SERVICEABILITY BASED CONCRETE PAVEMENT DESIGN : JSCE METHOD

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INTRODUCTION

Since its inauguration in 1991, the Pavement Group of the Concrete Standard Specifications Revision Subcommittee of the Concrete Committee in the Japan Society of Civil Engineers (JSCE) has worked to prepare serviceability-based model codes for the design and construction of concrete pavement for any facility, including roads, airports, and yards. In March 1996, JSCE published the Pavement Volume of the Concrete Standard Specifications, providing standards for both design and construction. The principal features of the Specification are as follows.

- It uses a limit state design method related to pavement serviceability.
- Both the amplitude and frequency of loads are utilized as design input values.
- Flexural fatigue cracking can be allowed if necessary.
- It incorporates rules for continuously reinforced concrete pavement, prestressed concrete pavement, and roller compacted concrete pavement.

SERVICEABILITY LIMIT

Table 1 presents a comparison between the contents of the new and old editions of the Pavement Volume of the JSCE Concrete Standard Specifications. The inclusion of the detailed description on concrete pavement structural designing changed the contents of the new edition drastically. In addition, four types of pavement are now addressed: jointed plain concrete pavement, continuously reinforced concrete pavement, prestressed concrete pavement, and roller compacted concrete pavement.

Conventional concrete pavement design methods basically involve setting a design life and determining the slab thickness that would prevent flexural cracking during that period. Due to its simplicity and clarity, this approach has occupied the mainstream of concrete pavement design in many countries. But in fact, it is difficult to define pavement failure only in terms of cracking. For example, there have been cases where the required pavement serviceability was maintained even after flexural cracking occurred, while other joints have occasionally been damaged before flexural cracking began.

Pavement damage that exceeds the allowable serviceability limit could be caused by the onset of cracking, the number of cracks increasing, the crack width increasing, the appearance of joint faulting, erosion caused by water intrusion through these cracks and joints, and other phenomena. Thus, it is necessary to forecast when problems will appear and to estimate their severity to ensure that the required pavement serviceability is satisfied throughout the design period. However, this currently cannot be done analytically. Therefore, serviceability limit state concepts were incorporated into the design method stipulated in the Pavement Volume by adopting both analytical and empirical methods.

To define the required serviceability, we identify the deformation of subgrade and subbase, the occurrence of flexural fatigue cracking, riding comfort, and traveling safety. We set two levels, I and II, as serviceability limit states. Serviceability limit state I is the limit state where essential functions are no longer provided, while serviceability limit state II is where intermediate functions are no longer satisfied. This design method covers the period to serviceability limit state II.

DESIGN PRINCIPLES

In the basic approach to the design of concrete pavement, the serviceability limit state must first be selected according to the required level of function. The limit states on deformation of subgrade and subbase, flexural fatigue cracking of concrete slabs, riding comfort, and traveling safety within the design life must then be examined. The Specification requires that construction work is properly conducted and maintenance works, such as the waterproofing of cracks and the injection of new joint sealant, are carried out properly as necessary.

Serviceability Limit State

As shown in Figure 1, the serviceability limit state is categorized into serviceability limit state I and serviceability limit state II. After the pavement condition reaches serviceability limit state II, the pavement gradually loses its function and only performs at a much lower level.

There are three cases for setting the design life. The selection of the case is left to the pavement designer.

- Case 1: the design period is set as the number of years until the pavement condition reaches serviceability limit state I.
- Case 2: the design period is set as the number of years until the pavement condition reaches serviceability limit state II, and serviceability limit state I is not considered.
- Case 3: the design period is set as the number of years until the pavement condition reaches serviceability limit state II, and serviceability limit state I is also considered.

Table 2 summarizes the pavement limit states and their corresponding inspection items.

Deformation of Subgrade and Subbase

If the subgrade and subbase deforms excessively at an early stage due to large stresses, it will lose its bearing capacity, which must be maintained throughout the design period as a precondition. As a result, the concrete slab will be overstressed, causing early deterioration of the pavement. However, prediction of the amount of subgrade and subbase deformation caused by the cyclic loads over time is difficult. Therefore, the approach taken in the Specification is to keep the design deflection of the pavement at or below the limit value and thereby maintain the bearing capacity of the subgrade and subbase throughout the design period (Equation 1).

$$\gamma_i w_d / w_{dl} \leq 1.0 \qquad (1)$$

where, w_d : design deflection $(=\gamma_a w)$,

w : deflection at the maximum value of design load F_d,

γ_a : structure analysis factor that shall be determined in consideration of the uncertainty in structural analysis (1.0 can be used),

w_{dl} : limit value on design deflection $(= w_e / \gamma_b)$,

w_e : limit value on deflection,

γ_b : component factor that shall be determined in consideration of the seriousness of constructional error in pavement components (1.0 can be used), and

γ_i : importance factor that shall be selected between 1.0 and 1.2 in consideration of importance of pavement.

Table 3 shows the limit values on deflection.

Flexural Fatigue Cracking

Flexural fatigue cracking is evaluated by calculating the cumulative fatigue damage caused by repetitive loading based on Miner's hypothesis, and comparing it to the limit values on the design fatigue damage (Equation 2). In addition, this limit state can be checked by using the design flexural fatigue strength (Equation 3).

$$\gamma_i FD_d / FD_{dl} \leq 1.0 \quad (2)$$

where, FD_d : design fatigue damage $(=\gamma_a FD)$,
γ_a : structure analysis factor,
FD : fatigue damage,
FD_{dl} : limit value on design fatigue damage,
γ_b : component factor, and
γ_i : importance factor.

$$\gamma_i \sigma_{rd} / (f_{rd} / \gamma_b) \leq 1.0 \quad (3)$$

where, σ_{rd} : design variable stress,
γ_i : importance factor,
f_{rd} : design fatigue strength $(= f_{rk} / \gamma_m)$,
f_{rk} : characteristic value of fatigue strength that tested strength is guaranteed to be below,
γ_m : material factor that shall be selected between 1.0 and 1.3 in consideration of the unfavorable variations of material strength, and
γ_b : component factor.

Riding Comfort

Riding comfort varies with the type of vehicle, its speed, and the pavement surface condition. Evaluations of riding comfort vary from passenger to passenger. This means that an objectively quantified index, by which the pavements of streets, highways, airports, and all others are evaluated identically, is necessary when setting a limit state on riding comfort. As a typical index such as PSI (Present Serviceability Index) developed by AASHTO is not suitable for this purpose, the Specification stipulates that pavement be evaluated using IRI (International Roughness Index), which can account for traveling speed (Equation 4), instead (*I*). The limit values on IRI are shown in Table 4.

$$\gamma_i I_d / I_{dl} \leq 1.0 \quad (4)$$

where, I_d : design IRI $(=\gamma_a I(FT_d))$,
$I(FT_d)$: IRI in case of design fault FT_d and distance between joints and cracks,
γ_a : structure analysis factor,
I_{dl} : limit value on design IRI $(= I_l / \gamma_b)$,
I_l : limit value on IRI, and
γ_b : component factor.

The Specification also identifies the cases where the riding comfort must be evaluated from the viewpoint of the amount of faults at joints and cracks (Equation 5). This approach is provided because the IRI cannot be used to evaluate riding comfort on continuously reinforced concrete pavement or prestressed concrete pavement; the IRI becomes smaller in case of larger joint faults as the joint spacing ranges from several tens of meters to a few thousand meters. Table 5 shows the limit values for this case.

$$\gamma_i FT_d / FT_{dl} \leq 1.0 \quad (5)$$

where, FT_d : design fault $\left(= \gamma_a FT(F_d)\right)$,
$FT(F_d)$: fault in case of design load F_d,
γ_a : structure analysis factor,
FT_{dl} : limit value on design fault $(= FT_l / \gamma_b)$,
FT_l : limit value on fault,
γ_b : component factor, and
γ_i : importance factor.

Traveling Safety

As IRI-based riding comfort evaluations are affected by the traveling speed of a moving load, there might be cases in which the limit state for riding comfort is satisfied even with large faults at joints and cracks. If these faults exceed a certain amount, they will damage tires or interfere with vehicle operation. Therefore, in the Specification, the limit state for traveling safety is evaluated by checking the faults at joints and cracks (Equation 6).

$$\gamma_i FTS_d / FTS_{dl} \leq 1.0 \quad (6)$$

where, FTS_d : design fault $\left(= \gamma_a FTS(F_d)\right)$,
$FTS(F_d)$: fault in case of design load F_d,
γ_a : structure analysis factor,
FTS_{dl} : limit value on design fault $(= FTS_l / \gamma_b)$,
FTS_l : limit value on fault,
γ_b : component factor, and
γ_i : importance factor.

To estimate the amount of a fault for evaluating riding comfort and traveling safety, empirical equations derived in the United States are employed (Equations 7 and 8) (*2*). These equations can be used for design, as their applicability to concrete pavement in Japan has been verified. The confirmation process is presented below.

- for doweled joint

$$FT = ER^{0.25} \cdot \left\{ 0.038332 \left(\frac{PRE}{254} \right)^{1.84121} + 0.057763 (3.2808 JS)^{0.38274} \right\} \quad (7)$$

- for undoweled joint

$$FT = ER^{0.25} \cdot \left\{ 0.009759 \left(\frac{PRE}{25.4} \right)^{0.91907} + 0.060291 (3.2808 JS)^{0.54428} - 0.016799 DR \right\} \quad (8)$$

where, FT : faulting at transverse joints (mm),

ER : calculated accumulated erosion (%, $= \sum \{100 \cdot n_i (C_2 / N_i)\}$),

C_2 : 0.06 for pavements without a shoulder and 0.94 for pavements with a tied concrete shoulder,

i : axle-group,

n_i : expected number of axle-load repetitions for axle-group i,

N_i : allowable number of repetitions for axle-group i ($\log N_i = 14.524 - 6.777(C_1 \cdot P - 9.0)^{0.103}$),

$C_1 = 1 - \left(187 \frac{k}{h}\right)^2$,

P : power $\left(= 357481 \frac{w^2 k^{1.27}}{h}\right)$,

w : corner deflection due to axle-group I (mm),

k : modulus of subgrade reaction (N/mm^3),

h : slab thickness (mm),

PRE : annual precipitation (mm),

JS : average joint spacing (m), and

DR : drainage dummy variable, 0 for none, 1 for edge drains.

Data on faults at joints and cracks was collected from national highways and expressways all over Japan. Questionnaires were distributed to obtain information about the maximum fault, year of construction, service years, traffic volume, subgrade and subbase condition, and concrete slab thickness.

We assumed the following conditions for calculating faults based on these equations.

- The foundation reaction coefficient was estimated by accounting for the subgrade and subbase and then performing a multi-layered elastic analysis.
- The joint structures for all sections were assumed to be uniform, and the load transfer efficiency was set at 70%.
- The stress and deflection were calculated using the finite element method.
- Traffic volume was assumed to be that incurred since its service started.
- The amplitude and frequency of loads were assumed properly in accordance with the road pavement design manual in Japan.

- Loads were assumed to be normally distributed in the transverse direction with a standard deviation of about 20%.
- The slab width, joint spacing and drainage conditions were assumed to be constant.
- The average value of annual precipitation between 1961 and 1990 was used.

Figure 2 shows the faults estimated using the equations and those that were measured. Faults greater than 40 mm could not be caused only by deformation of the foundation, but may be caused by excessive settlement of the ground itself. These equations were based on the premise that the fault accumulated due to deflections of the subgrade and subbase by repetitive loading, so they cannot be applied to the case described above. Therefore, it was concluded that, except for faults in excess of 40 mm, the calculated values give the average values of actual faults.

Cracking

The limit value on crack width from corrosion of the steel reinforcement in continuously reinforced concrete pavement and prestressed concrete pavement is assumed to increase with the covering depth. Therefore, in the Specification, this value is specified according to the environmental conditions and the type of steel reinforcement (Table 6).

Design Input

Design data is input for the design of concrete slabs, and the subgrade and subbase. Because the flexural strength of concrete decreases as the specimen height increases, the design flexural strength must be calculated using Equation 9, which can take into account the difference between the slab thickness and specimen height. Furthermore, Equation 10 is used to determine the design flexural fatigue strength of concrete, which can account for its failure probability.

$$f_{bd,h} = 0.8 \times (0.6 + 2.25 / h^{1/4}) \times f_{bd,h0} \quad (9)$$

where, $f_{bd,h}$: design fatigue strength with slab thickness h,

$f_{bd,h0}$: design fatigue strength with specimen thickness h_0, obtained by f_{bk} / γ_c (h_0 is 150mm),

f_{bk} : characteristic flexural strength of concrete, and

γ_c : material factor for concrete that shall be between 1.0 and 1.3.

$$f_{rd} = k_1 f_d (1 - \sigma_p / f_d)\left(1 - \frac{\log N}{K}\right) \quad (10)$$

where, N : fatigue life (N210^6),

f_{rd} : design fatigue strength,

f_d : design strength of concrete, obtained by f_k / γ_c (1.0 is used as γ_c),

K : coefficient (17 is used),

k_1 : coefficient (0.85 is used for flexural compression, 1.0 for tension and flexural tension), and

σ_b : stress due to permanent loads.

In addition to the foundation reaction coefficient, the Specification provides methods for using CBR or resilient modulus as design input values for the subgrade and subbase.

Loads

The loads that act on the concrete pavement are roughly divided into two types according to their frequency, duration and degree of variation. The first are permanent loads such as seasonal temperature variations, shrinkage and creep of concrete, and prestressing force. The others are variable loads such as traffic loads and daily temperature variations. This means that the design load is determined by appropriately combining permanent and variable loads, and setting the values for each according to the limit state and pavement type.

Because a longitudinal axial stress is caused by the temperature drop of a concrete slab in continuously reinforced concrete pavement and prestressed concrete pavement, the average concrete slab temperature must be set appropriately. In addition, the characteristic values for concrete shrinkage and creep must be set properly, as they affect the allowable crack spacing and crack width of continuously reinforced concrete pavement and the stress loss of prestressed concrete pavement.

The variable loads due to traffic and daily temperature variations fluctuate greatly. The design loads cannot be uniquely defined for this reason, but must be determined as the design values of the amplitude and frequency of the loads. The design load amplitude is the product of the maximum value in each load group that the variable loads are divided into, and a load factor for amplitude. On the other hand, the design load frequency is the product of the frequency of each load group as determined from the traffic volume during the design life, and a load factor for frequency. The load factors shall be selected between 1.0 and 1.2 in consideration of their unfavorable variations.

The transverse distribution of the traffic loads must be defined appropriately because the traffic load greatly influences the performance of concrete pavement. When there is a difference in the temperature between the surface and bottom of a concrete slab, the resulting warping stress causes fatigue cracking of the concrete slab by combining with flexural stress due to traffic loads. Therefore, the temperature distribution between the surface and bottom of the concrete slab must be defined for the serviceability limit state under study.

Structural Analysis

Concrete pavement consists of stiff concrete slabs over a subbase with relatively low stiffness. The Specification includes numerical models for conditions such as concrete slabs on both a Winkler foundation and a semi-infinite elastic continuum.

As some load transfer device is installed at the joints, it must be incorporated into the structural design. The Specification provides coefficients that can incorporate the effect of these devices, and the flexural stress and deflection can be reduced from those at a free edge or corner by multiplying them by corresponding coefficient (Table 7) (3).

The Specification permits the use of numerical analysis methods such as the finite element method to analyze nonlinear phenomena due to plastic deformation of the subgrade and subbase, stress caused by restraint against warping, discontinuity at joints and cracks, and so on.

STUDIES OF VARIOUS PAVEMENT TYPES

Jointed Plain Concrete Pavement And Jointed Reinforced Concrete Pavement

The structural design of both jointed plain concrete pavement and jointed reinforced concrete pavement follows the flow chart shown in Figure 3. First, limit states are set for the deformation of the subgrade and subbase, flexural fatigue cracking of the concrete slab, riding comfort and traveling safety.

The pavement structure is then examined to satisfy these limit states. This procedure is basically identical to that used for continuously reinforced concrete pavement, prestressed concrete pavement, and roller compacted concrete pavement.

Continuously Reinforced Concrete Pavement

The structure of continuously reinforced concrete pavement can be determined in accordance with the flowchart shown in Figure 3. However, determining the permissible width of transverse cracks is important to prevent problems of poor load transfer efficiency and corrosion of steel reinforcement. For this reason, the serviceability limit state of this pavement is essentially evaluated in view of transverse crack width. Therefore, Equations 11 through 15 are used to calculate the crack width through the slab thickness, crack spacing and steel reinforcement stress. These equations are simplified versions of the equations developed to account for eccentricity of the steel reinforcement installation, shrinkage and creep of concrete, bond rigidity between steel reinforcement and concrete, and so on (*4*, *5*).

- maximum crack spacing

$$l_{\max} = 2\frac{\eta_1 / \eta_2}{\frac{np}{\rho_u}\left[\frac{E_c(\varepsilon'_{cs,s}+\varepsilon_{\Delta T,s})}{f_{bd,h}} - \frac{\rho_u}{\rho}\right]} \tag{11}$$

- reinforcement stress at cracked section

$$\sigma_s = \frac{f_{bd,h}}{\left[1+\frac{(h/2-d)h/2}{I_c/A_c}\right]\cdot p} \tag{12}$$

- maximum crack width at the depth of reinforcement

$$w = 2\left(1+\frac{np}{\rho}\right)\frac{\eta_1}{\frac{2}{l_{\max}}\eta_1 + \frac{np}{\rho}\eta_2}(\varepsilon'_{cs,s}+\varepsilon_{\Delta T,s}) \tag{13}$$

- maximum crack width on the surface

$$w_u = \frac{1+\frac{np}{\rho_u}}{1+\frac{np}{\rho}}\cdot w + \left[\frac{1-\frac{\rho}{\rho_u}}{1+\frac{np}{\rho}}\cdot\frac{np}{\rho}\cdot\frac{\sigma_s}{E_s} - \frac{1+\frac{np}{\rho_u}}{1+\frac{np}{\rho}}(\varepsilon'_{cs,s}+\varepsilon_{\Delta T,s}) + \varepsilon'_{cs,u}+\varepsilon_{\Delta T,u}\right]\cdot l_{\max} \tag{14}$$

- maximum crack width at the bottom

$$w_l = \frac{1+\frac{np}{\rho_l}}{1+\frac{np}{\rho}} \cdot w + \left[\frac{1-\frac{\rho}{\rho_l}}{1+\frac{np}{\rho}} \cdot \frac{np}{\rho} \cdot \frac{\sigma_s}{E_s} - \frac{1+\frac{np}{\rho_l}}{1+\frac{np}{\rho}} (\varepsilon'_{cs,s} + \varepsilon_{\Delta T,s}) + \varepsilon'_{cs,l} + \varepsilon_{\Delta T,l} \right] \cdot l_{\max} \qquad (15)$$

where, $1/\rho = 1 + \frac{(h/2-d)^2}{I_c / A_c}$,

$1/\rho_u = 1 + \frac{(h/2-d)h/2}{I_c / A_c}$,

$1/\rho_l = 1 - \frac{(h/2-d)h/2}{I_c / A_c}$,

$\eta_1 = L_b + 7.5\phi + 26.6\phi^2 / L_b$,

$\eta_2 = 1 + 6.25\phi / L_b$,

$L_b^2 = \frac{\phi}{4(1+np/\rho)} \frac{E_s}{K_s}$,

$f_{bd,h}$: design flexural strength of concrete (N/mm^2),
E_c : Young's modulus of concrete at 28 days age (N/mm^2),
n : Young's modulus ratio of reinforcement to concrete (= E_s/E_c),
: diameter of reinforcement (mm),
K_s : bond rigidity (N/mm^3),
A_c : area, inertial moment of concrete (mm^2),
I_c : area, inertial moment of concrete (mm^4),
p : steel ratio (= A_s/A_c),
d : distance of reinforcement from the surface (mm),
$\varepsilon'_{cs,s}, \varepsilon'_{cs,u}, \varepsilon'_{cs,l}$: freely contracting strain due to drying shrinkage in concrete at the depth of reinforcement, on the surface, at the bottom, respectively,
$\varepsilon'_{\Delta T,s}, \varepsilon'_{\Delta T,u}, \varepsilon'_{\Delta T,l}$: free strain due to temperature change at depth of reinforcement, on the surface, at the bottom, respectively,
α_c : thermal expansion coefficient of concrete (1/), and
σ_s : reinforcement stress at cracked section (N/mm^2).

In order to simplify these, the effects of concrete creep and the bond rigidity between steel reinforcement and concrete were studied. Figures 4 and 5 show the effects of the creep coefficient and the bond rigidity on the maximum crack spacing. As shown in Figure 4, the maximum crack spacing increases with the creep coefficient. However, ignoring the effect of creep is conservative from the viewpoint of durability because the calculated crack width increases and the crack width at the surface becomes larger than that at the bottom. The measured bond rigidity is about 400 N/mm^3, and it decreases over time under the effect of creep. However, as shown in Figure 5, the bond rigidity has little effect on the maximum crack spacing above 200 N/mm^3. Therefore, the process can be simplified by assuming that the bond rigidity is 400 N/mm^3.

Crack width must be kept at 0.5-0.6mm or less, as both the faulting can be prevented and the load transfer can be provided. In the calculation, both the freely contracting strain due to drying shrinkage and the free strain due to temperature change must be determined properly in consideration of the construction period and the site characteristics. As the freely contracting strain on the surface and at the bottom, 20010^{-6} and 0 might be used respectively, unless any data are available.

Prestressed Concrete Pavement

In Japan, prestress is generally introduced by applying tensile force to the concrete slab using prestressed concrete (PC) reinforcement. The Specification is not applicable for cases using jacks or expansive concrete admixtures to apply prestress.

The structure of PC pavement can be structurally designed by using the flowchart shown in Figure 3. As cracking must not occur on the surface of PC slab in principle, the serviceability limit state is evaluated at the bottom of the slab. There are two cases in designing whether cracking at the bottom of a concrete slab is acceptable or not. In the latter case, the crack width is checked indirectly by specifying a limit value for the increment of PC reinforcement stress of 100 N/mm^2, as

no method for calculating the flexural crack width at the bottom has been established (*6*). In this case, fatigue of the reinforcement is not evaluated because the reinforcement stress is between 60 and 70% of the tensile strength, and its stress fluctuation is small at about 10%.

Deformation of the subgrade and subbase is a particularly important limit state for prestressed concrete pavement (*7*). In cases where flexural cracking at the bottom of the concrete slab is not permissible, the entire cross section can be included in the calculation process. Where flexural cracking is permissible, the reduction of stiffness in the cross section caused by cracking must be checked.

Roller Compacted Concrete Pavement

Roller compacted concrete pavement is basically designed in the same way as jointed plain concrete pavement. However, because the concrete slabs are compacted by rollers, roller compacted concrete pavement differs from other kinds of concrete pavement in that the steel reinforcement, dowel bars and tie bars are difficult to install, and special treatment is necessary to obtain an even surface and uniform density throughout the concrete slab.

Transverse joints over which vehicles pass frequently become the structurally weakest part because dowel bars cannot be used at joints in roller compacted concrete slabs. Therefore, flexural fatigue cracking must be evaluated at transverse joints in the design.

The compaction rate of a roller compacted concrete slab influences the flexural strength. In particular, the low compaction rate in the lower portion cannot be ignored in thicker slab cases. In order to determine the design flexural strength in such cases, an appropriate material factor must be established based on experiments and experience. For example, a material factor between 1.15 and 1.27 can be used for a design flexural strength of 4.5 N/mm^2 when the compaction rate in the lower portion decreases by 3% (*8*).

CONCLUDING REMARKS

The Specification itself contains design, materials, construction and quality control, although this paper only introduces design issues. Because the Specification includes drastic changes to the existing design methods, some problems might be revealed when it is actually used. However, we hope that it is beneficial to refining the current design manuals for roads, airports, yards, and so on.

Some parts of the contents in Design and Construction Volumes of the JSCE Concrete Standard Specification are included in the Pavement Volume. We would like to express our thanks here to the members involved in the process of revising the JSCE Concrete Standard Specification. The members in Pavement Group are as follows: A. Kasahara, R. Sato, Y. Hachiya, M. Ishitani, H. Inukai, K. Otsuka, N. Otsuki, N. Omoto, S. Kameta, H. Kikkawa, M. Koyanagawa, S. Shimeno, R. Tomita, T. Nakamura, T. Nishizawa, M. Mashima, H. Wakui, N. Watanabe, M. Okudaira, K. Kai, M. Kagata and S. Kokubun.

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TABLE 1 Comparison of New and Old Editions

Item	Chapters in	
	Old Edition	New Edition
General	1	1
Design	2	2 - 10
Quality	3	11
Material	4	12
Concrete Production	5	13
Construction	6, 7	14
Concrete in Severe Climate	8, 9	16
Inspection	10	15

TABLE 2 Limit State and Inspection Item

Limit State	Inspection Item	Case			
		1	2	3	
				SLS [a]	
				I	II
Foundation Deformation	Deflection	○	○	○	○
Fatigue	Crack	○	○	○	○
Riding Quality	IRI	○	○	○	○
	Fault at Joint	○	○	○	○
	Fault at Crack		○		○
Traveling Safety	Fault at Joint	○	○	○	○
	Fault at Crack				○
Durability	Crack Width	○	○	○	○

[a] Serviceability Limit State

TABLE 3 Limit Value on Deflection

Road (Edge Loading)	Airport (Interior Loading)
0.75	1.25

(unit: mm)

TABLE 4 Limit Value on IRI

Serviceability Limit State	Facilities				
	Yard, Street	Rural Road	Highway	Expressway	Airport Runway
I	1.5	1.5	1.5	0.5	0.5
II	11.0	7.0	5.0	3.5	1.5

(unit: mm/m)

TABLE 5 Limit Value on Fault for Riding Comfort

Serviceability Limit State	Facilities				
	Yard, Street	Rural Road	Highway	Expressway	Airport Runway
I	15	12	10	5	2
II	20	15	12	10	5

(unit: mm)

TABLE 6 Limit Value on Crack Width

Reinforcement Type	Environmental Conditions		
	Normal	Corrosive	Severely Corrosive
Deformed and Plain Bar	0.005 c [a]	0.004 c	0.0035 c
Prestressing Steel	0.004 c	- [b]	-

(unit: mm)

[a] Concrete Cover (mm)
[b] not specified

TABLE 7 Coefficient for Stress and Deflection Reduction

Position	Load Transfer Efficiency	Stress Reduction Coefficient α	Deflection Reduction Coefficient β Corner with	
			Two Reinforced Edges	One Reinforced Edge
Free Edge Dummy Joint Free Corner	0	1.0	1.0	1.0
Tied Joint	0.80 - 0.95	0.70 - 0.80	0.70 - 0.80	0.85 - 0.95
Joint with Dowel bar	0.70 - 0.90	0.75 - 0.85	0.75 - 0.85	0.85 - 0.95
Crack	0 - 0.40	1.0	- [a]	0.95 - 1.0
Crack (CRCP)	0.95 - 1.0	0.60 - 0.70	0.60 - 0.70	0.80 - 0.90

[a] does not exist

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Figure 2 Comparison of Calculated and Measured Faults

Figure 3 Flowchart of Jointed Plain Concrete Pavement Design

Figure 4 Influence of Creep Coefficient on Crack Spacing

Figure 5 Influence of Bond Rigidity on Crack Spacing

Figure 1 Serviceability Limit State

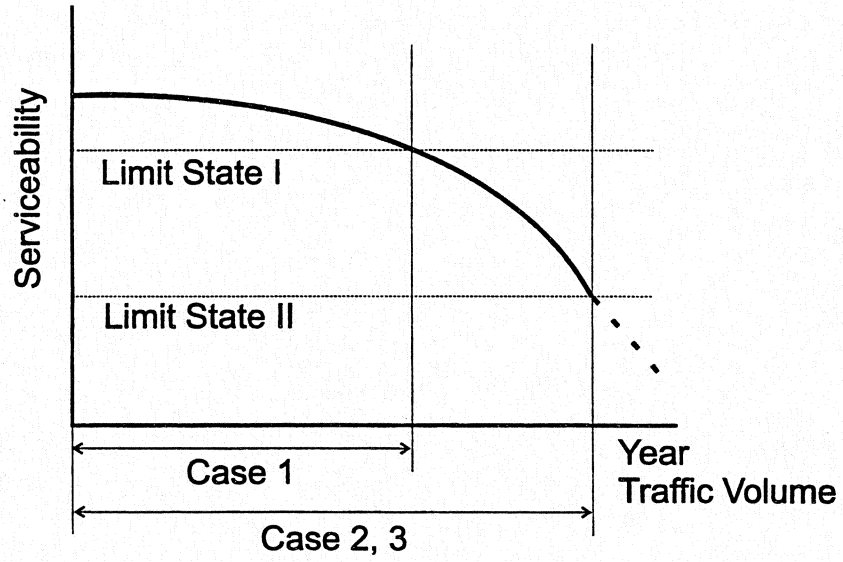


Figure 2 Comparison of Calculated and Measured Faults

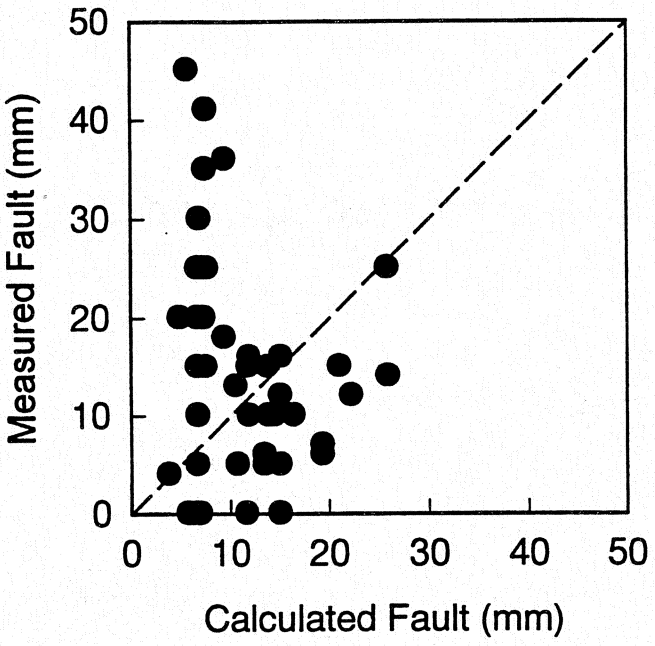


Figure 3 Flowchart of Jointed Plain Concrete Pavement Design

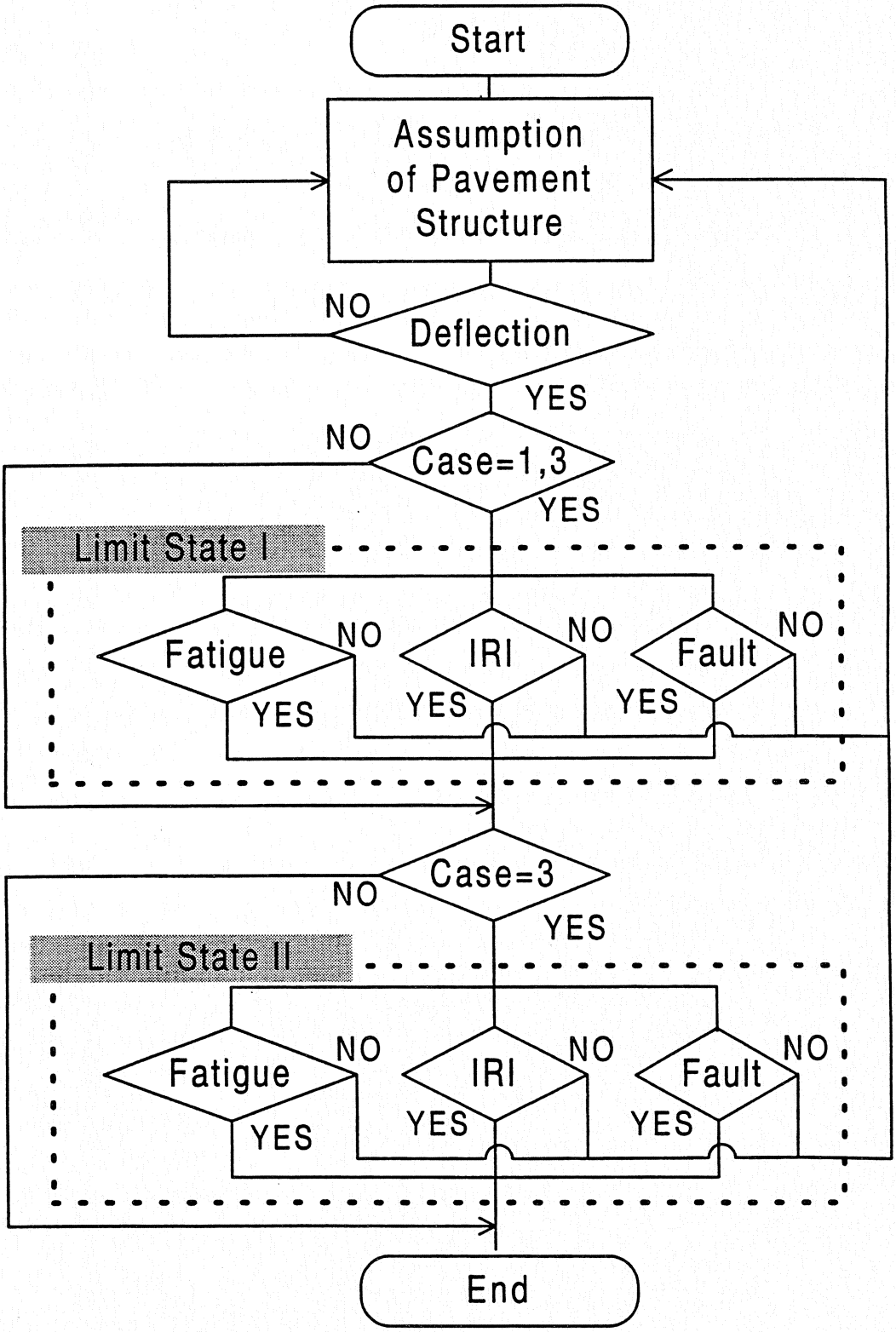


Figure 4 Influence of Creep Coefficient on Crack Spacing

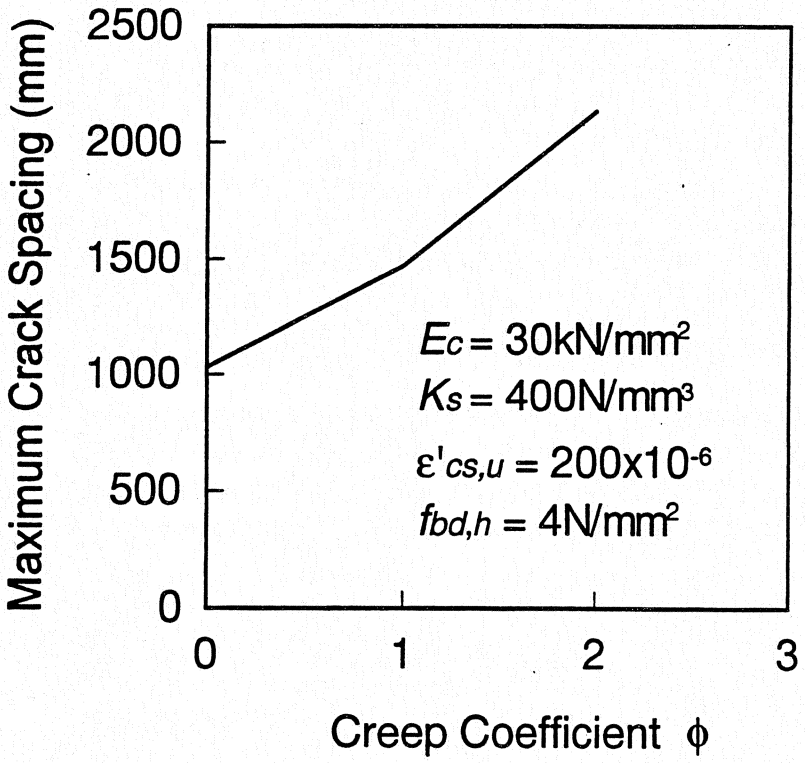
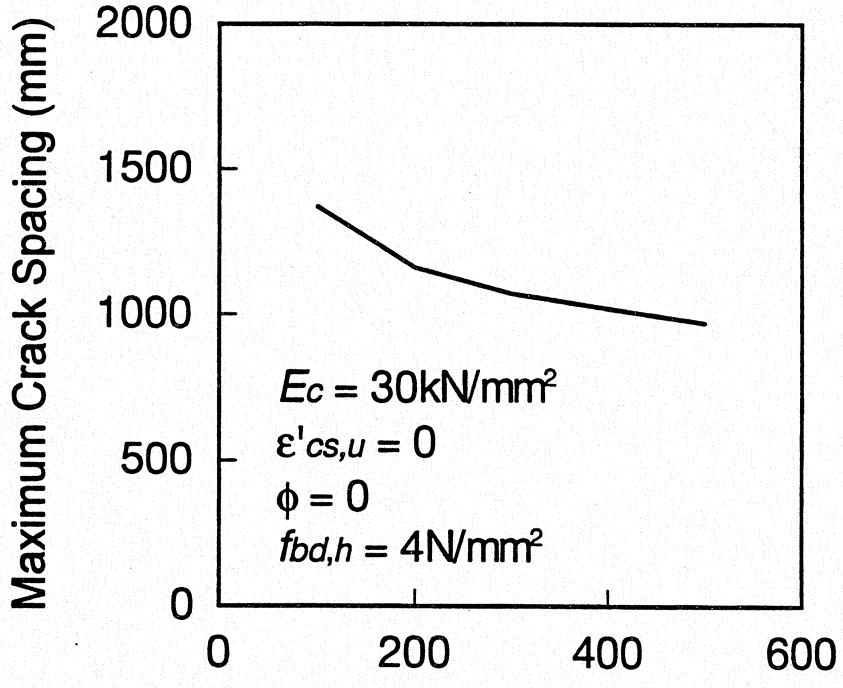


Figure 5 Influence of Bond Rigidity on Crack Spacing



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