matsui K and Kurobayashi, I. *Influence of Correlation among Deflection Data on the Back-calculation Results*. JSCE. No.648/V-47. pp.243-252. 2000.
7. Maina, J.W., Yokota, H. and Matsui K. *Effect of Errors in Layer Thickness on Back-calculated Layer Moduli*. Journal of Pavement Engineering. JSCE. Vol. 3. pp.39-47. 1998.
8. Matsui, K. Kasahara, A. and Okada, K. *Effect of Errors in Deflection Readings and Pavement Model on Back-calculated Layer Moduli*. JSCE. No.526/V-29. pp.55-6. 1995. (in Japanese)
9. Hansen, P.C. and O'leary, D.P. *The Use of the L-curve in the Regularization of Discrete Ill-Posed Problems*. SIAM Journal of Scientific Computations. Vol.14. No.6. pp.1487-1503. 1993.
10. Yamanokuchi, H. and Inoue, T. *Asphalt Pavement and Overlay Design for Heavy Traffic Road in Japan*. 2nd international symposium on pavement evaluation and overlay design. Brazilian Paving Association. pp.2.6.0–2.6.21. 1989.
11. Matsui, K., Maina, J. and Inoue T. *Axi-symmetric Analysis of Elastic Multi-layer System Considering Interface Slip*. IJP Vol.1. No.1. pp.55-66. 2002.
12. Hoffman, M. S., and Thompson, M. R. *Comparative Study of Selected Nondestructive Testing Devices*. Transportation Research Record 852. pp.32-41. 1982.
13. Thollen, O., Sharma, J., and Terrel, R. L. *Comparison of Falling Weight Deflectometer with Other Deflection Testing Devices*. Transportation Research Record 1007. pp.20-25. 1985.
14. Bentsen, R. A., Nazarian, S., and Harrison, J. A. *Reliability Testing of Seven Nondestructive Pavement Testing Devices*. Nondestructive Testing of Pavements and Backcalculation of Moduli. ASTM STP 1026. A. J. Bush III and G. Y. Baladi, Eds. American Society of Testing and Materials. Philadelphia. pp.41-58. 1989.
15. Tokumaru, H., Soeda, T., Nakamizo, T., and Akizuki, K. *Computation and Measurement: Random Data Processing, Its Theory and Application*. Baifuukan Co. 1982. (in Japanese)
16. Asphalt Institute. *Research and development of the Asphalt Institute's thickness design manual (MS-1)*. 9th edition. Research report 82-2. 1982.

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TABLE 1 Traffic volume classification

| Class | One-way Daily Traffic Volume of Heavy Vehicles, T |
|---|---|
| 1 | T < 100 |
| 2 | 100 ≤ T < 250 |
| 3 | 250 ≤ T < 1,000 |
| 4 | 1,000 ≤ T < 3,000 |
| 5 | 3,000 ≤ T |

TABLE 2 Target value for $T_A$

| Design CBR | Target Value (cm) | | | | |
|---|---|---|---|---|---|
| | Class 1 | Class 2 | Class 3 | Class 4 | Class 5 |
| | $T_A$ | $T_A$ | $T_A$ | $T_A$ | $T_A$ |
| 3 | 15 | 19 | 26 | 35 | 45 |
| 4 | 14 | 18 | 24 | 32 | 41 |
| 6 | 12 | 16 | 21 | 28 | 37 |
| 8 | 11 | 14 | 19 | 26 | 34 |
| 12 | - | 13 | 17 | 23 | 30 |
| 20 | - | 11 | 15 | 20 | 26 |

Note 1 $T_A$ represents the pavement thickness required if the entire depth of the pavement were to be constructed of hot asphalt mixtures, used for the binder and surface courses

Note 2 In the case of a road with various CBR values in the vertical direction, a filter course need not be constructed, provided the CBR value of the uppermost layer is 3 or more, and its thickness is 30 cm or more, even if the design CBR value is 2.

TABLE 3 Equivalent layer coefficient for $T_A$ calculation

| Pavement Course | Method and Material of Construction | Conditions | Coefficient, $a_n$ |
|---|---|---|---|
| Surface and binder course | Hot asphalt mix for surface and binder course | | 1.00 |
| Base | Bituminous stabilization | Hot-mixed stability: 3.43 kN or more | 0.80 |
| | | Cold-mixed stability: 2.45 kN or more | 0.55 |
| | Cement & asphalt stabilization | Unconfined compression strength: 1.5 ~ 2.9 MPa<br>Linear deformation: 5 ~ 30 (1/100 cm)<br>Percentage residual strength: 65 % or more | 0.65 |
| | Cement stabilization | Unconfined compression strength (7days): 2.94 MPa | 0.55 |
| | Lime stabilization | Unconfined compression strength (10days): 0.98 MPa | 0.45 |
| | Crushed stone for mechanical stabilization | Modified CBR value: 80 or more | 0.35 |
| | slag for mechanical stabilization | Modified CBR value: 80 or more | 0.55 |
| | Hydraulic slag | Unconfined compression strength (14days): 1.2 MPa or more | 0.55 |
| Sub-base | Crusher-run, slag, sand, etc. | Modified CBR value:<br>30 or more<br>20 to 30 | <br>0.25<br>0.25 |
| | Cement stabilization | Unconfined compression strength (7days): 0.98 MPa or more | 0.25 |
| | Lime stabilization | Unconfined compression strength (7days): 0.7 MPa or more | 0.25 |

TABLE 4 MCI and its corresponding maintenance level

| MCI | Damage level | Necessary level of maintenance |
|---|---|---|
| $5 \leq$ MCI < 10 | 1 | Routine maintenance |
| $3 \leq$ MCI < 5 | 2 | Minor repair/maintenance |
| MCI < 3 | 3 | Major repair/rehabilitation |

TABLE 5 Equivalent layer coefficient for $T_E$ calculation

| Pavement Course | Components of Existing Pavement | Conditions of Each Layer | Coefficient | Remarks |
|---|---|---|---|---|
| Surface and binder course | Hot asphalt mix for surface and binder course | *C | 0.90 | *A |
| | | *D | 0.85 ~ 0.60 | |
| | | *E | 0.50 | |
| Base | Hot asphalt treated base | | 0.80 ~ 0.40 | *B |
| | Cement & asphalt stabilized base | | 0.65 ~ 0.35 | |
| | Cement stabilized base | | 0.55 ~ 0.30 | |
| | Lime stabilized base | | 0.45 ~ 0.25 | |
| | Hydraulic mechanically stabilized slag | | 0.55 ~ 0.30 | |
| | Crushed stone for mechanical stabilization | | 0.35 ~ 0.20 | |
| Sub-base | Pit-in gravel and crusher-run | | 0.25 ~ 0.15 | |
| | Cement and lime stabilized base | | 0.25 ~ 0.15 | |
| Concrete slab | Cement concrete slab | *F | 0.90 | |
| | | *E | 0.85 ~ 0.50 | |

*A: Range of values: the maximum value should be selected for the condition of damage level 1, the minimum value for the condition level 3 and appropriate intermediate values for other conditions

*B: The maximum value is allocated to the road condition equivalent to that of a newly constructed road and other values are selected according to conditions of damage

*C: Damage condition is level 1 and has a risk to proceed to level 2

*D: Damage condition is level 2 and has a risk to proceed to level 3

*E: Damage condition is level 3

*F: Damage condition is level 1 or 2

Note: Refer to TABLE 4 for damage levels

TABLE 6 Measured deflection, standard deviation and coefficient of variation

| | $D_0$ | $D_{20}$ | $D_{45}$ | $D_{60}$ | $D_{90}$ | $D_{150}$ |
|---|---|---|---|---|---|---|
| Mean deflection (μm) | 1033 | 815 | 495 | 378 | 243 | 136 |
| Standard Deviation (μm) | 14.4 | 12.2 | 3.4 | 3.7 | 3.5 | 2.6 |
| Coefficient of Variation | 0.014 | 0.015 | 0.007 | 0.010 | 0.014 | 0.019 |

TABLE 7 Correlation matrix of deflection test results

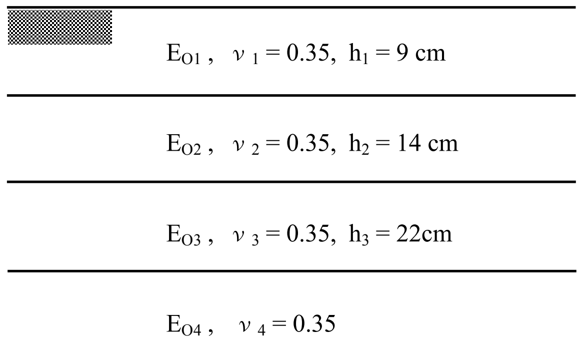
| | $D_0$ | $D_{20}$ | $D_{45}$ | $D_{60}$ | $D_{90}$ | $D_{150}$ |
|---|---|---|---|---|---|---|
| $D_0$ | 1 | 0.921 | 0.862 | 0.672 | 0.378 | 0.093 |
| $D_{20}$ | | 1 | 0.851 | 0.678 | 0.295 | 0.073 |
| $D_{45}$ | | | 1 | 0.685 | 0.346 | 0.264 |
| $D_{60}$ | | | | 1 | 0.369 | 0.157 |
| $D_{90}$ | | | | | 1 | 0.327 |
| $D_{150}$ | | | | | | 1 |

TALBE 8 Mean back-calculation results using FWD deflection data

| | $E_1$ | $E_2$ | $E_3$ | $E_4$ |
|---|---|---|---|---|
| Mean Layer Moduli (MPa) | 3252 | 164 | 56 | 65 |
| Standard Deviation (MPa) | 182 | 23 | 2 | 0.5 |
| Coefficient of Variation (%) | 5.6 | 13.8 | 3.7 | 0.8 |
| Confidence Interval ( 95% ) | [3226, 3277] | [161, 167] | [55.7, 56.2] | [64.5, 64.7] |

TALBE 9  Mean layer coefficient and $T_E$ value

| | $a_1$ | $a_2$ | $a_3$ | $T_E$ (cm) |
|---|---|---|---|---|
| Mean layer coefficients & $T_E$ | 1.12 | 0.39 | 0.23 | 19.45 |
| Standard Deviation | 0.024 | 0.018 | 0.0074 | 0.13 |
| Coefficient of Variation (%) | 2.19 | 4.53 | 3.26 | 0.68 |




($E_{Oi}$ , $\nu_i$ , $h_i$ = Young's modulus, Poisson's ratio, and thickness of layer i)

FIGURE 1  Pavement profile

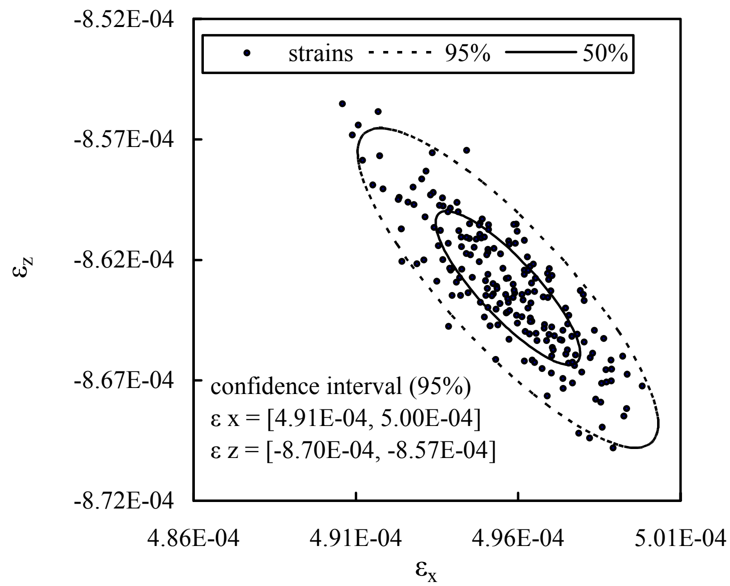



FIGURE 2 Confidence regions for strains

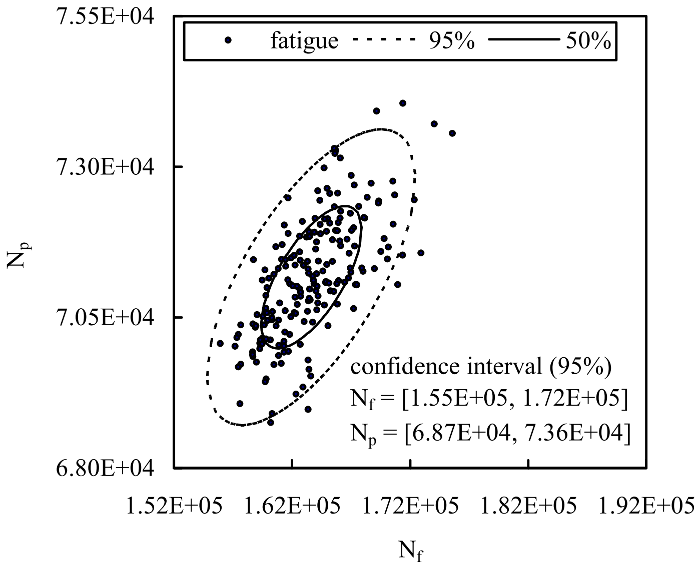



FIGURE 3 Confidence regions for fatigue loads