FATIGUE AND CORROSION IN CONCRETE DECKS WITH ASPHALT SURFACING

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Abstract

In 20th century, a large number of highway bridge concrete decks in U.S. suffered from corrosion due to deicing salt, but those in Japan suffered from fatigue due to cyclic loading of heavy trucks. Lately, significant corrosion of concrete decks due to deicing salt have been reported in Japan, giving rise to further concerns about combined deterioration from fatigue and corrosion in existing bridge decks designed according to old specifications. This paper provides chloride profiles in concrete decks surfaced with asphalt mixtures, which is one of reasons concerning a significant difference in deterioration between both countries. Additionally, with regard to mechanisms of fatigue deterioration of concrete decks, a current result obtained from the truck wheel traveling tests is presented.

Introduction

In 1960s, construction of interstates began in U.S. But spalling due to corrosion frequently appeared on concrete decks in several years since completion. It was caused by a rapid increase in salt use for deicing in those days shown in Fig. 1. In 1970s, the deterioration affects traffic safety, drivability, traffic congestion during repair, and budget, being a major problem. A large number of deficient bridges due to corrosion still remain as current issues in bridge management in U.S.[1]

In Japan, fatigue due to cyclic loading of heavy trucks is major deterioration of concrete decks shown in Fig. 2 to date. Currently, however, attention should be paid not to only fatigue, but also chloride-induced deterioration on the concrete decks, because salt use have been increasing since spike tires were prohibited in 1990s as shown in Fig. 3.[2, 3] Figure 4 shows chloride-induced deterioration observed on a bridge deck in Japan the late 1990s. While corrosion due to deicing salt is not often observed in national roads and rural roads, significant corrosion on bridge decks in expressway was reported.[4] In addition, most of existing bridges were built from the mid-1950s to the mid-1970s, having lower potential for fatigue durability than current bridges (Fig. 5, 6). The possibility of combined deterioration from fatigue and corrosion should be taken into account in maintenance of existing highway bridges.

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With respect to the reasons why both countries differ in major deterioration of concrete decks, quantity of salt use, traffic loading including illegal vehicles and asphalt surfacing (newly constructed bridges are almost surfaced with asphalt pavement in Japan) contrast apparently (Fig. 7).[2, 3] Chloride ingress from the asphalt surface to concrete decks is probably governed by water. As-built asphalt mixtures should not be expected as waterproofing because of its permeability, whereas it was reported that asphalt surfacing contributed to prevent concrete decks from salt ingress while being imperfect.[3] Additionally water significantly affects fatigue durability of concrete decks.[5] Thus field performance of asphalt surfacing on the decks should be investigated for managing to prevent existing concrete decks from fatigue and corrosion in Japan.

This paper indicates results of field investigations on asphalt surfacing and chloride profiles in decks using two closed bridges; a post-tensioned concrete bridge at coastal area (ATG bridge) and a steel girder bridge with a concrete deck (TYD bridge) subjected to deicing salt. In addition, new results regarding mechanisms of fatigue deterioration in concrete decks are presented for discussing the influence of spalling on fatigue behavior.

Field Investigations of Chloride Ingress under Asphalt Surfacing

Test cores consisting of asphalt mixtures and portland cement concrete of decks were taken from the closed bridges before removing asphalt surfacing (Fig. 8). Content of acid-soluble chloride ions in each slice with 10 mm depth of concrete cores was determined. A percentage of void content of asphalt mixture cores V_v was determined as described below.

$$V_v = 1 - (\gamma_M / \gamma_{max}) \times 100\% \qquad (1)$$

where, γ_M is measured mixture density (= dry weight in air /(wet weight in air - weight in water)); γ_{max} is a percentage of the maximum density. The values of γ_{max} were determined using a fragment of the removed asphalt mixtures according to the test method similar to ASTM D 2041.

Water permeability tests of asphalt mixture cores were conducted under a constant pressure of 150 kPa. Water permeability k was determined from the test results based on Darcy's law as below.

$$k = LQ / (AHT) \qquad (2)$$

where, Q is total discharge (cm^3); H is head (cm, =1 500 cm); T is time (s); A is area of cores (cm^2); L is depth of cores (cm). No leakage of water in 24 hours was defined as impermeable.

In Fig. 9, chloride profiles of cores taken from ATG bridge are shown. The ATG bridge was built at coastal area in 1971, replaced due to corrosion of post-tensioned girders in 2005. No corrosion was observed on the road surface. Curbs were replaced around 1985 for upgrading traffic safety. The thicknesses of asphalt mixtures in the cores ranged from 37 mm to 66 mm. From the figure, it can be seen that chloride contents at the surface of deck concrete were much less than that of a core A-3 taken from the top of curb not covered with asphalt, even though low chloride permeability of prestressed concrete was taken into account. All asphalt cores of ATG bridge were determined as impermeable.

Likewise, results from the investigation of TYD bridge are shown in Fig. 10. The water permeability k is indicated, no indication of k means impermeable except a core T-9 untested. Deicing salt was applied on TYD bridge, being somewhat apart from the sea. The thicknesses of asphalt mixtures in the cores ranged from 44 mm to 71 mm. Chloride ingress of deck concrete was definitely observed at a joint between asphalt mixtures (core T-9). Except the joint, much chloride contents were not observed in the concrete of decks. All asphalt cores taken from TYD bridge were not impermeable. Determined permeability ranged from 1 to 20 × 10^{-7}cm/s except cores near curbs.

Most of asphalt cores taken from existing bridges including 20 bridges investigated previously[3] and the above mentioned ATG and TYD bridges had the void contents V_v of less than 2%, being determined as impermeable. It was reported that the void contents of asphalt mixtures decreased with increasing the service time.[6] With regard to the decrease in the void content in service, compaction due to traffic loading probably influences. But since impermeable areas were not observed only in loading areas determined by ruts, there must be reasons other than the effect of compacting in service, such as melting due to high temperature in summer and clogging. Although waterproofing was installed on the TYD bridge since 1996, a fact that deicing salt had been applied since the day before the installation indicates that the decks was not prevented from chloride ingress by only waterproofing.

Several routes of chloride-contaminated water from the road surface to decks seem to exist depending on depth of asphalt mixtures, magnitude of compaction, finishing at joints of surfacing, damage level and maintenance level of surfacing (Fig. 11). After chloride-contaminated water reach on the concrete decks, chloride ions penetrate into concrete. Chloride penetration in uncracked concrete is ordinarily expressed by Eq. (3).[7]

$$C(x,t) = C_o\left(1 - erf\frac{x}{2\sqrt{D_c t}}\right) \tag{3}$$

where, x is depth from the surface; t is time; C_o is chloride level at the surface of concrete; erf( ) is error function, D_c is diffusion coefficient of chloride ions in concrete.

As shown in Fig. 12, the diffusion coefficient obtained from ponding tests in salt water significantly differs from that obtained from exposure tests in airborne chloride environment at coast.[8] Consider the diffusion coefficient of top cover concrete in decks, while the condition of bare concrete of curbs corresponds to that of the exposure tests, the condition of concrete decks covered with chloride-contaminated water under asphalt mixtures is probably close to that of the ponding tests indicating rapid chloride penetration. In addition, water ingress accelerates disbonding between asphalt surfacing and concrete, providing a fine soil layer in the boundary. The soil layer keeps the surface of concrete wet for long time, forming the condition closer to the ponding tests.

When chloride ingress continues, chloride level at reinforcing steel bars reach the threshold level initiating corrosion of steel in concrete. Expansion of the steel bars due to corrosion yields cracking and spalling of concrete as well as asphalt mixtures.

Curve fitting to Eq. (3) using chloride profiles obtained from decks without asphalt surfacing provides two parameters of the boundary chloride level C_o and the diffusion coefficient D_c. Then exposure time t is simply determined as the term from construction. But the chloride profiles obtained from decks surfaced with asphalt mixtures are not available to determine the diffusion coefficient of the decks because the exposure time governed by quality, durability and maintenance level of asphalt surfacing is unknown.

As an attempt, the exposure time is estimated using the diffusion coefficient predicted from Fig. 12. Since water-cement ratios of old structures are not exactly predictable, average values of reinforced concrete and post-tensioned concrete are roughly assumed as 0.55 and 0.43, respectively. Fig. 13 shows the estimated exposure time of the concrete decks under asphalt surfacing. With regard to the results of wet condition, the estimated exposure time of decks on ATG bridge is only 0.2 to 1 year despite the severe chloride environment at coast and the lifetime of 34 years. In cores taken from TYD bridge, obvious chloride profile was obtained from only core T-9 at a joint of asphalt, while deicing salt was applied for long years. The estimated exposure time at T-9 is 8 years. It cannot be determined whether chloride ingress took place after installing a waterproof layer or before.

Consequently, it was found that although both bridges investigated did not had waterproofing at completion, the expected exposure time was much shorter than the service term. The results indicate that the impermeable or low permeable asphalt mixtures contributed to prevent bridge decks from chloride contamination in a manner.

Mechanisms of Fatigue in Reinforced Concrete Decks under Traffic Loading

Concrete spalling due to corrosion on decks influences fatigue durability of concrete decks. Fatigue mechanisms of reinforced concrete decks subjected to cyclic wheel loads have been studied since 1960s. Some types of truck wheel traveling tests for real-size decks were proposed to simulate real fatigue deterioration in 1980s,[9] contributing various research of bridge decks so far. To identify the influence of spalling on fatigue behavior, a new approach focusing on the change of internal stress with crack propagation was attempted in our study using a specimen N for the test (Fig. 14).[10, 11] From the results, at relatively early cycles, arch action consisting of concrete arch rib and steel ties (lower main steel reinforcing bars) was recognized through a truck wheel traveling area, similarly to a shear resistance mechanism of reinforced concrete beams.

As shown in Fig. 15, cracks were observed in a deck specimen after the truck wheel traveling test. For showing one of the behavior indicating arch action, distributions of axial forces acting on a main tensile reinforcement at the center of the deck are shown in Fig. 16, being measured using strain gages. The distribution at 50 cycles shows typical flexural behavior of a slab. As the number of cycles increases, the distributions changed even through the bar, indicating the behavior of the tie bar. The change was recognized at early stage corresponding to the cycles N_s that an apparent neutral axis calculated using measured strains of upper and lower steel bars turned to increase as shown in Fig. 17.

Similarly, using previous data of 11 specimens,[12] the cycles to the change of the apparent neutral axis N_s were calculated. The cycles N_s corresponded to the cycles to the crack density (crack length per area) reaching 90% of the final crack density, ranging from 0.05 to 0.3 N_f,[9] where N_f is the cycles to failure. All specimens were designed according to the old specifications in 1964 except a specimen having details of 1972 specifications.

Fig. 18 shows the cycles N_s of the 11 specimens and the specimen N with the applied load level normalized by P_{sx}, where P_{sx} is the reduced punching shear strength[5] considering an equilibrium of forces at failure under cyclic loading. From the figure, it was found that the cycles N_s related to developing arch action can also be expressed as a kind of fatigue behavior similarly to N_f.

Fatigue tests in compression using 100 mm dia. cylinders made of the same batch of concrete for the deck specimen N were conducted. A relationship between the measured strain ranges at the initial cycle and the cycles to failure was obtained. From the result, the measured strain ranges of 800 to 1000 microstrains in compression resulted in fatigue failure at several hundred thousands cycles corresponding to the cycles to failure of 0.36 million cycles for the deck specimen N.

During the truck wheel traveling test of specimen N, after arch action was formed, the regions, where the strain ranges measured using embedded gages in concrete exceeded the strain range of 800 microstrains, gradually appeared. The deck specimen shifted into the situation that concrete in the region could fail owing to fatigue in compression.

It is known that an elastic modulus of concrete decreases with cycles in fatigue tests. To estimate the elastic modulus of concrete in the deck specimen N, each share of concrete and main compressive reinforcing bars in the internal compressive resultant force under arch action was calculated using measured strains of upper and lower reinforcing bars as shown in Fig. 19. Subsequently, assuming that the share equals to the ratio of concrete area to steel area considering young's modulus ratio, the elastic modulus of concrete in the internal compression zone of arch action was estimated. The result is shown in Fig. 20. It can be recognized that the elastic modulus of concrete in the compression zone rapidly decreased after developing arch action. The turn to increase of the apparent neutral axis observed after arch action was caused by the increase of strains in the upper main reinforcing bars due to the decrease of elastic modulus of concrete in the compression zone.

From this viewpoint, spalling on decks definitely affects fatigue durability of concrete decks under heavy traffic conditions. In fact, another deck specimen having artificial spalling on the top showed a significant decrease in the cycles to failure (N_f = 0.14 million cycles) when compared with the specimen N.[11]

Conclusions

From our investigations regarding fatigue and corrosion of concrete decks, the following two topics were introduced.

1) In several surveys to date, asphalt surfacing often prevented concrete decks from chlorides in a manner regardless of waterproofing. Most of asphalt mixtures covering

old removed bridges were determined as impermeable or low permeable. Asphalt mixtures seems to be getting impermeable relatively early in service, while being imperfect.

2) At early stage during the truck wheel traveling tests of full-scale concrete decks, arch action consisting of concrete arch rib and steel ties similarly to a shear resistance mechanism in reinforced concrete beams was recognized through the truck wheel traveling area. Unless arch action is constructed, concrete in decks is out of the range available to be fatigued in compression (Fig. 21). The cycles to arch action are likely to be expressed as a kind of fatigue behavior.

Acknowledgments

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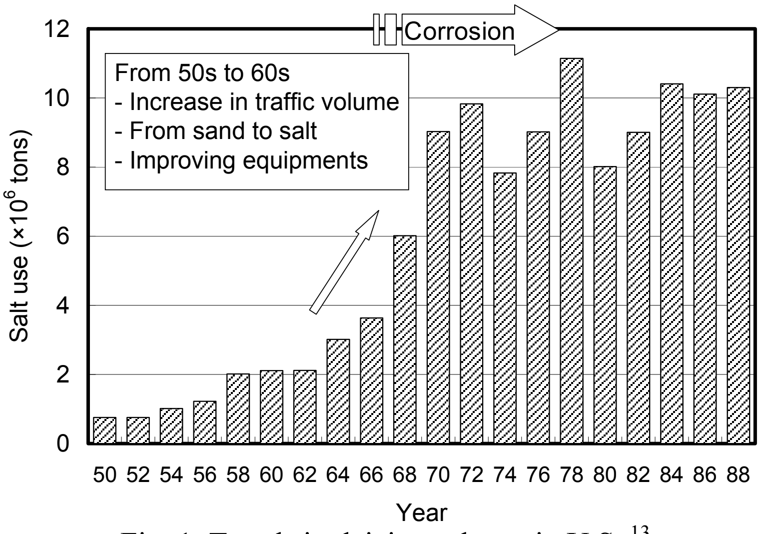

Fig. 1 Trends in deicing salt use in U.S. [13]

(a) Grid cracks (b) Through hole

Fig. 2 Deterioration due to fatigue in reinforced concrete decks

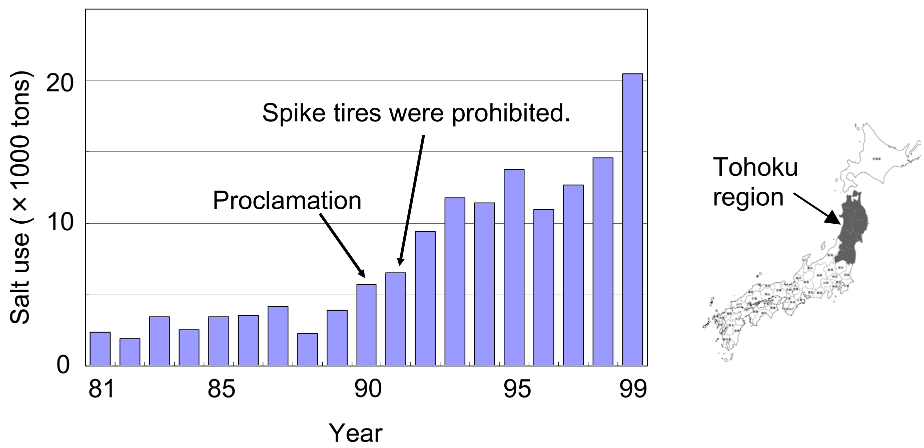

Fig. 3 Trends in deicing salt use in Tohoku region in Japan

Fig. 4 Corrosion of top mat reinforcement due to salt use in reinforced concrete decks
Both photo shows top of decks after asphalt and spalled concrete cover were removed.

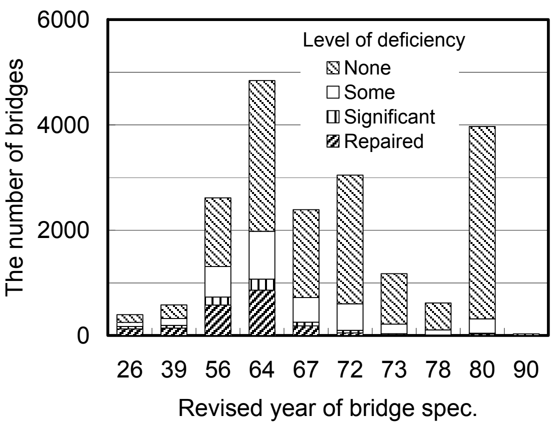

Fig. 5 The number of steel girder bridges classified under the level of deficiency due to fatigue in concrete decks (investigated in 1991)

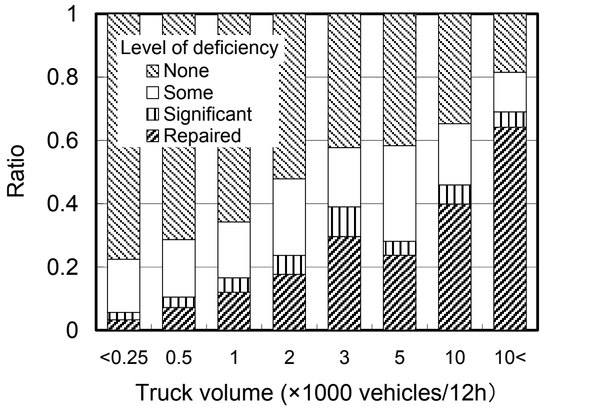

Fig. 6 The influence of traffic volume of large vehicles on fatigue in concrete decks
The ratio is based on the number of steel girder bridges designed according to highway spec. in 1964.
(investigated in 1991)

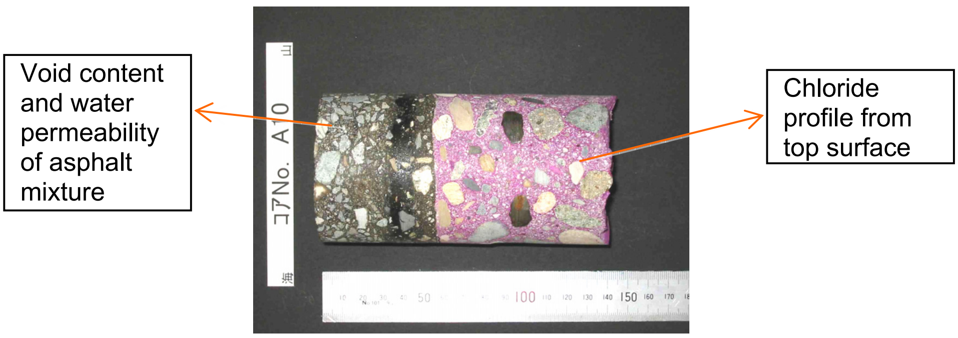

Fig. 8 A core with 70 mm dia. taken from the road surface of a closed prestressed concrete bridge at coastal area and purposes of the core

Note: Fig. 7 is shown in next page.

Comparisons of Deck Deterioration between U.S. and Japan

- **Chloride Attack** — **U.S.**
- **Fatigue** — **Japan**

(a) Major problems in concrete bridge decks

Items of Comparisons

Details of Decks	-
Materials - Concrete	Similar?
Quantity of Salt	U.S. > Japan
Over-loading	U.S. < Japan

(b) Key items of comparisons

Details of Decks
Deck Depth & Cover Depth

		Deck Depth	Cover Depth
U.S.			
Before corrosion	46 - 60s	6 in.	1.5 in. (to center)
	70s	7-8 in.	1.5 in. (to surface)
	Late 70s		2 in.
Japan			
Before fatigue	56 -	5.5 in.	1.2 in.
	68 -	6.5 in. *	1.2 in.
	71 -	7.3 in. *	1.2 in.

*) at Deck Span of 8.2 ft (2.5m)

(c) Comparisons in details of decks

Details of Decks
Bridge Deck Surface

- **U.S.**

Wearing Surface	% of Deck Area
Concrete	70.78%
Epoxy	4.87%
Bituminous	3.47%
Timber	1.09%
Gravel	1.18%
Other, None, N/A	16.89%

- **Japan**

Asphalt Pavement Almost

(d) Comparison in surfacing[14]

Quantity of Salt Use

Massachusetts	19	tons/lane-mile
New Hampshire	16	
Vermont	17	
New York	17	
Michigan	13	
California	3	
(on State Highways, 1980s?)		
National Highway	0.3	tons/lane-mile
Expressway	2	
(Average 1982-1987)		

(e) Comparison in salt use[13,15]

Over-loading

- **U.S.**
  - **1.2-1.3 times** of Design Loads
- **Japan**
  - **2 - 3 times** of Design Loads
  (Investigated in 1984-90)

(f) Comparison in overloading

Fig. 7 Comparisons in deterioration of bridge decks between U.S. and Japan
(Presented by Y. Tanaka in a breakout session in Reno, 2000)

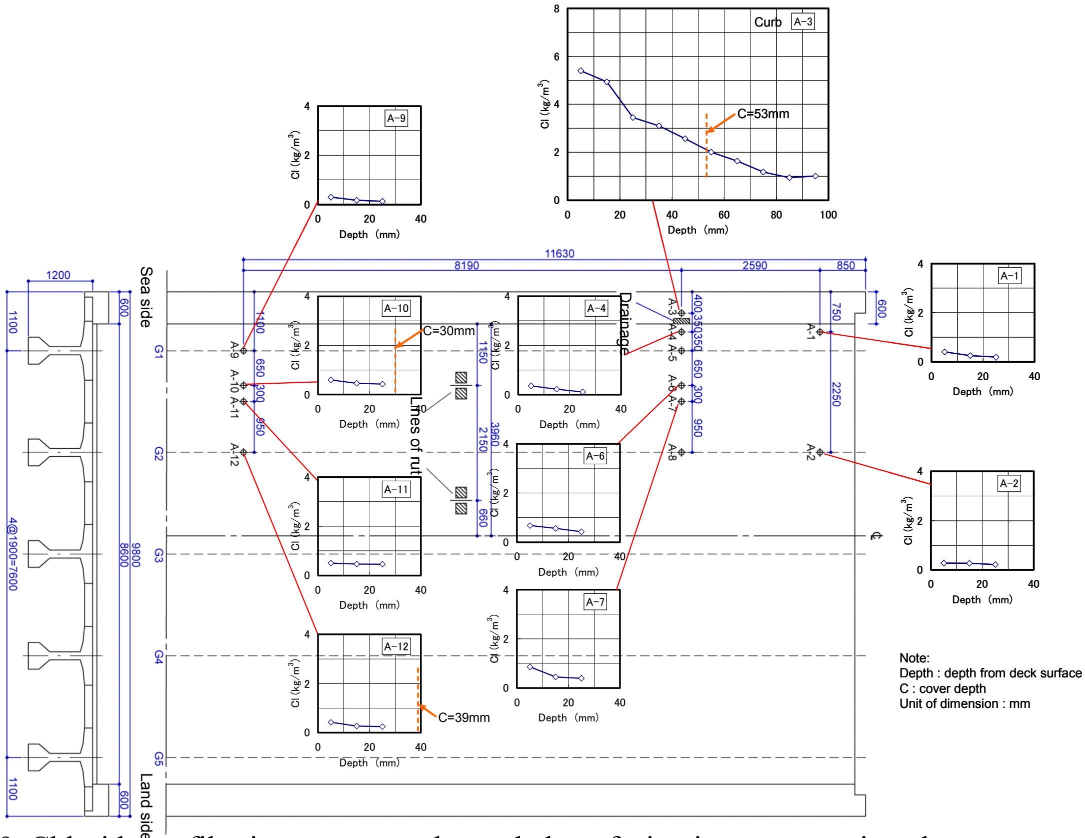

Fig. 9 Chloride profiles in concrete under asphalt surfacing in a post-tensioned concrete bridge exposed at coastal area for 34 years (ATG bridge)

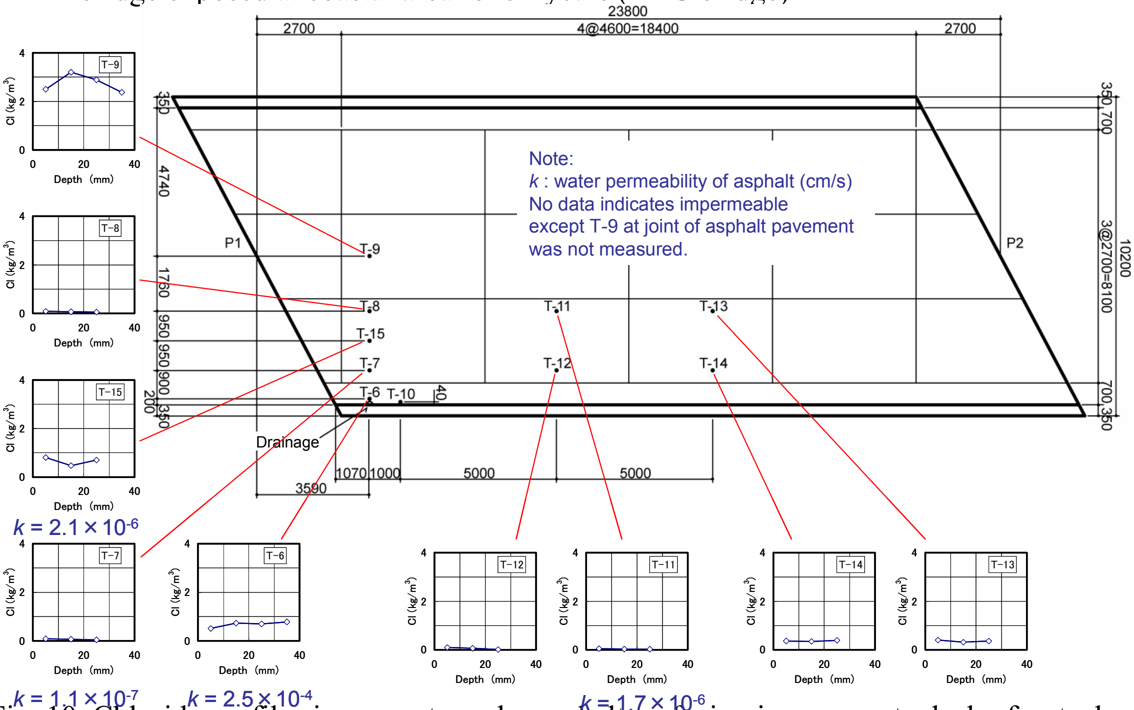

Fig. 10 Chloride profiles in concrete under asphalt surfacing in a concrete deck of a steel girder bridge used for 43 years suffering from deicing salt (TYD bridge)

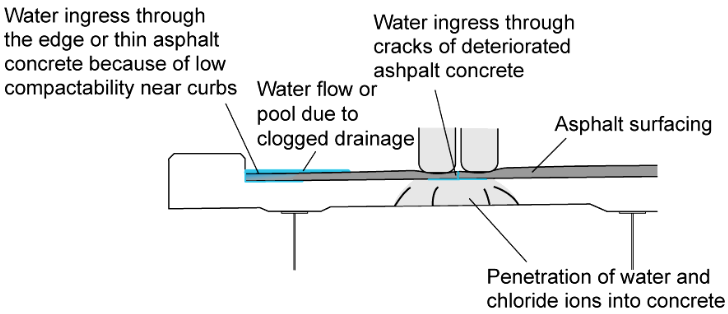

Fig. 11 Routes of chloride ingress to decks surfaced with asphalt mixtures

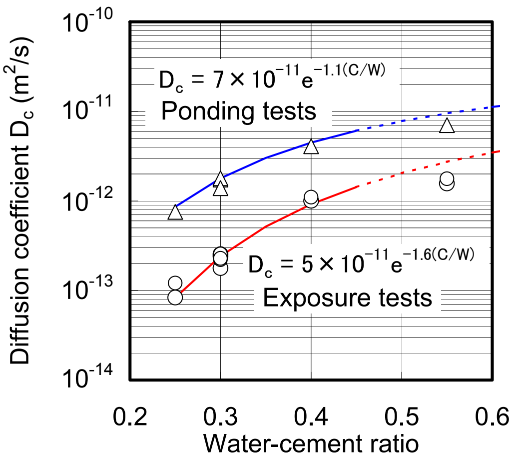

Fig. 12 Difference in diffusion coefficients of chloride ions in concretes between exposure test at coastal area and ponding test in salt water[8]

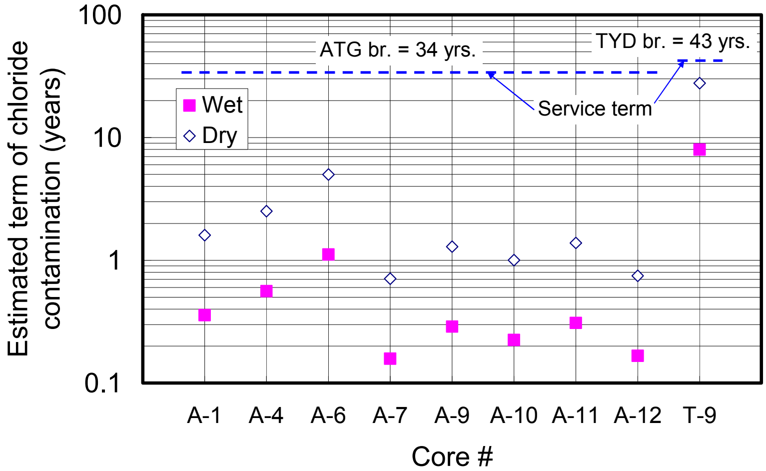

Fig. 13 The estimated terms of chloride contamination at top of decks

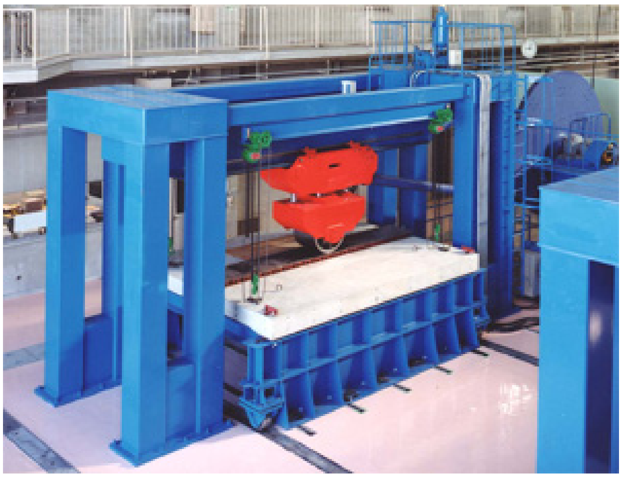

Fig. 14 Truck wheel traveling test using real-size concrete decks with 2.8 m x 4.5 m

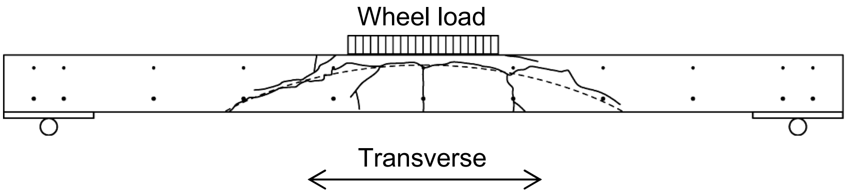

Fig. 15 Cracks in a deck specimen due to cyclic truck wheel loading
(N_f = 0.36 million cycles, wheel load = 157 kN)

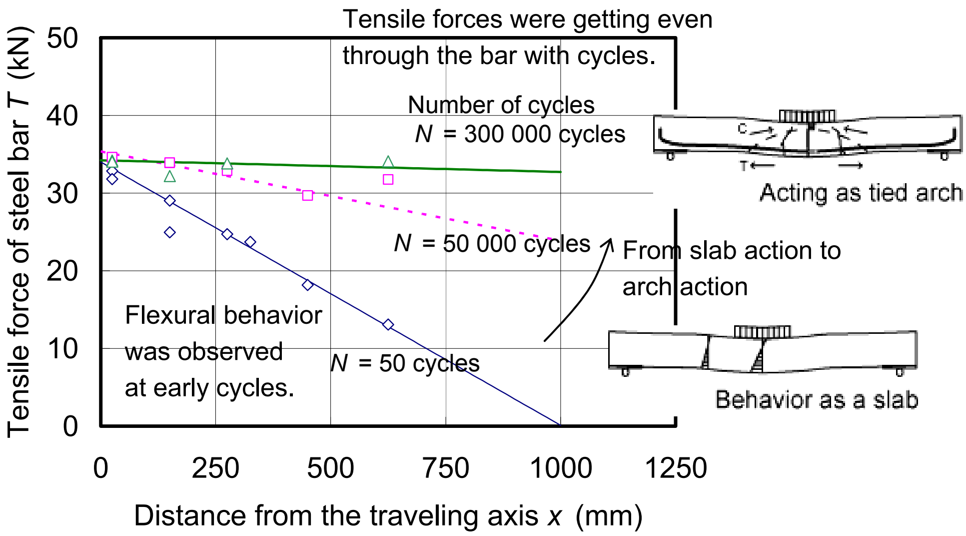

Fig. 16 Distributions of tensile axial force acting on a main reinforcing bar

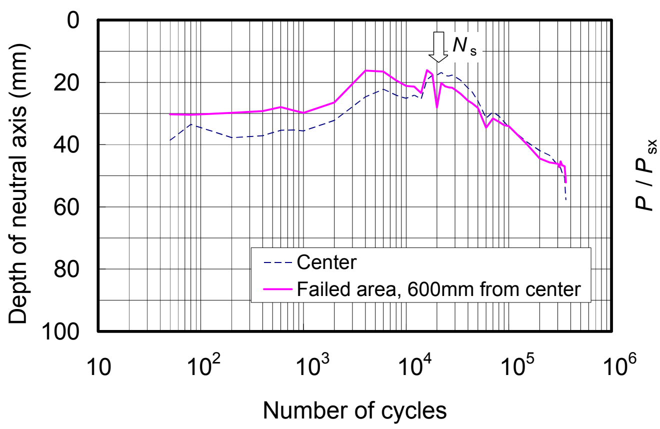

Fig. 17 Relation of depth of the apparent neutral axis from the top surface with cycles

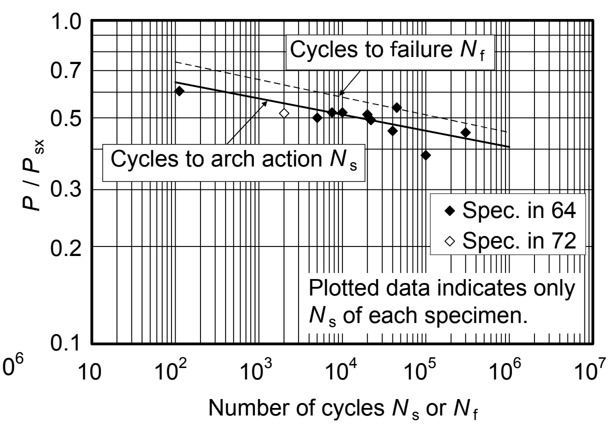

Fig. 18 Relation of loading level with cycles to arch action

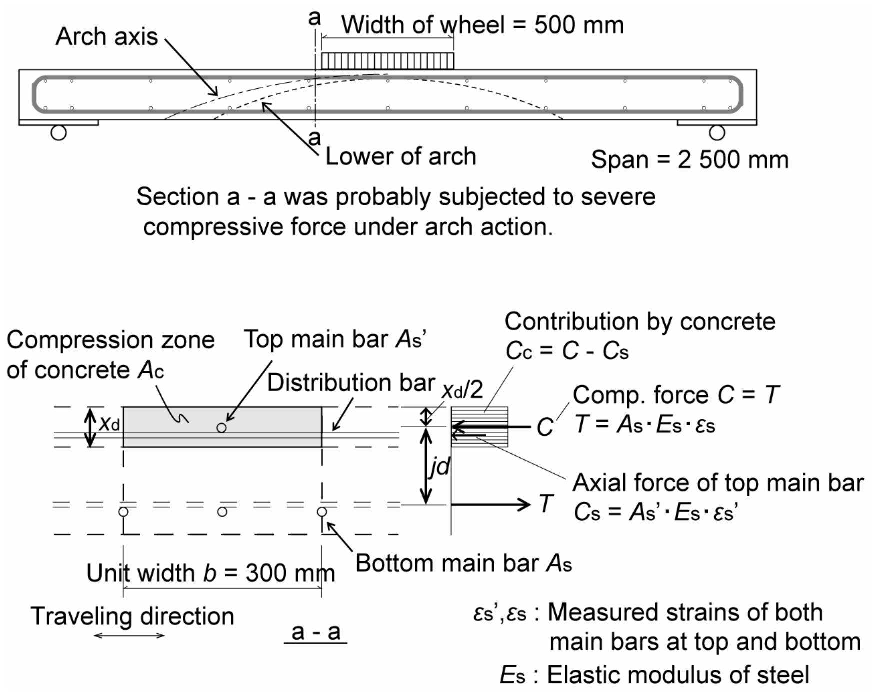

Fig. 19 Schematics of contributions of concrete and a top main steel bar subjected to the internal compressive force under arch action

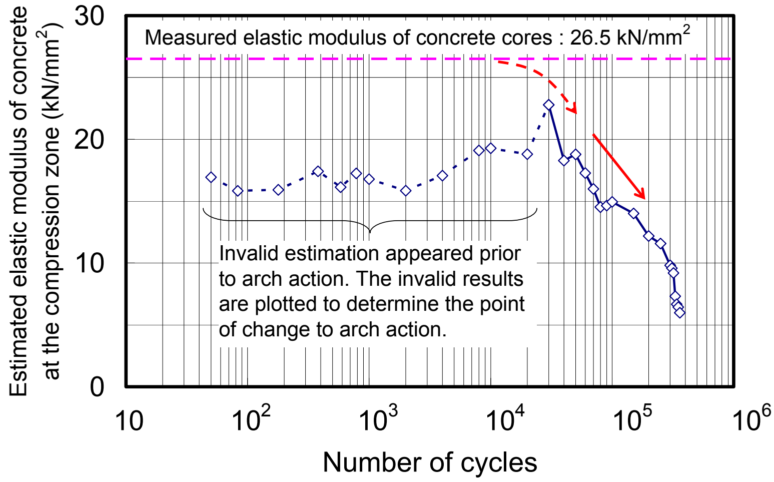

Fig. 20 Decrease in elastic modulus of concrete due to fatigue under arch action

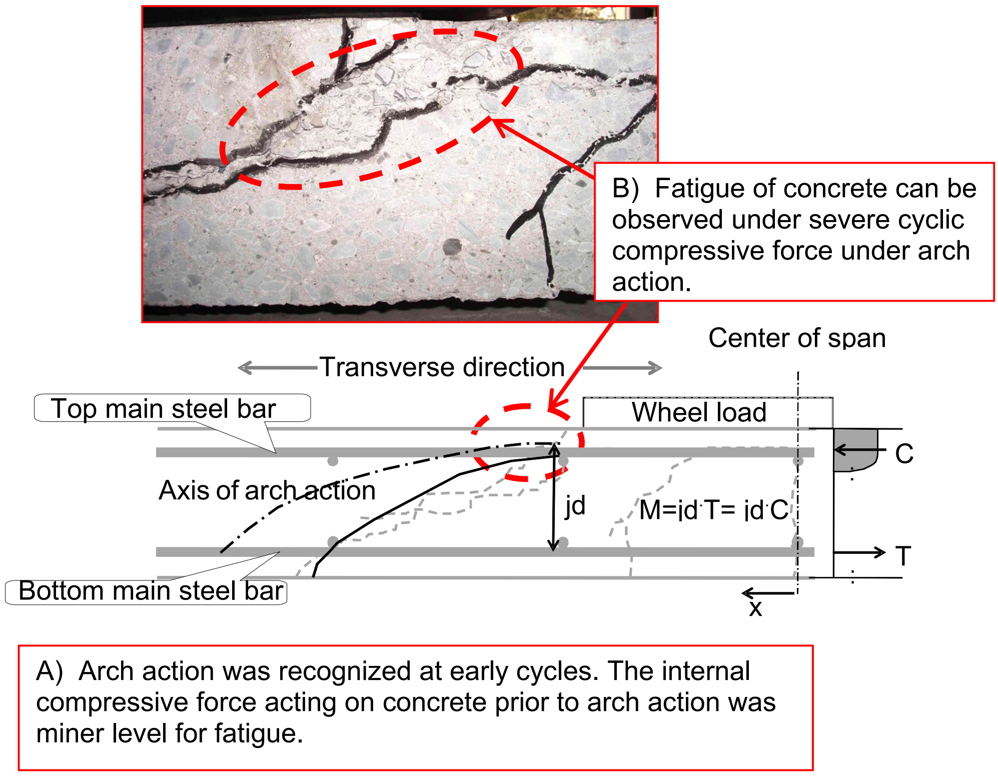

Fig. 21 Arch action and fatigue of concrete in compression