Fatigue Life of Damaged Bridge Deck Panels Strengthened With Carbon Fiber Sheets

Hongseob Oh, Jongsung Sim, and Christian Meyer

ABSTRACT

To simulate fatigue damage to bridge deck slabs, cyclic loading was applied to test panels, which were then reinforced with carbon fiber sheets (CFS) using two different methods. Subsequently, the strengthened panels were tested under cyclic loads. The observed response to fatigue loads differed markedly from the results of static tests. Isotropic reinforcement was found to be more effective than unidirectional strengthening. A simple life span prediction method is proposed for bridge decks, based on cumulative damage theory.

Keywords: bridge deck slab, carbon fiber sheets, fatigue resistance, cumulative damage theory, strengthening methods

A considerable body of knowledge exists of the response of metal structures to repeated loads. Microscopic flaws, inclusions, and other stress raisers give rise to an initiating crack, which under multiple load application grows and propagates to become critical and eventually lead to failure. This phenomenon is generally known as "fatigue". [1] The theory is well-established for metals, and modern metal structures subjected to multiple load applications, such as aircraft structures and railroad bridges are routinely designed on the basis of this theory.

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In the case of reinforced concrete structures, the situation is more complicated because of the complex nature of their response to load. Concrete contains numerous pores and microcracks, even before any load application. The nature of crack propagation and eventual fracture are subject to numerous influence factors, such as specific mix design parameters and environmental conditions (e.g. humidity), which complicate the fatigue behavior and make the development of a general theory a difficult undertaking. It is known that the failure modes of concrete structures under cyclic or dynamic loads can be different than under monotonically applied static loads.[2-7] This fact needs to be considered when methods to repair or strengthen damaged structures are contemplated.

Although the behavior of concrete bridge decks has been studied by various authors with respect to damage accumulation and fatigue failure[2], relatively little knowledge exists on how to systematically strengthen such bridge decks. For monotonically applied static loads an in-depth evaluation of different strengthening schemes using carbon fiber sheets (CFS) was presented elsewhere. [8] It is the objective of the current work to present the results of an experimental investigation which was conducted to evaluate bridge decks strengthened with CFS and subjected to repeated load applications. The other purpose of this study was to explore the suitability of a simplified cumulative damage theory for RC structural members as a tool to decide on strengthening strategies.

RESEARCH SIGNIFICANCE

Concrete bridge decks are strengthened either to restore their decreased load-carrying capacity or to extend their fatigue life. If the main purpose is an extension of fatigue life and endurance limit, then such improvement needs to be quantified. At present, methods to determine the remaining life of deteriorated bridge decks as well as strengthened decks are woefully inadequate. The experimental results reported in this study provide useful information in this regard and permit a comparison of the effectiveness of two different strengthening patterns with carbon fiber sheets. A simple theoretical method is proposed to predict the fatigue life of both deteriorated and strengthened bridge decks.

FATIGUE DAMAGE CONCEPT FOR STRENGTHENING

Classical linear fatigue damage theory dates back to the observation by August Wöhler that the number of load cycles up to failure, N, correlates strongly with the applied stress level, S. In actual structures, the situation is complicated by the fact that stress amplitudes vary greatly from load cycle to load cycle. The most commonly held assumption, based on the well-known Palmgren-Miner hypothesis, holds that damage accumulates linearly, according to the equation,

$$\sum_{i=1}^{k} \frac{n_i}{N_i} = 1 \tag{1}$$

where n_i is the number of applied load cycles causing a certain stress level s_i, and N_i is the corresponding number of cycles at which stress level s_i leads to failure.

For analysis of concrete members with large fracture process zones, several researchers have suggested the use of the cohesive crack model which takes into account the size effect. [9-11] But it is difficult to apply mechanical damage theory based on linear elastic fracture mechanics and FE analysis to reinforced concrete bridge decks because of the constantly changing boundary conditions and equilibrium states during successive load cycles, as the damaging cracks propagate.

According to Holmen[12], the Palmgren-Miner hypothesis overestimates damage for variable amplitude loading. Fatigue damage to structural elements subjected to cyclic loading either at constant or variable amplitudes can be analyzed by applying cumulative damage theory. [13-19]

Hashin and Rotem[14] used nonlinear cumulative damage theory to estimate the fatigue life of metal elements. Oh[15] and Grzybowski and Meyer[16] proposed nonlinear cumulative damage models for plain concrete under varying loading amplitude based on test results. By definition, a RC element subjected to cyclic load at some stress level s causes fatigue failure when the number of cycles reaches N_f , i.e. fatigue damage D equals 1. A smaller number of cycles n' ($n' < N_f$) causes a fatigue damage value D between 0 and 1. The relationship between n', s and D is characterized by the S-N-D surface shown in Fig. 1. A critical issue is how to define fatigue damage (D). Grzybowsky and Meyer[16], Williams and Sexsmith[17], and Paskova and Meyer[18] suggest that a damage index be defined as the ratio between the dissipated energy E and the total energy dissipation capacity E_{tot} for a given stress or strain level (i). Fatemi and Yang[13], Hashin and Rotem[14] and Oh[15] defined damage as a strain ratio for either a homogenous material or plain concrete. But fatigue damage of strengthened RC members consisting of concrete, rebars, and strengthening material cannot be readily defined using such a strain concept. It is particularly difficult to characterize the residual life of deteriorated RC bridge deck panels that have been subjected to an unknown number of load cycles and the other external influences, using a simple local strain concept.

By strengthening a RC member, the S-N-D surface (Fig. 1), by definition, is shifted such that, for each combination of stress level and number of load cycles, a smaller damage level is to be expected. However, whereas the degradation of a member's load-carrying capacity can be assessed either experimentally or analytically, it is not straightforward to quantify the effect of strengthening measures taken to improve the fatigue behavior of a member that has been damaged before being strengthened. In this study, fatigue damage of deck panels before and after strengthening is evaluated, taking their load-displacement relationship as representative for overall structural response. Standard composite RC theory as described in Ref. 8 is used to assess the static load-carrying capacity of unstrengthened and strengthened RC members.

The authors have previously reported[8] that two-directional strengthening with CFS strips is more effective than any other strengthening scheme, if the load is applied statically. In order to effectively extend the fatigue life of a bridge deck by strengthening, the fatigue damage experienced prior to strengthening must be taken into account. For a fatigue damage theory to quantify the enhancement due to strengthening, all these factors need to be included.

EXPERIMENTAL TEST PROGRAM

Specimens

Four two-way RC slab specimens were constructed, Fig. 2. A prototype deck panel of dimensions 160 by 240 cm and 22 cm thickness, supported by two girders was selected to simulate a real bridge deck. In the transverse direction, the reinforcement ratio was 0.551%, and in the longitudinal direction, it was 67% of this amount, i.e. 0.367%. The panels, considered to be representative of secondary bridge decks used in Korea, are designed for traffic loads similar to HS20 trucks.

Materials

The concrete used for the specimens consisted of ordinary Portland cement, natural sand, and crushed coarse aggregate with a maximum size of 25 mm. The mixture had a 28-day cylinder strength of 31 MPa. Deformed bars 15.9 and 9.35 mm in diameter (D16 and D10) with average yield strength of 300 MPa were used to reinforce the slab panels and beams. The shear reinforcement of the edge beams consisted of 9.35-mm diameter closed stirrups.

The carbon fiber sheets (CFS) used to strengthen the damaged slabs had an ultimate strength of 3,550 MPa, a Young's modulus of 2.35×10^5 MPa, and an ultimate strain of 1.5%. The properties of the materials used in the specimens are listed in Table 1. The CFS were bonded to the deck panels in an upside-down position as follows. To remove all laitance and smoothen the surface, the deck area was ground by hand and cleaned afterwards with compressed air. A resin was applied as a primer by roller and cured for 24 hours with a protective cover to keep it dry. After blending the epoxy adhesive in a suitable container, it was spread by roller evenly over the bottom surface of the deck. CFS were then attached to the epoxy-coated surface and further pressed into the epoxy coating with a screw type roller until they were completely immersed and no air voids between the concrete and the sheets remained. The strengthened panels were then cured for at least 10 days before testing. For panels with both longitudinal and transverse CFS strengthening, the CFS in the longitudinal direction were attached first.

Test set-up, test procedures, and measurements

The test set-up is illustrated in Fig. 2b. The specimens were supported by hinge supports. A 250×500-mm rectangular steel plate and rubber pad simulated the contact surface of a truck wheel, and the load was applied at a rate of 2 Hz by an actuator with a capacity of 500 kN. The rubber pad was inserted to avoid stress concentrations. Data were recorded by an automated data acquisition system.

Linear variable displacement transducers (LVDTs) were used to obtain deflection profiles along the slab center line. Concrete strain gauges attached in the longitudinal and transverse direction at midspan to the compressive deck surface measured the concrete strain variations, and electrical resistance strain gauges were bonded to the main reinforcement and CFS to obtain strain profiles (see Fig. 2a).

Test program

All four test specimens were loaded cyclically such as to incur fatigue damage, before they were strengthened. The load levels for this part of the test program were selected such that the reinforcing bars were stressed to either 40% or 60% of their yield strengths, as obtained from previous static test results and summarized in Table 2. Those panels whose bars were stressed to 40% yield underwent 200,000 load cycles, whereas those whose bars were stressed to 60% yield were exposed to either 10,000 or 100,000 load cycles (Table 3).

After having been damaged by such “precycling”, the specimens were repaired by injecting epoxy into the cracks and strengthened with CFS using either the T1L2 or T2L2 schemes shown in Fig. 3. In both schemes, five 15 cm wide CFS strips were attached in the longitudinal direction (L). In scheme T1L2, five 15 cm wide T1L2 CFS strips were also attached in the transverse direction (T), whereas in the T2L2 scheme, three 12.5 cm wide strips were attached in the transverse direction. The specimens were allowed to cure for 14 days before further testing.

Because of the different strengthening patterns, the same applied load caused different stresses in the reinforcing bars of the two panels. Thus, after strengthening, the two panels that had been precycled to 40% of the rebar yield strength, were loaded cyclically with 122.5 kN. This load, equivalent to an actual wheel load on a real bridge deck, stressed the steel bars to 70% or 80% of their yield strengths, depending on which strengthening pattern was used (Table 2), e.g. specimen 40-T1L2-70 was precycled with 40% of the rebar yield load, then strengthened by the T1L2 scheme, and finally subjected to cyclic loads with 70% of the rebar yield load (Tables 2 and 3). The panels that had been precycled to 60% of the rebar yield strength were subjected after strengthening to cyclic loads which caused either 40% or 50% of yield in the rebars (see Table 3), again for the same applied load amplitude.

TEST RESULTS AND DISCUSSION

Crack patterns and failure modes

The crack patterns of test panels caused by monotonic loads in the previous test are depicted in Fig. 4, while Fig. 5 shows the propagation of the cracks experienced by unstrengthened decks exposed to 60% of rebar yield loads in the current cyclic load test. The unstrengthened reference panel CON failed in biaxial bending under monotonic loading. Initially, cracks developed in the longitudinal direction, and as the load increased, also transverse cracks appeared that propagated from the initial longitudinal cracks up to failure. The crack propagations in unstrengthened decks under cyclic loads, depicted in Fig. 5, were slightly different from those of the reference panel CON. The longitudinal cracks in specimens subjected to fatigue loads were more developed than in the CON specimen. In the unstrengthened panel, two-directionally developed initial cracks dispersed to the panel edges as load cycling proceeded.

Typical crack patterns of two deck specimens before and after strengthening are shown in Figs. 6 and 7. They were similar to those that can be observed in real bridge decks. Prior to strengthening, the panels developed the crack patterns of Figs 6a and 7a. As expected, the cracks in panels subjected to higher loads (60-T1L2-40 and 60-T2L2-50) were wider that those of panels that received lower loads (40-T1L2-70 and 40-T2L2-80) not shown here.

After panels 40-T1L2-70 and 40-T2L2-80 were strengthened and again loaded cyclically, they experienced partial delamination of the carbon fiber sheets after excessive flexural cracking. Yet, because the CFS distributed tensile stresses and kept the flexural cracks relatively small, complete debonding did not occur. Because of the good bond between concrete and CFS reinforcement, no dominant large cracks were able to develop, only numerous closely spaced small-sized cracks. The crack patterns of panels 60-T1L2-40 and 60-T2L2-50 after strengthening were strongly influenced by those produced in the pre-cycled deck panels prior to strengthening. After strengthening, the newly developed flexural crack patterns appeared to be very similar to the original patterns observed after preloading. Of course, the CFS had to be removed to expose those cracks.

In the previous study[8] it was observed that under static loads, all unstrengthened and two-directionally strengthened decks exhibited typical flexural failure modes. The four test specimens of the present investigation, which were subjected to cyclic loads, all failed eventually in a brittle mode after rupture of the transverse carbon fiber sheets.

Load-displacement relationships

The numbers of load cycles to failure for the various specimens are listed in Table 3. Load-displacement curves for representative load cycles are depicted in Figs 8 and 9 for two panels before and after strengthening. All displacements shown represent absolute values, i.e. they include all permanent deflections experienced during prior load cycles. The load-deformation curves of Figs 8 and 9 indicate that the fatigue life of a specimen can be divided into three phases: (1) an initial damage phase, which may last for about 10 cycles, with rapidly increasing permanent displacements, (2) a steady-state damage accumulation phase with much smaller residual displacement increments, and (3) a final fatigue damage phase. As expected, structural response during the first load cycle was comparable to that observed under static load. Likewise, the maximum displacements experienced during the final load cycle before failure were comparable to those recorded during the static load test. A similar observation has been made by Otter and Naaman.[20]

In all cases the strengthening measures restored the original structural stiffness, as measured by the initial slope of the load-displacement curve. For example, strengthening of panel 60-T1L2-40, which had undergone plastic deformations during preloading, increased the stiffness by 50%, compared with that exhibited after 100,000 load cycles in the preloading phase, not shown here.

Fig. 10 illustrates how the compliance of the various test panels changed as they underwent cyclic loading. The compliance can be determined from a load-displacement or load-crack mouth-opening displacement curve and can be used to measure the fatigue response of brittle materials. In this study, it is defined as $C = d / P$ from the load-displacement curve. It is apparent that panel 60-T2L2-50, after strengthening, experienced no fatigue failure and only a minimal increase of its compliance, while all other panels experienced appreciable increases in compliance. This observation agrees with the stiffness drops observed in Fig. 9.

Dissipated energy

The total energies dissipated before and after strengthening are illustrated in Fig. 11. Energy dissipation during each load cycle was defined as the difference between the areas under the load-displacement curves for loading and unloading. The total dissipated energies were then determined by summing the dissipated energies for all load cycles up to failure. The energies dissipated by specimens with lower stress levels after strengthening were considerably less than when stress levels were higher.

Panel 60-T2L2-50 which dissipated the least amount of energy before strengthening turned out to dissipate the most after strengthening. This fact seems to indicate that fatigue life after strengthening depends on the damage experienced before strengthening and can be further illustrated with the help of Fig. 12, which shows the qualitative variation of relative stiffness during cyclic loading. An unstiffened panel would have a normalized stiffness and fatigue life as characterized by curve A. Strengthening such a panel before any load application will increase both its stiffness and fatigue life (curve B). If the panel is strengthened after having experienced a modest amount of damage, a considerable increase of its remaining fatigue life can be expected as a result of that strengthening (curve C). The experimental results for panel 60-T2L2-50 indicate that the increase in fatigue life is pronounced even if the strengthening is applied after a considerably larger amount of damage has been sustained (curve D). The extraordinary increase in fatigue life of panel 60-T2L2-50 after strengthening far exceeds what should be expected and therefore may simply be an experimental aberration. If the damage experienced during the pre-loading phase was extensive and approached failure, the effect of strengthening on fatigue life extension should be less dramatic (curve E).

This finding underscores the importance of the correct timing of strengthening a bridge deck panel, which has undergone fatigue-type loading.

CUMULATIVE DAMAGE THEORY

One purpose of this study was to propose a simple procedure to estimate the remaining life of deteriorated bridge decks and to assess quantitatively the increase in fatigue life due to strengthening. This is a difficult task, because classical damage mechanics does not apply, since boundary conditions keep changing as damage increases. Therefore, the method proposed herein is based on the empirical information provided by the load-deformation relationships presented in the previous section. The load-displacement relationship of a deck panel is a better representation of the structural deterioration than some local stress-strain relationship conventionally adopted to characterize fatigue damage.

Fatigue damage shall be defined here as d / d_{max}, where d is the cumulative displacement at any load cycle and d_{max} is the maximum displacement prior to failure. As pointed out earlier, d_{max} for cyclic loading is very similar to the maximum displacement observed in a static test, for both unstrengthened and strengthened panels, provided the same strengthening method is applied (see Table 4).

For design purposes, it is not advisable to rely on such a failure displacement, because it is difficult to reliably determine it. Instead, it is suggested to substitute in the cyclic loading case a maximum displacement 10% higher than that at which both transverse and longitudinal steel are expected to yield, as determined by the test. This added safety margin is justified because repeated loading causes damage which reduces the capacity of the deck for further load redistribution as compared with the static case.

In the case of strengthened deck panels, the maximum displacement is defined as that at which either the CFS sheets fail or the concrete crushes in compression.

In Fig. 13, the three damage phases mentioned in the previous section are illustrated: In Phase I, consisting of about 10 cycles, the first load cycle introduces a considerable amount of damage, whereas the subsequent cycles of Phase II cause damage increments of decreasing magnitude. In Phase III, fatigue damage accelerates, leading to failure within 1000 to 5000 cycles. Based on a number of assumptions, a degree of damage and the residual life of a damaged deck can be estimated for different stress levels.

The damage of Phase I is mostly due to that caused in the first load cycle, which is similar to that produced in a static test with the same load, i.e.

$$D_I = \frac{d_{static}}{d_{\max}} \qquad n = 1 \tag{2}$$

where d_{static} is a function of stress level. In Phase II, the small increments in damage can be represented by

$$D_{II} = b\,\frac{\ln n_{II}}{\ln N_f} \qquad 1 < n_{II} \le n_e \tag{3}$$

where N_f is the number of cycles to failure at a given stress level, n_{II} is the number of load cycles actually applied, b is the slope of the damage curve (Fig 13), and n_e is the number of cycles defining the end of Phase II. n_e depends on the applied stress level and is assumed to be reached when the deck deflection under cyclic loading is equal to 90% of the maximum deflection observed in a static test.

The slope b in Phase II can be estimated from the test results for different stress levels and expressed in the form of a log function as follows,

$$b = -1.644\ln\left(\frac{P}{P_s}\right) - 0.2955 \tag{4}$$

where P is the applied cyclic load and P_s the static failure load. In Phase III, fatigue damage can be estimated as follows:

$$D_{III} = g\,\frac{\ln(n_{III} - n_e)}{\ln N_f} \qquad n_e < n_{III} \le N_f \tag{5}$$

where,

$$g = 0.445\left(\frac{P}{P_s}\right) + 0.6215 \tag{6}$$

The fatigue life for any load level can then be estimated by the following modified Miner's rule[16]:

$$\left(\frac{n_1}{N_{f1}}\right)^{\frac{1-D_2}{1-D_1}} + \left(\frac{n_2}{N_{f2}}\right) = 1 \tag{7}$$

where n_1 and n_2 are the number of load cycles with two different load levels actually applied; N_{f1} and N_{f2} are the corresponding numbers of load cycles to failure; D_1 and D_2 are the damage levels caused by those load cycles. The exponent of the first term in Eq. 7 converts the damage produced by cycling with load level 1 to an equivalent amount of damage, if n_1 load cycles had been applied at load level 2. Using Eq. 7, it is possible to predict the number of load cycles to failure. The results are listed in Table 5.

The fatigue life $N_i^{'}$ of a strengthened concrete slab for any stress level can be expressed as the sum of fatigue life N_i of the unstrengthened deck and the fatigue life extension $N_{sheets}^{'}$ due to CFS sheets:

$$N_i^{'} = N_i + N_{sheets}^{'} \tag{8}$$

Alternatively, the remaining life of a deteriorated deck after strengthening is equal to the residual life of the deteriorated deck and the fatigue life extension provided by the strengthening material. The residual life of the deteriorated unstrengthened deck, n_2 , can be estimated using Eq. 7. It now remains to determine the fatigue life extension provided by the strengthening measure.

By adding the effect of the CFS, $N_{sheets}^{'}$, to the residual fatigue life of the unstrengthened deck,

$$N_i^{'} = \frac{1-n_1}{N_{f1}} \tag{9}$$

after it has been converted to an equivalent number of cycles with the new load amplitudes, we obtain

$$\left(\frac{n_1^{'}}{N_{f2}(1-(\frac{n_1}{N_{f1}})^{\frac{1-D_2}{1-D_1}}) + N_{sheets,f1}^{'}} \right)^{\frac{1-D_2^{'}}{1-D_1^{'}}} + \left(\frac{n_2^{'}}{N_{f2}^{'}} \right) = 1 \tag{10}$$

Table 6 summarizes the theoretical predictions of fatigue life and compares them to the experimental results.

CONCLUSIONS

The results of the research presented here lead to the following conclusions. Strengthening pre-damaged concrete bridge deck panels with CFS can substantially increase their fatigue life and restore their structural stiffness. Specimen 60-T1L2-40, which had almost failed during the pre-loading test phase, experienced significant improvement of its fatigue life.

Strengthening a deck by the T2L2 pattern proved to be more effective than by the T1L2 pattern. Since the transverse reinforcing steel is 50% larger than the longitudinal reinforcement, the specific CFS strengthening scheme T2L2 makes the combined reinforcement more isotropic. The resulting redistribution of strength and stiffness appear to have a beneficial effect on the fatigue behavior of the deck. Interestingly, in the static load case, the T1L2 reinforcing pattern proved to be more effective.

The proposed simple damage theory can be used to estimate the residual life of bridge decks with unknown damage, even if these are strengthened with CFS sheets. Thus, this theory might be useful to determine the cost-effectiveness of bridge deck strengthening strategies. However, this proposed model is based on a small number of tests and therefore limited to the range of the parameters governing those tests. Refinements of the model are to be expected as additional fatigue test data on strengthened deck panels become available.

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REFERENCES

1.Comité Euro-International du Béton d'Information No188, *Fatigue of concrete structures, State of the Art*, 1988.

2. Schläfli M. and Brühwiler E., "Fatigue of existing reinforced concrete bridge deck slabs," *Engineering Structures*, V. 20, N. 11, 1998, pp.991-918.

3. Sonoda K., "Fatigue strength of reinforced concrete slabs under moving loads," *IABSE Reports*, V. 37, 1982, pp. 455-462.

4. Okada K., Okamura H. and Sonada K., "Fatigue failure mechanism of reinforced concrete bridge deck slabs," *Transportation Research Record 664*, V. 1, 1978, pp.136-144.

5. Mufti A. A. and Newhook P. J., "Punching shear strength of restrained concrete bridge deck slabs," *ACI Structural Journal*, V. 95, N. 4, 1998, pp. 375-381.

6. Perdikaris P. C. and Beom S., "RC bridge decks under pulsating and moving load," *Journal of Structural Division*, ASCE, V. 114, N. 4, 1988, pp. 591-607.

7. Fang I. K, Worley J. A, Burns N. H. and Klingner R. E. "Behavior of isotropic R/C bridge decks on steel girders," *Journal of Structural Engineering, ASCE*, V. 116, N. 3, 1990, pp. 659-678

8. Oh H., Sim J. and Meyer C., "Experimental assessment of the bridge deck panels strengthened with Carbon Fiber Sheets," *Composites Part B: Engineering*, V. 34, N. 6, 2003, pp. 527-538.

9. Bažant Z. P. and Xi Y., "Statistical size effect in quasi-brittle structures: II. Nonlocal theory," *Journal of the Engineering Mechanic*, ASCE, V. 117, N. 11, 1991, pp. 2623-2640.

10. Hordijk D. A. and Reinhardt H. W. "A fracture mechanics of plain concrete," *Fracture Mechanics of Concrete Structures*, Bažant Z. P. ed, Elsevier Applied Science, London, 1992, pp. 924-929.

11. Horii H., "Mechanism of fracture in brittle disordered material," *Fracture Processes in Concrete, Rock*

and Ceramics, J. G. M. van Mier, J. G. Rots and Abakker eds, E&FN Spon, London, 1991, pp. 95-110.

12. Holmen J. O., "Fatigue of concrete by constant and variable amplitude loading," *Fatigue of Concrete Structures*, ACI SP-75, Detroit, 71-110.

13. Fatemi A. and Yang L., "Cumulative fatigue damage and life prediction theories: a survey of the state of the art for homogeneous materials," *International Journal of Fatigue*, V. 20, N. 1, 1998, pp. 9-34.

14. Hashin Z and Rotem A. "A cumulative damage theory of fatigue failure," *Material Science and Engineering*, V. 34, 1978, pp. 147-160.

15. Oh B.W., "Fatigue Analysis of Plain Concrete in Flexure," *Journal of Structural Engineering*, ASCE, V. 112, N. 2, 1986, pp. 273-288.

16. Grzybowski M. and Meyer C., "Damage Accumulation in Concrete with and without Fiber Reinforcement" *ACI Materials Journal*, V. 90, N. 6, 1993, pp. 594-604.

17. Williams M. S. and Sexsmith R. G., "Seismic assessment of concrete bridge using inelastic analysis," *Engineering Structures*, V. 19 N. 3, 1996, pp. 208-216.

18. Paskova T. and Meyer C., "Low-cycle Fatigue of Plain and Fiber-Reinforced Concrete," *ACI Materials Journal*, V. 94, N. 4, 1997, pp. 273-285.

19. Kujawski D. and Elyin F., "A cumulative damage theory of fatigue crack initiation and propagation," *International Journal of Fatigue*, V. 6, N. 2, 1984, pp. 83-88.

20. Otter D. E. and Naaman A. E. "Properties of Steel Fiber Reinforced Concrete Under Cyclic Load," *ACI Materials Journal*, V. 85, N. 4, 1988, pp. 254-261.

List of Tables

List of Figures

Table 1 Physical Properties of Materials

Properties / Materials	Yield strength, MPa	Ultimate strength, MPa	Modulus of elasticity, GPa	Ultimate strain
Reinforcing bars	300	350	200	-
CFS	-	3,550.	235	1.5
Properties / Materials	**Compressive strength, MPa**	**Young's modulus, GPa**	**Modulus of rigidity, GPa**	**Poisson's ratio**
Epoxy	88.	7.0	2.3	0.3
Concrete	31.0	26.0	8.6	0.18

Table 2 Strengthening scheme and static test results (Ref. 8)

Specimen	Strengthening ratio $(\frac{A_{cfs}}{h \cdot b}, \times 10^{-4})$		Strengthening scheme	Failure mode	Yield load (kN) (rebar yield point)		Failure load (kN) (CFS rupture or kinking)	
	T	L			Test	Theory	Test	Theory
CON	-	-		Flexural-shear	405 (Type 4)	605 (Type 4)	573 (Type 4)	-
T1L2	1.875	2.344	Two-directional strip	Flexural-Shear	450 (Type 4)	638 (Type 2)	626 (Type 2)	573 (Type 4)
T2L2	0.9375	2.344	Two-directional strip	Flexural-Shear	520 (Type 4)	454 (Type 2)	638 (Type 2)	626 (Type 2)

Table 3 Fatigue test variables

Specimen	Stress level for pre-damage*	No. of cycles for pre-damage	Strengthening scheme	Post-strengthening stress level**	No. of cycles to failure, N_f
40-T1L2-70	40% of yield strength	200,000	T1L2	70% of yield strength	55,868
40-T2L2-80	40% of yield strength	200,000	T2L2	80% of yield strength	79,776
60-T1L2-40	60% of yield strength	100,000	T1L2	40% of yield strength	262,911
60-T2L2-50	60% of yield strength	10,000	T2L2	50% of yield strength	-***

* : obtained from static test of control specimen
** : obtained from static test of T1L2 or T2L2 specimen
***: after completion of 750,000 cycles, static test to failure

Table 4 Relationship between static deflection and cumulative deflection

Strengthening type	Maximum static deflection (mm)	Cumulative deflection (mm)			
		60-T1L2-40	60-T2L2-50	40-T1L2-70	40-T2L2-80
T1L2	21.2	22.03		20.73	
T2L2	18.8		-*		18.42
$\frac{Cumulative\ \ Deflection}{Static\ \ Deflection}$	-	104%		97.8%	98%

* - No fatigue failure

Table 5 Residual life predictions for panels prior to strengthening

Specimen	Cycles for Pre-damage	D_{II}	$D_{Total} = D_I + D_{II}$	Expected Life (N_1)	Residual Life (n_2)
		Eq. 3	Eq. 2 and 3	Eq. 7	Eq. 7
40-T1L2-70	200,000	0.112	0.3	over 10^8	over 10^8
40-T2L2-80	200,000	0.11	0.256	over 10^8	over 10^8
60-T1L2-40	100,000	0.582	0.887	130,862	30,862
60-T2L2-50	10,000	0.481	0.786	130,862	120,862

Table 6 Fatigue life predictions for pre-damaged and strengthened panels

Specimen	S-N relationship	Predicted fatigue life	Experimental number of cycles to failure
40-T1L2-70	$S = 1 - 0.0274\ln(n)$	58,000	55,868
40-T2L2-80	$S = 1 - 0.0177\ln(n)$	85,000	79,776
60-T1L2-40	$S = 1 - 0.0477\ln(n)$	280,000	262,911
60-T2L2-50	$S = 1 - 0.0264\ln(n)$	1.64×10^8	Static failure after 750,000

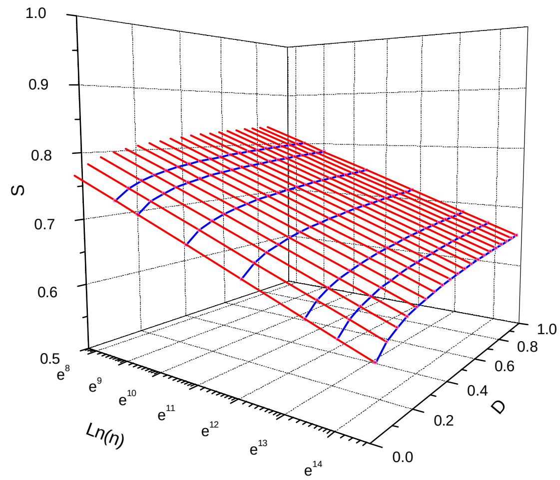
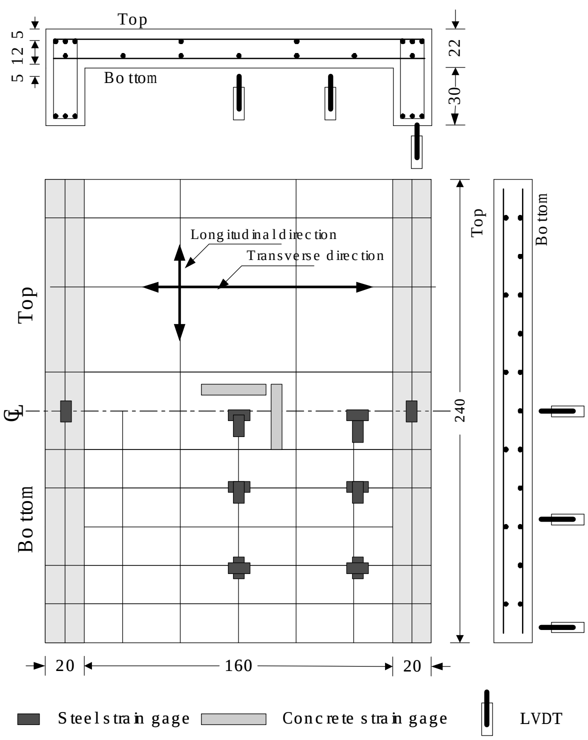
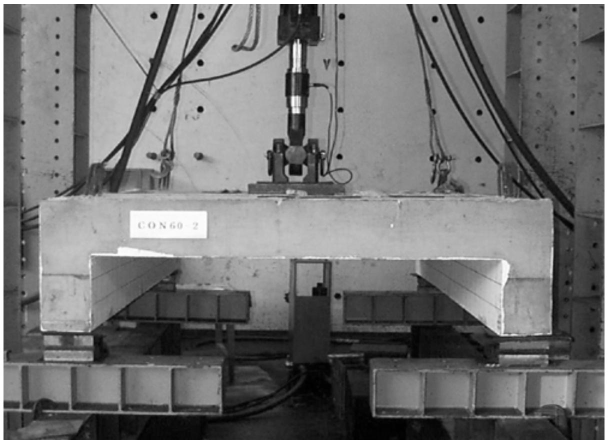


Fig. 1 Conceptual S-N-D surface
(S-stress level, N-no. of load cycles, D-damage)

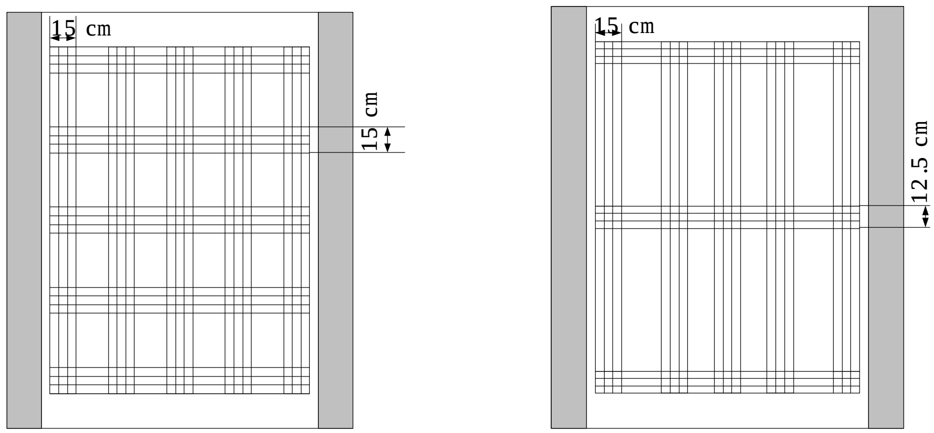


a) Dimensions and reinforcement details (unit: cm)



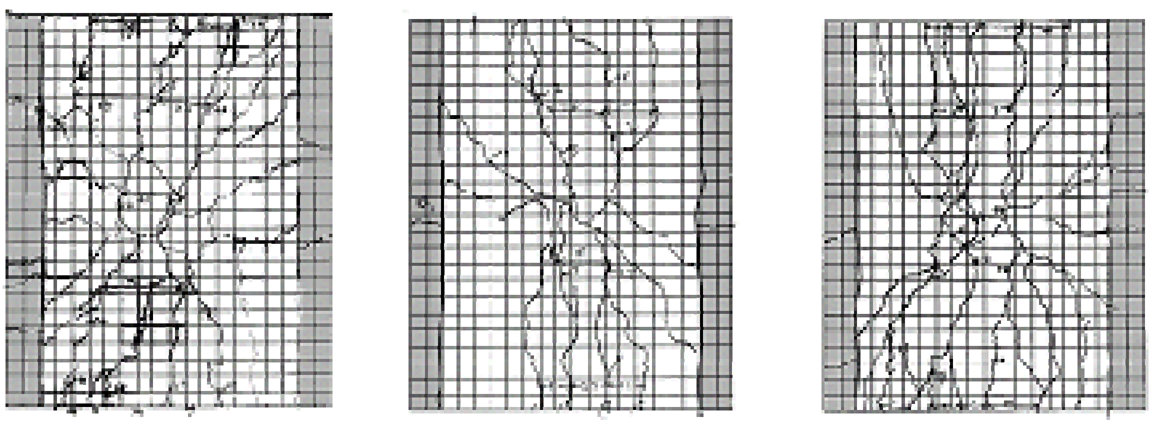
b) Test setup

Fig. 2 Test specimen details and test setup



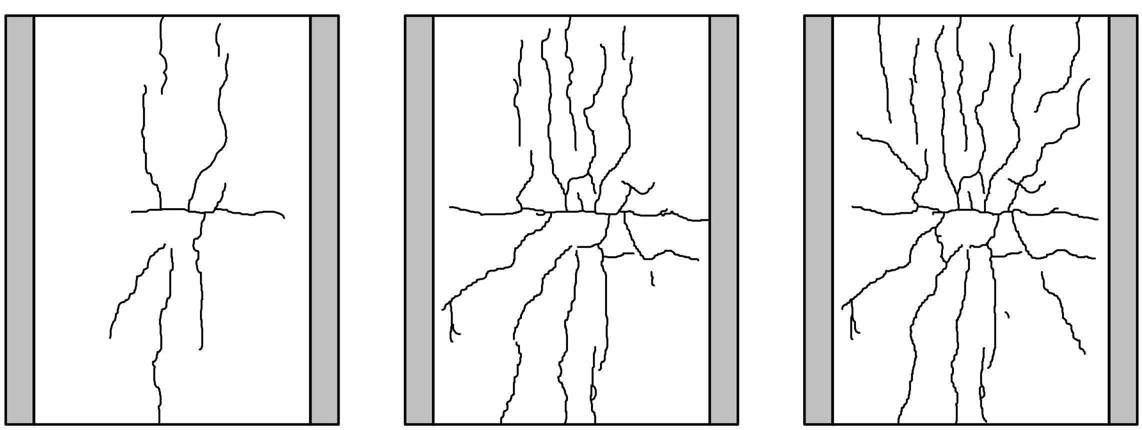
a) T1L2 b) T2L2

Fig. 3 Strengthening schemes



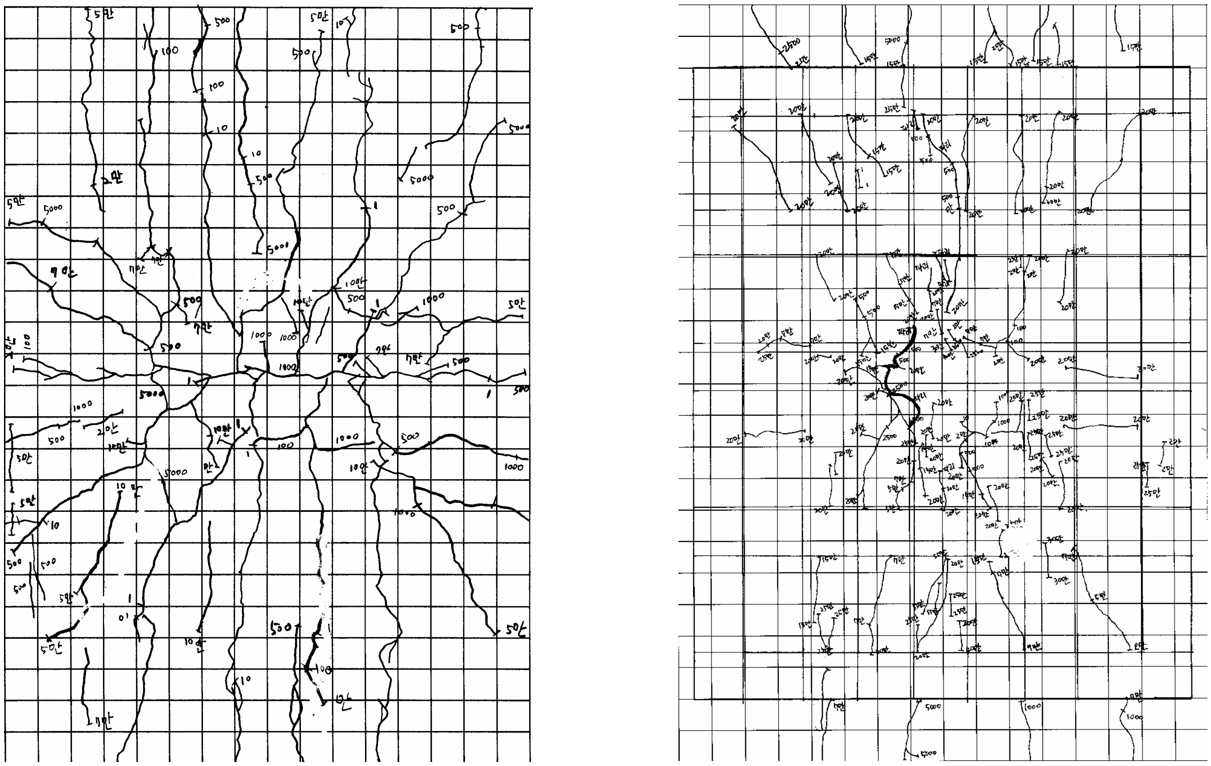
(a) CON (b) T1L2 (C) T2L2

Fig. 4 Crack patterns of deck panels subjected to monotonic loading



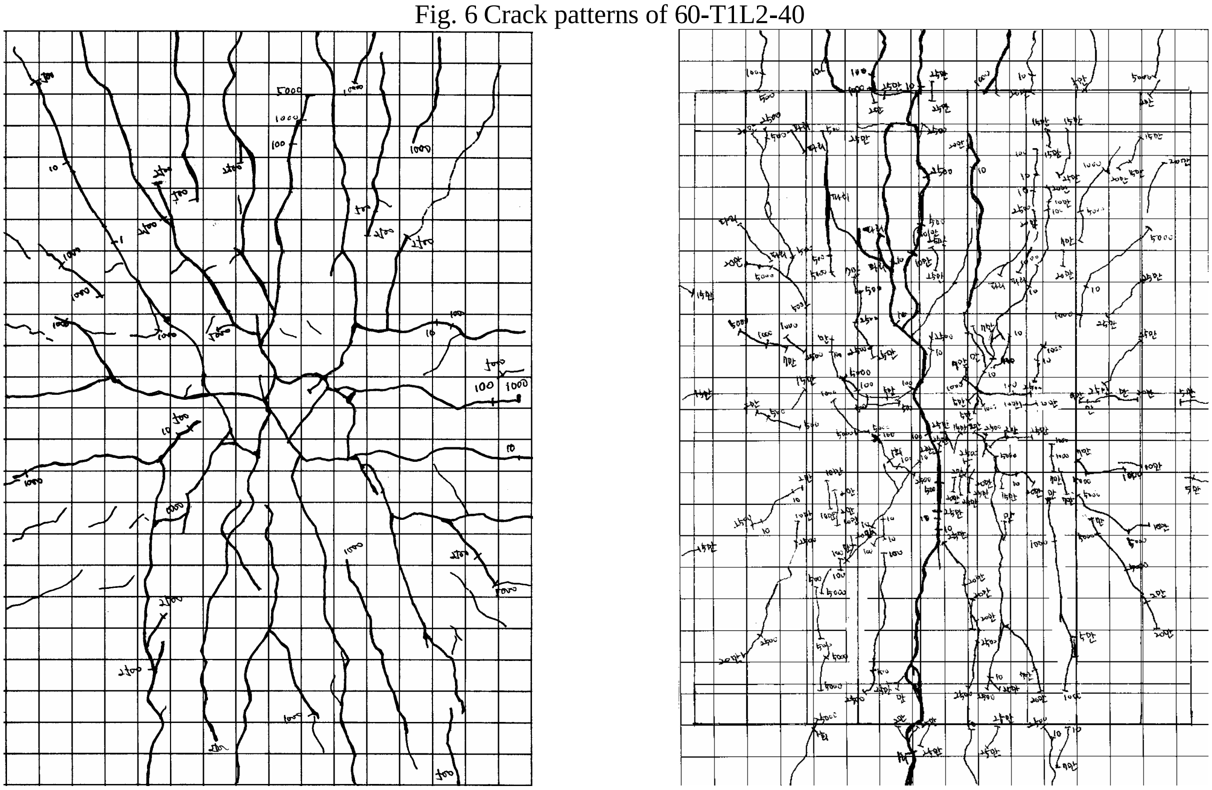
(a) N=10 cycles (b) N=1,000 cycles (c) N=50,000 cycles

Fig. 5 Fatigue crack propagation of unstrengthened panel



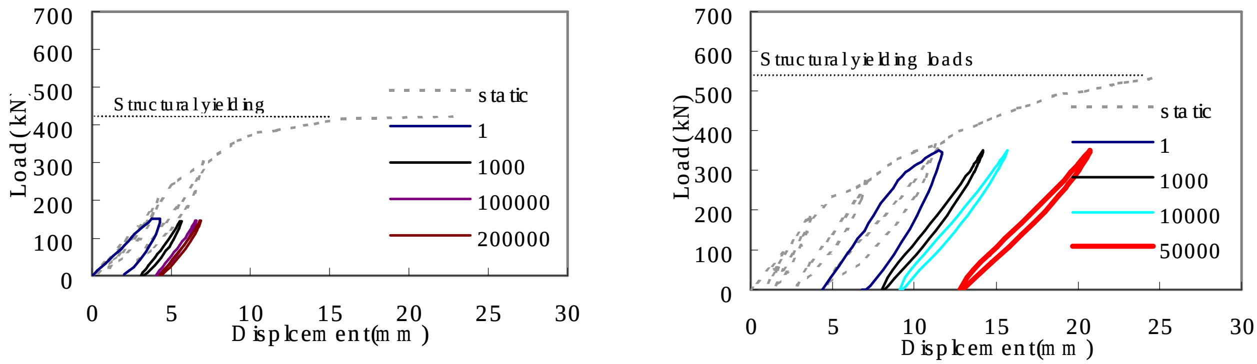
(a) before strengthening (b) after strengthening

Fig. 6 Crack patterns of 60-T1L2-40



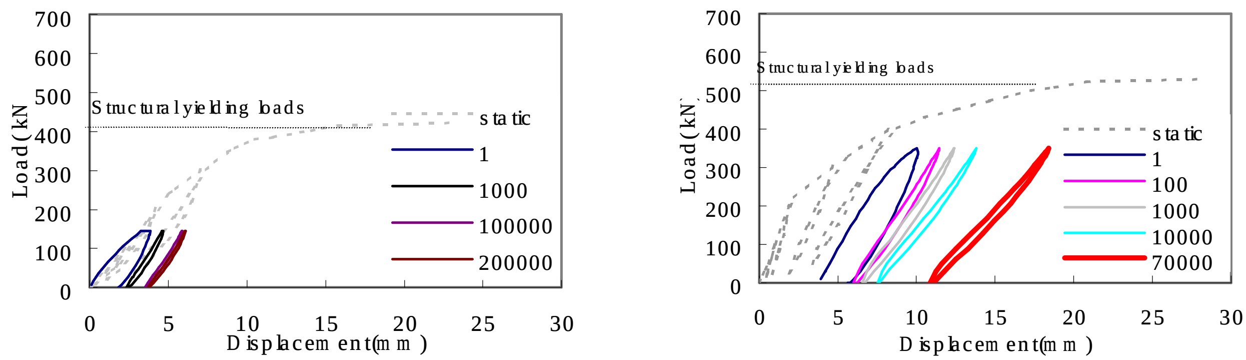
(a) before strengthening (b) after strengthening

Fig. 7 Crack patterns of 60-T2L2-50



(a) before strengthening (b) after strengthening

Fig. 8 Load-displacement relationships through pre-damage and fatigue loading (40-T1L2-70)



(a) before strengthening (b) after strengthening

Fig. 9 Load-displacement relationships through pre-damage and fatigue loading (40-T2L2-80)

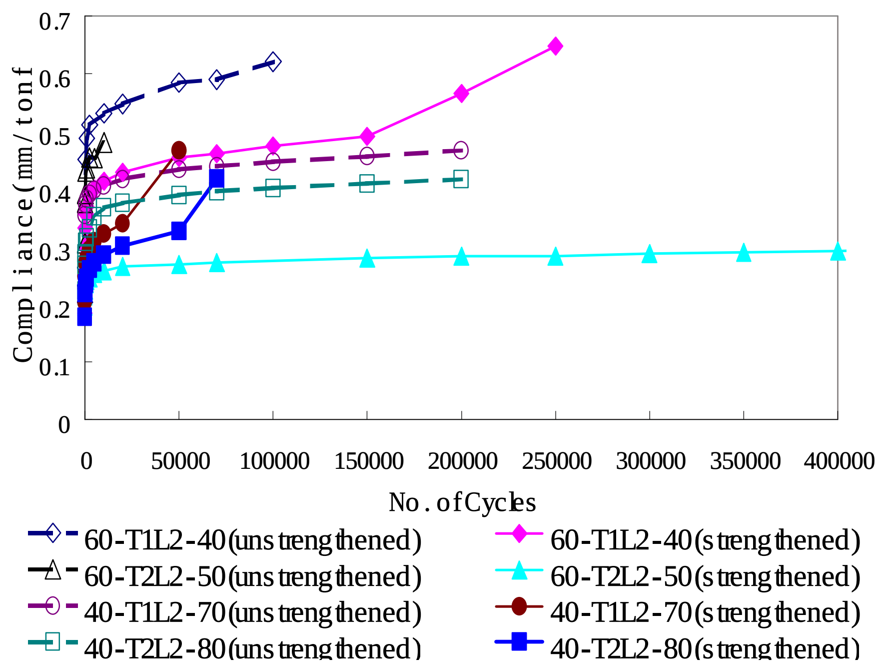


Fig. 10 Time histories of compliances

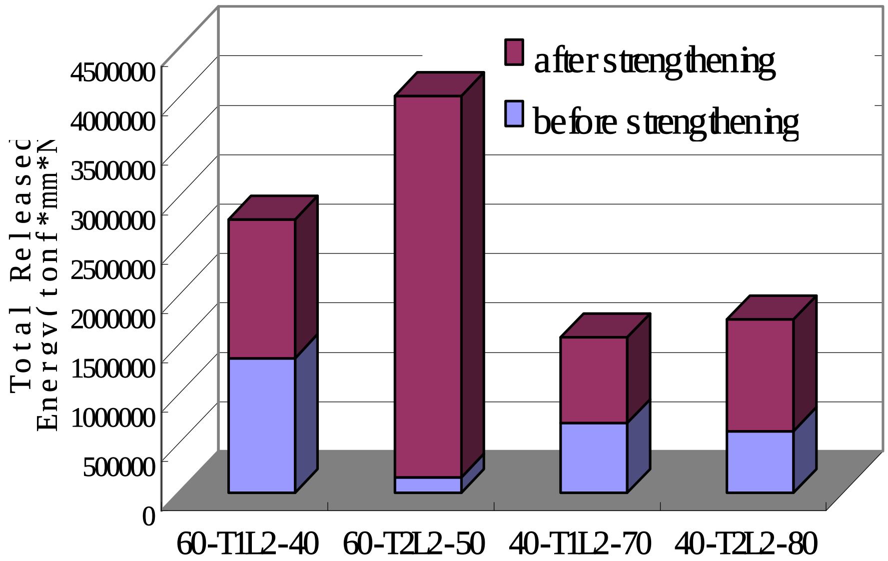


Fig. 11 Total dissipated energy

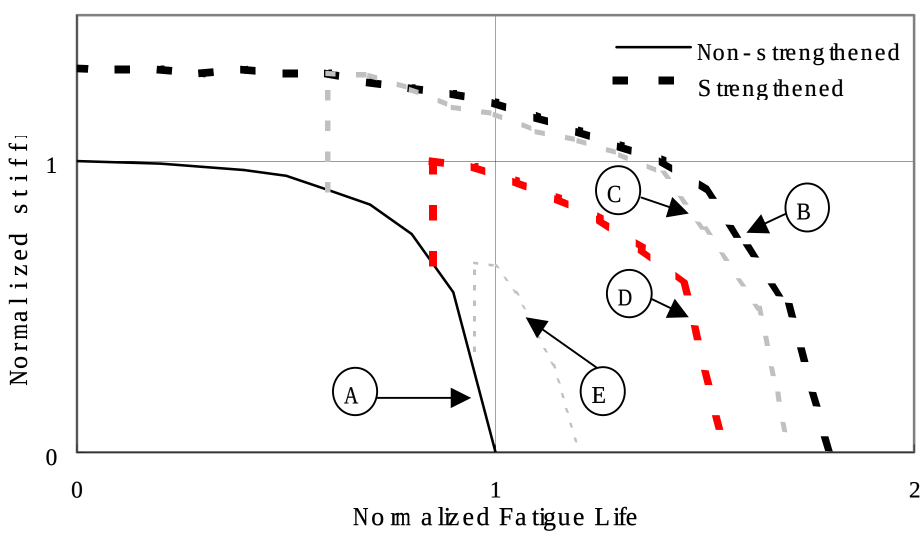


Fig. 12 Schematic effect of strengthening on fatigue life and stiffness

(strengthening applied at four different times)

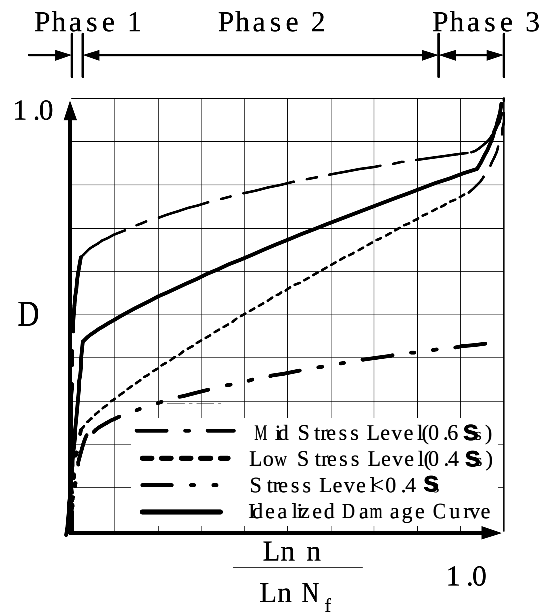


Fig. 13 Idealized damage accumulation