The Effectiveness of a Retrofit Method for Cantilever Slabs Using Reinforced Concrete Beams

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A new retrofit method was developed, geared mainly for cantilever rigid-frame viaducts, which involves the installation of RC beams in each of the viaduct’s columns to improve cantilever performance against strong wind loads, generated by noise prevention barriers which are higher than before. This paper describes the experiments and FEM analyses conducted to evaluate the effectiveness of this method. Firstly, confirmation was obtained that this method can be used on cantilever viaducts to increase resistance. Secondly verifications were made to ensure that that this method allows cantilevers to support barriers of about 5 m. Finally, a method was proposed which can be used to calculated the allowable bending strength using this method.

Keywords: retrofit, renewal, cantilever slab, wind load

1. Introduction

New large-scale repair or retrofit methods are increasingly in demand as the number of aged reinforced concrete (RC) rigid frame viaducts have increased [1]. Furthermore measures involving the installation of higher sound barriers to abate railway noise are being introduced, and therefore new retrofit methods have been proposed to carry out this work [2]. Figure1 shows that after applying these methods, the wind load exerted on the sound barriers increases the force acting on the cantilever slabs. As a result, the flexural capacity of the aged RC cantilever slabs around their ends and the reinforcement terminations may become insufficient.

To overcome this problem, a new retrofit method has been developed to increase the flexural capacity of cantilever slabs. The new method adds new RC beams discretely (retrofit beams) beneath the column-beam connections to raise flexural capacity of the cantilever slabs, not partly but totally. Figure 2 illustrates the new method which transforms the aged cantilever slabs around the tension flanges into new T-shaped beams. This paper confirms the performance of this new method through loading tests and 3D finite element method analyses. This study also identifies the effective area of the tension flange of T-shaped beam which is necessary for calculating the flexural capacity of the cantilever slab in design.

2. Performance of aged cantilever slabs

The bending moments acting on a cantilever slab are classified into general and accidental moments. General moments are generated by the dead load of the cantilever slabs and the sound barriers. Their magnitude at a given point depends on the distance of the point from the joint of the cantilever slab. Accidental moments are generated...
mainly by the wind load, and act uniformly on the cantilever slab while their magnitude is determined by the square of the height of the sound barriers. It follows, as shown in Fig. 3, that as the height of these barriers increases, the size of accidental moments exceeds general moments. Figure 4 shows the result of verification of the bending moment when the wind load acts. The wind load is set at 3.0 kN/m² which is equivalent to 50 m/sec wind speed in "Design Standards for Railway Structures and Commentary (Concrete Structures) (RC Design Standard)" [3]. It is also assumed that the actual cantilever slab has reinforced termination and RC barriers. Figure 4 shows, if the height of the sound barrier is over 4.0 m, the value of verification is over 1.0. In other words, the acting bending moment is bigger than the flexural capacity, and this needs some retrofit work at the reinforcement termination and the end point of the cantilever slab. However, at present, it is generally difficult to improve the performance of cantilever slabs with current retrofit methods. Therefore, if the flexural capacity of cantilever slabs becomes insufficient due to higher sound barriers, new walls supported by new poles are installed in the vicinity of aged RC rigid frame viaducts.

3. Development of the new retrofit method

3.1 Loading tests

3.1.1 Outline of the test

Table 1 shows the major characteristic of the specimens. There are 2 specimens. No.1 is an unreinforced specimen and No.2 is a reinforced specimen. They were not given reinforcement terminations because the purpose of these tests was to focus on checking the improvement in performance after the retrofit involving addition of the retrofit beams. The specimens were half the size of actual RC cantilever slabs. The retrofit beams were the same width as the columns and the same height as the longitudinal beams. The retrofit beams were installed at the same intervals as column. Table 2 shows the material test results.

In the actual construction, the undersurface of aged cantilever slabs is chipped and anchor bars are installed to join the retrofit beams and aged RC cantilever slabs. In these tests however, in order to confirm the effective area of retrofit beams, retrofit beams and cantilever slabs were constructed as a monolithic body.

In this loading test, four horizontal jacks were set at intervals of 1.25 m, and five vertical jacks intervals of 1.0 m. Table 3 shows that in this loading procedure, the dead load is first set as a constant value, and then the wind load is set with gradual increments (Step 1). As wind load was gradually increased however, cracks appeared at the root of the wind barrier and the load decreased. Under the decreased load, it was confirmed that the tension reinforcements of the cantilever slabs did not yield. Then, after all

![Fig. 3 Moment acting on the cantilever slab](image-url)

![Fig. 4 The height of soundproof wall and verification (Flexural capacity)](image-url)

| Specimen | Dimension of beam | Reinforcement |
|----------|------------------|---------------|
|          | Width (mm) | Height | Effective | Cantilever | Diameter | Reinforcement | Ratio of tension |
|          | Joint (mm) | Top (mm) | length (mm) |           | interval | standard | reinforcement |
| No.1     | 5000     | 200  | 100     | 175        | 1400     | D10-100 | SD295          | 0.41(%)        |
| No.2     | 5000     | 200  | 100     | 175        | 1400     | D10-100 | SD295          | 0.41(%)        |
| Retrofit | 300      | 600  | 500     | 575        | 1400     | D16-80  | SD345          | 0.35(%)        |

※ Ratio of tension reinforcement of retrofit beam=0.66% (D16-200, SD345)
the loads were removed, only the dead load gradually increased again (Step 2). This result indicates that it may be difficult to assume a situation where the bending moment is generated only by the gradual increase in dead load. Nevertheless, this loading procedure was adopted to investigate the final failure behavior of the cantilever slab fitted with retrofit beams, and the effective width of the tension flange at the root of the cantilever slabs.

3.1.2 Loading test results

(1) Failure State

Figure 7 shows the failure state of the surface of the cantilever slabs. No.1 had bending cracks generated in the longitudinal direction. No.2, in Step 1, had bending cracks at the foot of the cantilever slabs where the bending moment reached its maximum, and at the end where the height of the slab is at the minimum. In Step 2 of No.2, the number of cracks at the root of the cantilever slab increased. No.2 had diagonal cracks starting from the end of the cantilever slabs with the retrofit beams at the center.

(2) Load-displacement relationship

Figure 8 (a) shows the load-displacement relationship in Step 1. The moment at the joint of the cantilever slabs is plotted on the vertical axis and the horizontal displacement of the middle of the specimen is plotted on the horizontal axis. Compared with the bending stiffness of No.1, that of No.2 was higher, and the maximum bending moment increased 1.5 times. Nonetheless, before the longitudinal bar yielded (about 1200 μ), both specimens had cracks around the joints of the wind barriers and the load decreased.

Figure 8 (b) shows the load-displacement relationship in Step 2. Like Step 1, compared with the bending stiffness of No.1, that of No.2 was high. The maximum load of No.1 was 298 kN · m, and that of No.2 was 537 kN · m.

As explained above, the effectiveness of the retrofit
method was confirmed. Furthermore, the anchor bars of the retrofit beams did not yield after the maximum load was applied and both specimens suffered bending failure.

3.2 FEM analysis

3.2.1 Investigation

The specimens tested in the previous section were half the size of the actual structure. So, to confirm the effectiveness of the retrofit method for actual size structures, full size 3D FEM analyses must be carried out. Following confirmation of the validity of the 3D FEM analysis model which reproduced a loading test, a 3D FEM analysis of the actual structure was conducted.

3.2.2 3D FEM model

The program code used for the analysis was the multipurpose finite element software "DIANA (ver.9.4.4)". Concrete elements were modeled as solid elements, and nonlinearity of the cantilever slab and retrofit slab were considered. The compression softening characteristic of stress-strain relationship and the tension softening characteristic of stress-strain relationship were considered by adopting the Parabolic model and Hordijk model, respectively. The sound barriers were modeled as resilient material. Reinforcements were assumed to be elaspointlasticity with due consideration for nonlinearity, and were modeled as the embedded reinforcement element. It was assumed that the retrofit beam and the cantilever slab were constructed monolithically. Accordingly, the interface element between the retrofit beam and the cantilever slab was not considered.

3.2.3 3D FEM analysis reproducing loading tests

Figure 8 shows the load-displacement relationship based on the result of 3D FEM analysis. The values in Step1 and Step 2 were mostly the same as the experimental values. Figure 9 shows the maximum principal strain of the surface of the cantilever slab in Step 2 in which the dead load was increased gradually. In this analysis, the maximum principal strain extended in the diagonal direction starting from the end of the cantilever slabs. This confirmed that the crack pattern was similar to that observed during the loading tests, thus validating the analysis model.

3.2.4 3D FEM analysis of model assuming an actual structure

(1) The outline of the analysis

The analysis assuming the model of an actual structure was carried out using the model validated in the previous chapter. Figure 10 shows the analysis model.
The model was given a sound barrier which was 5 m high. Table 4 shows the case for analysis. The parameters of the model were the length of cantilever slab and the column pitch which was the same pitch as the retrofit beam. The model had one reinforcement termination taking into account ballast weight in addition to the dead load of the cantilever slab and sound barrier. The shape and bar arrangement in this model were based on ordinary railway rigid frame viaducts.

(2) Result of the analysis

Figure 11 shows the load-displacement relationship for a column pitch of 10m. The horizontal axis was the displacement at the top of the cantilever slab. Figure 11 (a) shows the case of wind load increasing and the 2.8 m cantilever slab. From the result, it was confirmed that the top area of cantilever slab yielded first because it had the smallest cross-sectional height. When the tension reinforcement yielded, the moment \( M_y \) in the retrofit model \( (R-L2.8-I10-H) \) was 1070 kN \( \cdot \) m while in the model without the retrofitted reinforcement \( (E-L2.8-I10-H) \), \( M_y \) was 720 kN \( \cdot \) m. Thus, \( M_y \) on the no-retrofit model \( (E-L2.8-I10-V) \) was 2318 kN \( \cdot \) m, while on the retrofitted model \( (R-L2.8-I10-V) \) it was 2318 kN \( \cdot \) m. Figure (a) also shows that the retrofit effect is confirmed.

3.3 The investigation of the effective width of the tension flange

3.3.1 The outline of the investigation

The effective width of the tension flange necessary for calculating the yield moment and the flexural capacity of the cantilever slabs reinforced by retrofit beam was investigated based on the strain distribution in the longitudinal...
direction [4]. To put it concretely, the effective width of the tension flange was calculated on the basis of the strain distribution obtained through FEM analysis, and then, the calculated value of the yielding moment assuming that the fiber strain is proportional to the distance from the neutral axis of the member section, was compared with the calculated value by the 3D FEM analysis. The effective width of the tension flange means the width of reinforcement which can be taken into account when the yielding moment and the flexural capacity are calculated. In the “RC Design Standard,” the effective width of the compressive flange is stipulated, but, in general, the effective width of the tension flange is not taken into account in the design of new structures. However, this new retrofit method was developed as a countermeasure against the existing old structure. Thus, from the view point of investigating the performance of the cantilever slab after a retrofit, it is assumed that the tension reinforcement is set with an identical diameter and identical intervals. As a result, it is assumed that the effective width of the tension flange can be taken into account when calculation of the capacity is carried out. However, when the wind load is exerted from outside the track, the cantilever slab should act as a compressive flange. Nevertheless, it was found that the performance of cantilever slab can be calculated according to “RC Design Standard.”

3.3.2 Stress distribution of the tension reinforcement

Figure 12 (a), (b) show the strain distributions at the end of the cantilever slab: (a) shows the result of the experiment, and (b) shows the result of 3D FEM model of the existing structure when the wind load increases gradually. The focus point is the end of the cantilever slab which yielded first. These figures show the strain distributions for the cases where the respective values reached mostly the maximum values of 500 micro, 1000 micro, and 1500 micro (for (a), 1200 micro at the finish of Step 1).

The value rose as the distance from the retrofit beam fell, because the retrofit beam which has the higher stiffness than that of the other parts bears the main part of the load. From this result, the stress distribution forms an upward convex shape with its center at the retrofit beam. Strain distribution however, of the tension reinforcement at the joint of the cantilever slab which is higher than the end of the cantilever slab, does not vary, in general, for moments corresponding to 1500 micro strain at the end of the cantilever slab.

Figure 12 (c), (d) show the strain distribution of the reinforcement at the cantilever slab joint, when the dead load increases gradually. This case should be considered as a reference, because the dead load barely increases in actual structures. As when wind load increases, the strain distribution forms an upward convex shape with its center around the retrofit beam. Comparing (b) and (d), the width of the up-facing convex shape is smaller at the end of the cantilever slab than that at the joint between cantilever slabs. In other words, this means that the effectiveness of the retrofit is weaker at the end of cantilever slab than on other parts.

Fig. 12  Strain distribution of reinforcement from the result of FEM analysis
3.3.3 Calculation method of the effective width of the tension flange

Based on the strain distribution of the tension reinforcement, the effective width \((be')\) of the tension flange was calculated. As in the method for calculating the effective width \((be')\) of the compressive flange in T-shaped beams, as shown in Fig.12 (c), first, the width \(be'\) is defined as the effective range of the tension flange, and then, the width \(be'\) is calculated so that the shaded area equals the whole strain area. As previously mentioned, there is a difference between strain distribution at the end of the cantilever slab and at the cantilever slab join, both of which are upward convex shapes with their centers around the retrofit beam. The strain range for calculating the effective range was thus defined as the range up to the point where the inflection point is obvious. This range was defined as the effective flange width.

In the design of actual structures, the ends and joints of cantilever slabs and reinforced terminations need to be verified. The effective flange width \(be'\) was then calculated for the longitudinal direction of the cantilever slabs. Figure 13 shows the result. On the horizontal axis, the longitudinal length of the cantilever slab is plotted, but the length is made dimensionless to exclude the influence of the size of the experimental model and the actual structure model. The vertical axis shows the quotient of \(be'\) divided by \(bw\), the width of retrofit beam, which is 300 mm for the experimental model and 600 mm for the actual structure model. When wind load increased, the value was calculated from the strain distribution at each distance from the joint when the strain of the tension reinforcement at the end of the cantilever slab reached 1500 micro. For increase in dead load, the value was calculated from the strain distribution at each distance from the joint when the strain of the tension reinforcement at the joint of the cantilever slabs reached 1500 micro.

Figure 13 (a) shows the results obtained when the wind load was increased gradually. The tendency in the results obtained through the experimental model agrees closely with that of the actual structure model regardless of the longitudinal length of the cantilever slab. The effective flange width \(be'\) is generally the constant value of \(2bw\). Figure 13 (b) shows the results obtained when the dead load was increasing gradually. This case was just for reference. As opposed to when the wind load increased, the effective flange width \((bw)\) demonstrated a tendency to fall when approaching the end of the cantilever slab. When the columns at 10 m intervals were compared with those at 6 m intervals, the effective flange width of the columns at 10 m intervals tended to be smaller. The effective flange width was generally \(10bw\) around the joints of the cantilever slabs.

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\text{Table 5} \quad \text{Calculation result for endurance capacity}
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| Case          | Verification Point | Mycal (kN \(\cdot\) m) | My (kN \(\cdot\) m) | Effective width \((\times bw)\) | Mycal/My |
|---------------|-------------------|-------------------------|---------------------|-------------------------------|-----------|
| E-L2.8-I10-H  | Top               | 498                     | 535                 | 2                             | 0.93      |
| E-L2.3-I10-H  | Top               | 498                     | 535                 | 2                             | 0.93      |
| E-L2.8-I6-H   | Joint             | 2042                    | 1884                | 10                            | 1.08      |
| R-L2.8-I10-H  | Joint             | 2042                    | 2220                | 10                            | 0.92      |

4. Conclusion

The new method of retrofitting RC beams discretely at the joint of column-beams was proposed as a measure
to raise the flexural capacity of cantilever slabs, not partly but totally. Based on experiments and 3D FEM analyses of ordinary RC rigid frame viaducts with column intervals of 10 m and with longitudinal beams of about 1m in height, the following knowledge was obtained:

It was confirmed that the flexural capacity of the actual cantilever slab could increase by adding retrofit beams with a similar width to the columns and height as the longitudinal beams respectively, and at the same intervals as the columns.

It was confirmed that, in the case where the wind load of 3.0 kN/m² acts on a sound barrier of height 5.0 m from the outside of the track, the new retrofit method could assure sufficient capacity against yield and flexural moments.

Based on the investigation of the effective tension flange width of the cantilever slab, assuming that the cantilever slab is the tension flange of the T-shaped beam, it was confirmed that the effective tension flange width is evaluated as being twice of the width of the retrofit beam.

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