PROPOSAL OF DEAD WEIGHT COMPENSATION STRUCTURES AND APPLICATION TO RAILWAY VIADUCT

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Abstract

In Japan the seismic design standard for railway facilities defines the L2 design earthquake motion as “the maximum earthquake motion.” However, the seismic design standard states that it is not the maximum possible physical earthquake motion, and setting of the L2 earthquake should be based on advanced engineering judgment. Furthermore, the seismic design standard does not deny that earthquake exceeding L2 design earthquake motion will occur. How to take measures against such unanticipated earthquakes has become a matter of significant concern after the 2011 off the pacific coast of Tohoku Earthquake. The seismic design standard does not deny the possibility of the critical stage considered in design being exceeded but requires some “consideration” for such unexpected circumstances. It is referred to as “anti-catastrophe” in the design standard. As a measure to improve the anti-catastrophe, the authors have already proposed “a dead weight compensation structure” which prevent a collapse by supporting a superstructure even when columns would be destroyed in an earthquake beyond anticipation. The dead weight compensation structure consists of two types of columns; normal columns, dead weight compensation columns. The normal columns are designed to satisfy the seismic performance against the L2 earthquake. On the other hand, dead weight compensation columns support a superstructure when normal columns are destroyed in an earthquake beyond anticipation, although they do not resist the inertial force at the time of L2 earthquake. To prevent the dead weight compensation columns from being damaged by an earthquake beyond anticipation, top of the dead weight compensation column and the superstructure are not jointed: sliding bearings are installed between the capitals of the dead weight compensation columns and the superstructure. In this study, large-scale shaking table tests targeting RC rigid frame viaducts were carried out. The experiment results clarified that dead weight compensation columns can support the superstructure even when the normal columns are destroyed, and dead weight compensation columns undergo cyclic earthquake loading. Then test designs for newly founded RC rigid frame viaducts with dead weight compensation structure were conducted, aiming at practical application. According to the test result, maximum response displacement of superstructure became larger than the conventional structure because the number of columns which resist against L2 design earthquake motion was less than those used for conventional structure. Nevertheless, the test design results show that it is possible to construct the proposed RC rigid frame viaducts which satisfy required performance such as safety and restorability against L2 design earthquake motion without increasing the column size or reinforcement ratio of the conventional RC rigid frame viaduct. Finally, a test design for existing RC rigid frame viaducts with dead weight compensation structure was conducted. The test design results showed that it is possible to apply the dead weight compensation structure without reinforcing the existing superstructure or footing beams by installing steel columns adjacent to the existing columns.

Keywords: anti-catastrophe; dead weight compensation structure; required performance

1. Introduction

The seismic design standard for railway facilities defines the L2 design earthquake motion as "the maximum earthquake motion." However, the seismic design standard states that it is not the maximum possible physical earthquake motion, and setting should be based on advanced engineering judgment. Furthermore, the seismic design standard does not deny that earthquake exceeding L2 design earthquake motion will occur. How to take measures against such unanticipated earthquakes has become a matter of significant concern after the 2011 off the pacific coast of Tohoku Earthquake. The seismic design standard does not deny the possibility of the critical stage considered in design being exceeded but requires some "consideration" for such unexpected circumstances. It is referred to as "anti-catastrophe" in the design standard. [1]

As a measure to improve the anti-catastrophe, some of the authors have already proposed a "Dead Weight Compensation structure," or DWC structure [1, 2, 3] which prevent a collapse by supporting a superstructure even when column would be destroyed in an earthquake beyond anticipation, as shown in Fig. 1. The DWC structure consists of two types of columns; normal columns, dead weight compensation (DWC) columns. The normal columns are designed to satisfy the seismic performance against the L2 earthquake. On the other hand, DWC columns support a superstructure when normal columns are destroyed in an earthquake beyond anticipation, although they do not resist the inertial force at the time of L2 earthquake. To prevent the DWC columns from being damaged by an earthquake beyond anticipation, top of the DWC column and the superstructure are not jointed: sliding bearings are installed between the capitals of the DWC columns and the superstructure.

Since structures in which DWC structure was installed most frequently were assumed to be RC rigid viaducts, some of the authors have already conducted large-scale static loading tests of model RC rigid frame viaducts and confirmed applicability of DWC structure to RC rigid viaducts [4]. On the other hand, it is necessary to verify the feasibility of DWC structure by conducting shaking table tests of model viaduct or test design of practical scale viaducts, aiming at practical application.

Therefore, large-scale shaking table tests targeting RC rigid frame viaducts were carried out in this study. In addition, test designs for newly founded RC rigid frame viaducts with DWC structure was conducted, aiming at practical application. Finally, test design for existing RC rigid frame viaducts with DWC structure was conducted.

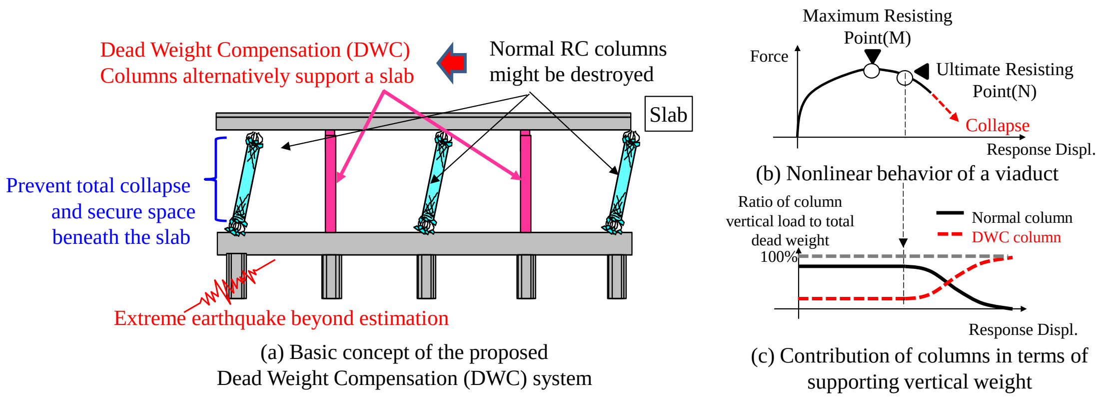


Fig. 1 – Schematic view of dead weight compensation structure [1, 3 (partially corrected)]

2. Concept of the Proposed DWC structure

As shown in the previous chapter, applicability of DWC structure to RC rigid viaducts was confirmed by the large-scale shaking table tests. Therefore, test designs to apply the DWC structure to actual RC rigid viaducts were conducted.

In this chapter, the design concept of the DWC device is described. In developing a new device to be applicable to the real structures, a reinforced-concrete (RC) rigid frame viaduct was selected as a target structure, since there are numerous number of viaducts in use for both railway and road structures. Fig. 1(b) shows a typical nonlinear behavior of a viaduct. In general, such a viaduct is designed so that structure would be resilient enough to enable rapid repair under designated design motions. It would be verified by confirming that response displacement of a structure is restrained below the maximum resisting level (Point M in Fig. 1(b)). Even if the response slightly exceeds maximum resisting displacement, a structure would be still safe enough to avoid a total collapse.

On the contrary, a structure gradually or rapidly loses its resisting capacity and goes to a total collapse if the induced earthquake exceeds the predetermined design level and vertical columns are severely damaged. The proposed device was intended to take effect under such a circumstance and prevent the total collapse. Fig. 1(a) shows the overview of the proposed DWC structure. As shown in the figure, supplemental “Dead Weight Compensation” columns are installed in parallel with normal columns. This DWC column prevents the total collapse of a viaduct by holding the vertical weight of the slab even if normal columns are fully destroyed.

Fig 1(c) shows schematic of the desirable distribution of the vertical weight of slab to both normal and DWC columns. As illustrated, the DWC column would gradually replace the function with respect to the vertical support of normal columns after exceeding the ultimate resisting point (Point N).

3. Experimental Verification [3]

3.1 Test Setup

The effectiveness of proposed device was investigated by a shake table test. In the series of tests, a viaduct model shown in Fig. 2 was constructed on the table and excited repeatedly until it came to a complete collapse. The proposed DWC columns were also employed in the specimen as illustrated in Fig. 2. The effectiveness of the proposed structure was confirmed by checking whether the DWC column successfully supported the slab and prevented the total collapse under strong motions.

As shown in Fig. 2, a rigid frame model consisted of total eight reinforced concrete columns, slab and foundation. The size of a slab was W2000 mm x D4700 mm x H500 mm, and they were supported by four normal RC columns. The size of a column was W200 mm x D200 mm x H1400 mm with reinforcing bars of 8-D10 (SD295). This specimen was designed so that it would be approximately 1/4 of real railway viaduct. The DWC columns were allocated at each corner of the slab. The dead weight on the slab was given by mounting a bunch of steel blocks on the slab. Consequently, section stress on each normal column was 1.72N/mm^2. According to the preliminary static analysis, yielding coefficient and maximum resisting displacement of the viaduct were 0.4 and 34.1 mm, respectively.

3.2 The DWC Column and Device

Fig. 3 shows the detail of the DWC column. The specification of the DWC column was identical to that of the normal column shown in previous section, excepts that a Teflon device was embedded on top of the DWC column. In addition, the steel plate was attached to the bottom of slab. The horizontal reacting force between the DWC column and the slab was restricted to the small extent due to the small friction coefficient between a Teflon and a steel plate (approximately 0.1). It followed that the DWC column will not be severely damaged due to the horizontal inertial force from the slab even after the vertical weight of the slab is totally induced to DWC columns.

3.3 Measurement and Excitation

For data acquisitions, absolute accelerations of slab and table as well as relative displacement between slab and table in horizontal and vertical directions were mainly measured. In addition, multi-directional load cells were embedded in-between columns and DWC device to measure reacting forces with respect to vertical and horizontal directions. See Fig. 2 for distribution of sensors.

The specimen was excited in a transverse direction using a Level-2 Spectrum I acceleration for a G3 soil condition. This waveform is a general surface motion in a good soil condition due to an inter-plate earthquake designated in the Japanese railway design standard [5]. This motion was selected because of its long duration time, by which the repetitive motions would be induced to the viaduct model. The time scale of the waveform was compressed to 1/2 of the original one to meet the law of similarity.

In the series of tests, maximum acceleration of the waveform was gradually increased from 100 gal (No.1) to 1300gal (No.13). After test No.13, moderated acceleration (800gal) were induced assuming the aftershock. All test conditions carried out are shown in Table 1.

3.4 Test Results and Discussions

Fig. 4(a) illustrates the snapshot of the specimen after all tests finished. Figs. 4(b) and 4(c) show comparison of bottom of piers with respect to the normal RC column and the DWC column. As shown in Figs. 4(a) and 4(b), normal columns were severely damaged and inclined due to the buckling of reinforce bars and crash of core-concrete. Nevertheless, the total collapse of the specimen did not take place since vertical weight of the slab was supported by supplemental DWC columns. In addition, as observed in Fig. 4(c), only few damage was found in the DWC column since the sliding device on top of the column moderated a horizontal inertial force of the slab.

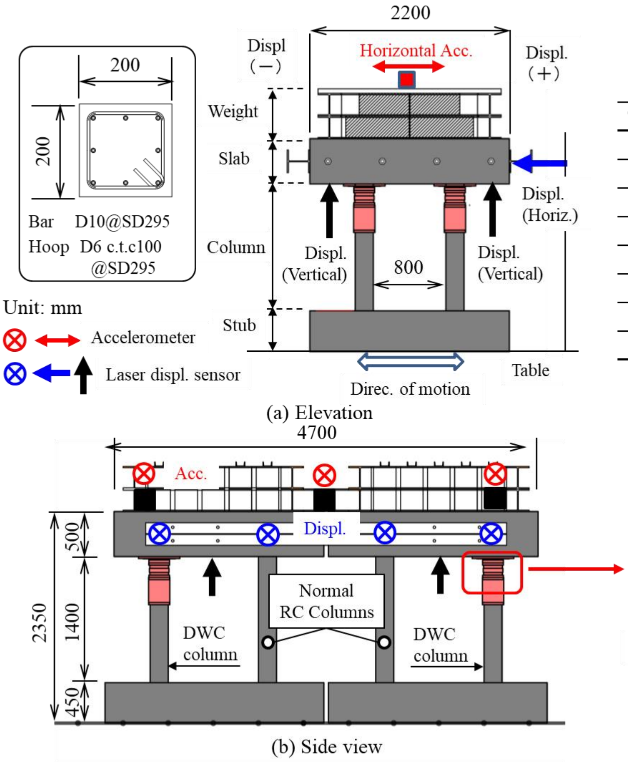


Fig. 2 – Test setup

Table 1 – Test condition

TestNo.	Max.Acc.	TestNo.	Max.Acc.
1	100gal	9	1100gal-1
2	200gal	10	1100gal-2
3	300gal	11	1100gal-3
4	400gal	12	1100gal-4
5	500gal	13	1300gal
6	600gal	14	800gal-1
7	800gal	15	800gal-2
8	1000gal		

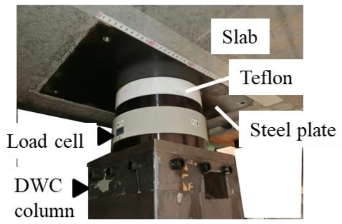
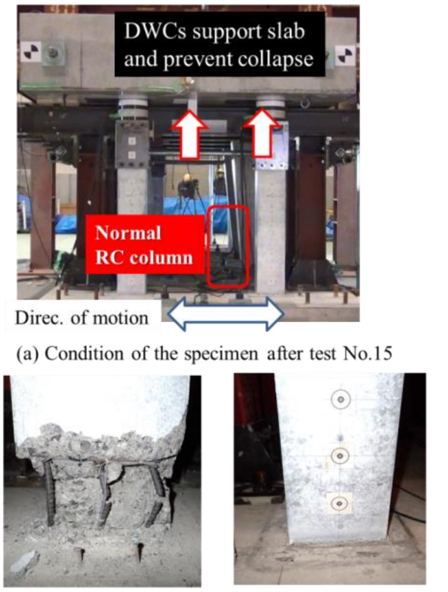


Fig. 3 – Head of a DWC column

Fig. 5 shows the vertical loads of all DWC columns as well as residual vertical and horizontal displacements of slab obtained from the test results. The vertical loads and residual displacements were measured at the end of each test. In Fig. 5, vertical load are expressed in a percentile, ratio of vertical load supported by all DWC columns to the total weight of a slab. It is found that DWC columns began supporting the slab at and after test No. 11, where the horizontal residual displacement drastically increased accordingly. It implies that normal columns were severely damaged and lost their resisting capacity after No. 11. It is, however, noted that the vertical residual displacement did not increase since the DWC columns supported the slab weight. It is also found that the DWC columns kept supporting the slab stably as an alternative to normal columns, and prevented the total collapse (tests No.12-No.15).



(a) Condition of the specimen after test No.15

(b) Bottom of normal column (c) Bottom of DWC column

Fig. 4 – General view of the specimen after tests

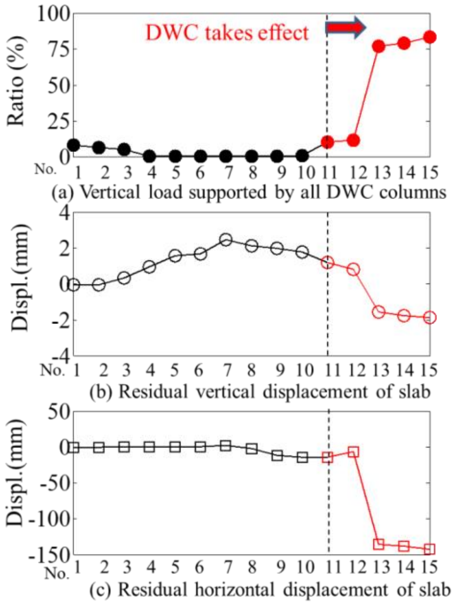


Fig. 5 – Vertical load distribution to DWC column and corresponding residual displacements

4. Design for newly founded RC rigid frame viaducts with DWC structure [6]

4.1 Target structure of test design

Target structure of test design is a railway rigid frame viaduct described in the design example of railway viaduct [7]. The outline of the viaduct is shown in Fig. 5 and ground conditions are shown in Table 2. This viaduct has six columns. In this study, columns of the 2nd and 5th rows are replaced by DWC columns so that the span between DWC columns of the 2nd row and those of the 5th row is not too large.

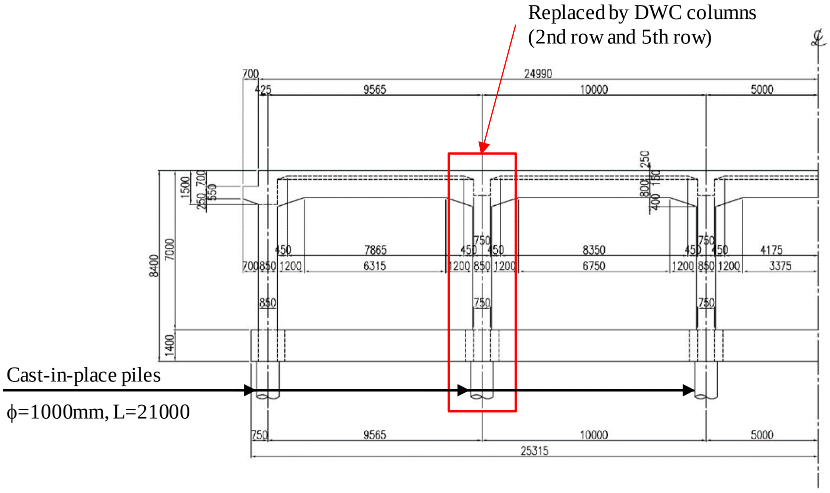


Fig. 5 – The outline of the viaduct [7]

Examination cases are shown in Fig. 6. Case 0 is the examination case for the original viaduct referred from the design example of railway viaduct [7]. Case 1 is the examination case for a viaduct with DWC columns. DWC columns in Case 1 are separated from the slab and will shear vertical load only after normal columns have totally broken. In Case 1, height of the vertical beam is probably large since the span between normal columns of 2nd (5th) row and those of 3rd (4th) row is 20m, which is twice as large as the original viaduct. Case 2 is also the examination case for a viaduct with DWC columns, however, DWC columns in Case 2 always shear vertical load. Therefore, height of the vertical beam probably become almost the same as the original viaduct. Connections between the slab and DWC columns in Case 2 were set to pinned connection when seismic load did not act, while those were modeled by friction spring when seismic load acted, the details are to be described later.

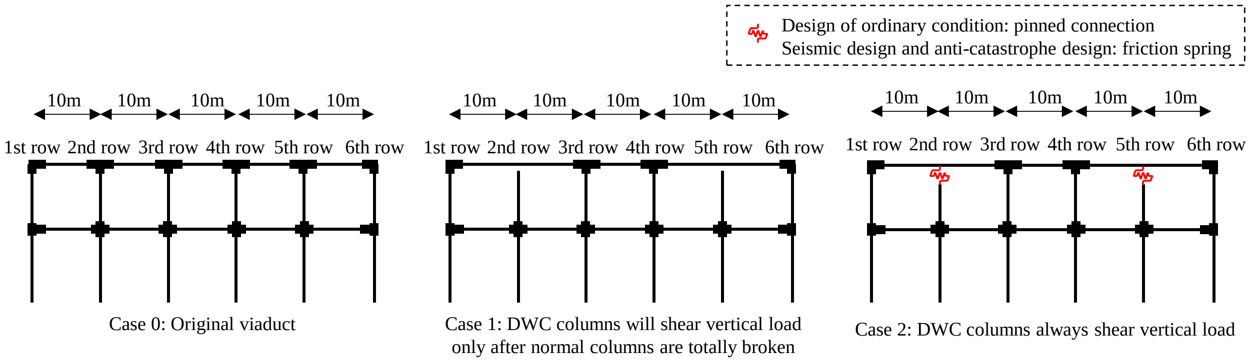


Fig. 6 – Examination cases

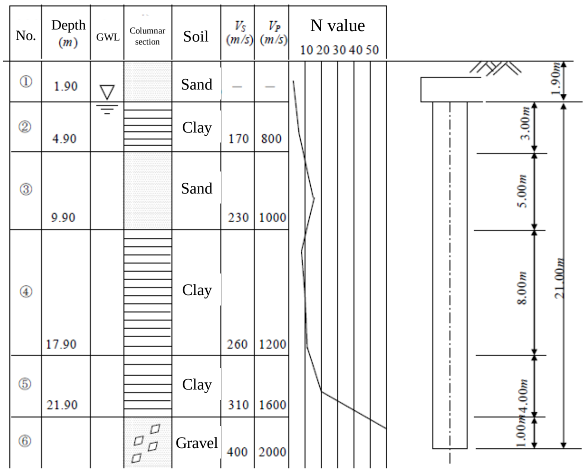


Fig. 7 – Ground conditions

4.2 Outline of the design of DWC structure

4.2.1 Required performance and design verification

Required performance of the rigid frame viaducts with DWC structure was set based on railway design standard as follows.

(i) Ordinary design (design of ordinary condition and seismic design): It is required to satisfy the performance which is specified in the current design code.

(ii) Anti-catastrophe design: It is required to support vertical load which consists of dead load and train load even after normal columns are broken.

Structural specifications were determined so as to satisfy the required performance (ii) as well as the required performance (i). The verification indices are shown in Table 3. Verification indices related to the results shown in this paper are shown in italics.

Table 3 – Verification indices

(a) Design of ordinary condition

Performance items	**Verification indices**
Safety (failure)	Bending moment of vertical beam and underground beam
Safety (fatigue failure)	Bending moment of vertical beam
Safety (running safety)	Flexure of vertical beam
Safety (foundation)	Omitted because if verification of serviceability (foundation) is satisfied, verification of safety (foundation) is also satisfied on the condition of this specification of viaduct
Serviceabiility (aesthetic appearance)	*Bending moment of lower side of vertical beam*
Serviceability (riding comfort)	Flexure of vertical beam
Serviceability (foundation)	*Limited to long-term support performance because increment of limiting value of short-term support performance is larger than fluctuating load*
Serviceabiility (damage)	Bending moment of vertical beam and underground beam
Durability	Bending moment of vertical beam and underground beam

(b) Seismic design

Performance items	**Verification indices**
Safety (failure)	*Bending moment and shear force of columns, vertical beam, horizontal beam, and pile head*
Restorability (damage)	*Bending moment and shear force of columns, vertical beam, horizontal beam, and pile head*

(c) Anti-catastrophe design

Performance items	**Verification indices**
Safety (failure)	*Bending moment of vertical beam and underground beam*
Serviceability (foundation)	Limited to short-term support performance

4.2.2 Analytical model of the viaduct

Since response of the viaduct would be complicated due to difference of stiffness ratio of columns when DWC columns installed, the viaduct was modelled as beam-spring space frame model. Seismic response of the viaduct was calculated by dynamic analysis models shown in Fig. 8, which are constructed by connecting free field and structure with interaction springs. Nonlinearities of free field were modelled by the GHE-S model [5]. Nonlinearities of interaction springs were expressed based on the design standard [8]. Nonlinearities of the viaduct were set by M-ϕ model. Their skeleton curve was modelled as tri-linear model which can consider crack (C), yield (Y), ultimate force (M) and the modified Takeda model was applied as hysteresis rule.

In Case 2, connections between the slab and DWC columns were set to pinned connection when seismic load did not act, while those were modeled by friction spring when seismic load acted. The friction coefficient between the slab and the columns is set to 0.1, which is based on the friction coefficient between a Teflon and a steel plate. Fluctuation of axial force of the friction spring is considered. In addition, in anti-catastrophe design, it is assumed that normal columns retain axial rigidity and flexural rigidity to some extent even after breaking. Although the axial and flexural rigidity of normal columns after breaking have not been clarified enough, it was set to be 2% of the rigidity before breaking [2].

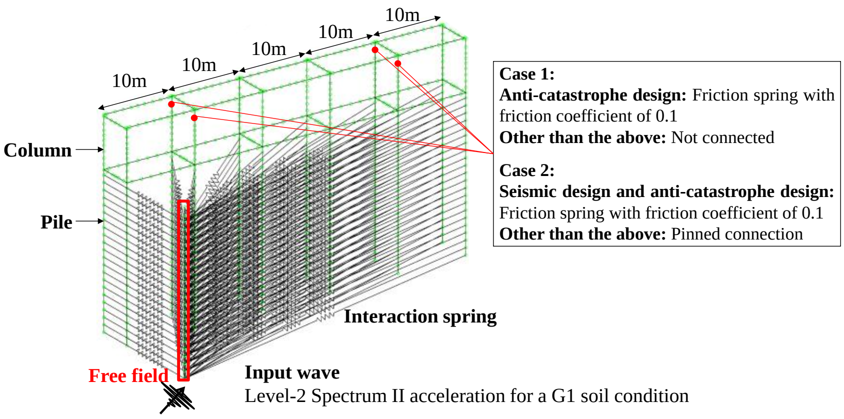


Fig. 8 – Analysis model

4.3 Results and discussion

4.3.1 Structural specifications

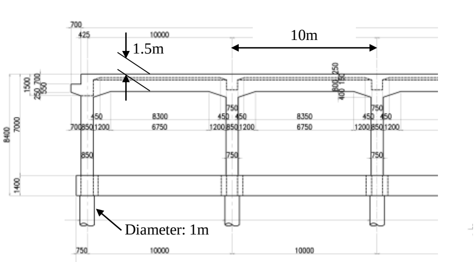
Fig. 9 compares structural specifications of the viaducts with regard to Case 0, Case 1 and Case 2. In Fig. 9, representative modification from the original viaduct are shown. Moreover, cross section dimensions and tension reinforcement ratio of columns are also shown in Fig. 9.

In Case 1, height of the vertical beam is 3m, which is twice as large as the original viaduct, as shown in Fig. 9 (b). This is because the span between normal columns of 2nd (5th) row and those of 3rd (4th) row become large (20m) and verification of flexural cracking is not satisfied. Moreover, increase of weight of the vertical beam follows to increase of pile diameter (set to 1.5 m) so that verification of stability of foundations is satisfied, which leads to increase of construction cost. On the other hand, all verifications are satisfied on the condition that section dimensions and reinforce ratio of normal columns and DWC columns are the same as the original viaduct.

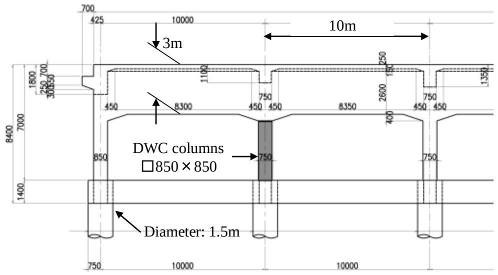
In Case 2, no modifications in span and section dimension were needed from the original viaduct, as shown in Fig. 9 (c). Reinforce bar ratio of normal columns and DWC columns is also the same as the original viaduct on the condition of this study, although the previous study by Nishimura et al. [2] pointed out that it is necessary to increase reinforce bar ratio of normal columns slightly when DWC structure is installed.

4.3.2 Bending moment of vertical beam

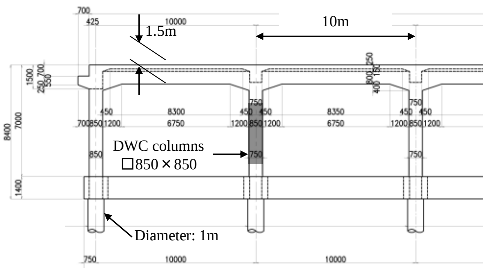
Bending moment diagrams of the vertical beams in the condition of no seismic action are shown in Fig. 10. The maximum absolute bending moment in Case 2 is 1255kN-m, which is approximately 0.13 times as large as that of Case 1. The bending moment diagrams of the vertical beams after normal columns have broken totally are shown in Fig. 11. The maximum absolute of bending moment in Case 2 is 3167kN-m, which is approximately 0.32 times as large as that of Case 1. Therefore, it is found that bending moment of the vertical beams can be reduced significantly on the condition that the DWC columns always share vertical load.



(a) Case 0 (original viaduct)



(b) Case 1 (DWC columns will shear vertical load only after normal columns are totally broken)

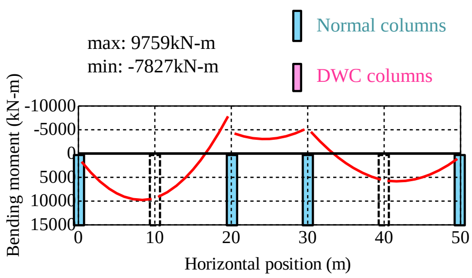


(c) Case 2 (DWC columns always shear vertical load)

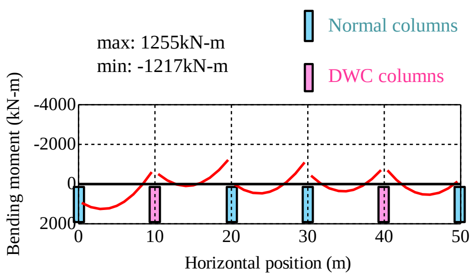
Cross section dimensions	Tension reinforcement ratio of normal columns	Tension reinforcement ratio of DWC columns*
850mm×850mm	0.660%**	0.550%**

* In the original viaduct tension reinforcement ratio of normal columns of 2nd and 5th rows are shown.
** The same value both in the direction of the bridge axis and in the direction perpendicular to the bridge axis

Fig. 9 – Structural specifications of the viaducts based on the design

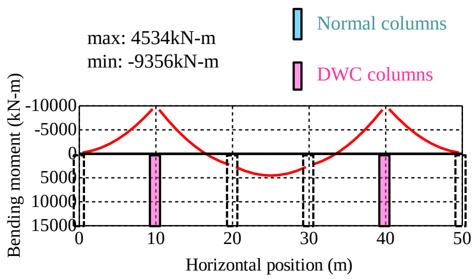


(a) Case 1

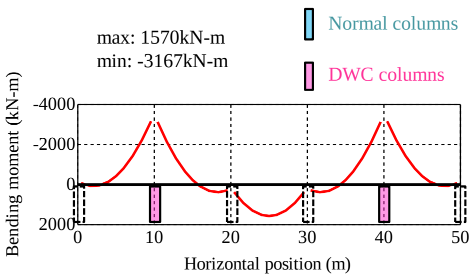


(b) Case 2

Fig. 10 – Bending moment diagrams of the vertical beams in the ordinary condition



(a) Case 1



(b) Case 2

Fig. 11 – Bending moment diagrams of the vertical beams after normal columns have broken totally

4.3.3 Seismic characteristics of models

Table 4 shows equivalent natural periods and yield seismic intensities of each model in longitudinal direction and in transverse direction. Here, yield displacement in Table 4 is the displacement at the crest of the slab. The equivalent natural period and the yield seismic intensity in Table 4 were calculated from the force-displacement relationship based on design standard [8]. The maximum displacement of each model from dynamic analysis is shown in Table 5.

In Case 1, number of the normal columns which resists the inertial force decreased from the original viaduct and weight of the slab increased. As shown in Table 4 (a), however, the yield seismic intensity increases and equivalent natural period become shorter compared than the original viaduct in longitudinal direction. Moreover, as shown in Table 4 (b), equivalent natural period become shorter compared than that of the original viaduct in transverse direction, although the yield seismic intensity decreases compared than the original viaduct. Those are probably because the height of the columns became shorter than that of the original viaduct and the diameter of piles became larger than the original viaduct. Although the maximum displacement of the crest of the slab became larger both in longitudinal and transverse direction as shown in Table 5, all verification indices are satisfied in the condition that the section dimensions and the tension reinforcement ratio are the same as the original viaduct.

In Case 2, number of the normal columns which resists the inertial force decreased from the original viaduct and weight of the slab did not change. As a result, as shown in Table 4 (a), the yield seismic intensity decreases, and equivalent natural period become longer compared than the original viaduct in longitudinal direction. On the other hand, as shown in Table 4 (b), the yield seismic intensity decreases, and equivalent natural period become shorter compared than the original viaduct in transverse direction. Although the maximum displacement of the crest of the slab becomes larger both in longitudinal and transverse direction as shown in Table 5, all verification indices are satisfied in the condition that the section dimensions and the tension reinforcement ratio are the same as the original viaduct.

From the above examinations, it is found that realistic specifications are realized in the case where DWC columns always shear vertical load, because with few modifications of design from the original viaduct.

Table 4 – Equivalent natural period and yield seismic intensity of each model

(a) Longitudinal direction

Case No.	Yield seismic coefficient K_{hy}	Yield displacement δ_{eq} (m)	Equivalent natural period T_{eq} (s)
Case 0	0.665	0.055	0.574
Case 1	0.684	0.041	0.491
Case 2	0.532	0.053	0.628

(b) Transverse direction

Case No.	Yield seismic coefficient K_{hy}	Yield displacement δ_{eq} (m)	Equivalent natural period T_{eq} (s)
Case 0	0.710	0.147	0.911
Case 1	0.701	0.132	0.867
Case 2	0.561	0.114	0.900

Table 5 – Maximum displacement of each model from dynamic analysis (m)

Case No.	Longitudinal direction	Transverse direction
Case 0	0.111	0.161
Case 1	0.152	0.232
Case 2	0.192	0.192

5. Design for existing RC rigid frame viaducts with DWC structure [9]

In chapter 4, results of test design targeting to the new rigid frame viaduct have been shown. On the other hand, it is assumed that there are many cases of installing the DWC structures to existing RC rigid frame viaducts. Therefore, a test design for existing RC rigid frame viaducts with DWC structure was conducted.

The outline of the existing viaduct is shown in Fig. 5, which is the same as the test design for newly founded viaduct mentioned in chapter 4. In this study DWC columns were installed adjacent to the normal columns. DWC columns were portal type and assumed to be made of steel to simplify construction. Separation between normal columns and DWC columns is set to the same value as the width of the normal columns in order not to inhibit the displacement of the normal columns before they are totally broken. Sliding devices were set on the top of the DWC columns to moderate a horizontal inertial force of the slab. Verification indices are shown in Table 3 (c). Calculation of sectional forces was conducted by static linear analysis using 3D beam-spring model. The lower end of the DWC columns were connected rigidly to the vertical underground beam. The axial rigidity of normal columns after breaking was set to be 2% of the rigidity before breaking. Actions considered in the design are dead load, train load, and horizontal force of the top of the DWC columns. Here, horizontal force of the top of the DWC columns is calculated as vertical force at the top of the DWC columns multiplied by friction coefficient (set to 0.1), which corresponds to friction force when an earthquake occurred in the condition that normal columns are totally broken and DWC columns support the slab load.

Structural specifications of the DWC columns installed in existing viaducts are shown in Fig 12. Sectional dimension of the DWC columns is 400 mm in width and 400 mm in height. Thickness of the DWC columns is 22 mm. In this condition the verification of safety (failure) was satisfied. Reinforcement of existing viaduct was unnecessary. Moreover, the size of the anchor frame at the lower side of the DWC columns was small enough to install in the underground vertical beam, and the anchor bolts did not interfere with the reinforce bars of the underground vertical beam.

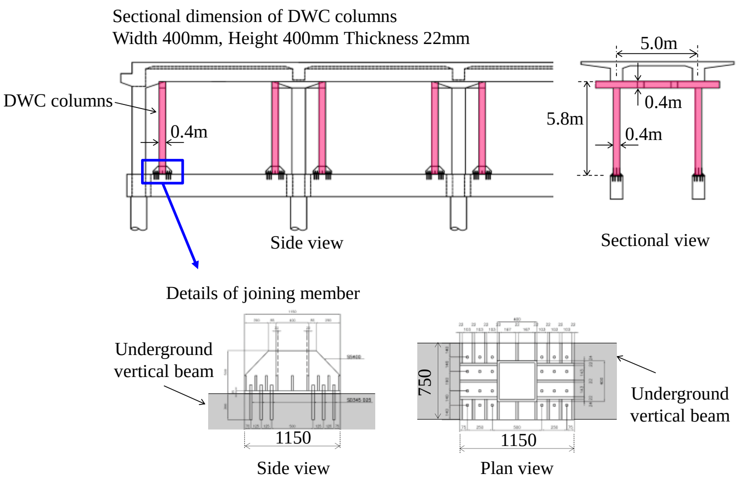


Fig. 12 – Structural specifications of the DWC columns installed in existing viaducts

6. Conclusions

In this study, large-scale shaking table tests targeting RC rigid frame viaducts with DWC structure were carried out. Then test designs for newly founded RC rigid frame viaducts with DWC structure were conducted, aiming at practical application. Finally, a test design for existing RC rigid frame viaducts with DWC structure was conducted. From those results, the feasibility of DWC structure was discussed. The conclusions of this study are as follows.

1) The results of the large-scale shaking table tests showed that the DWC columns successfully undergone the vertical weight of slab and prevented the collapse under repetitive extreme motions. It consequently followed that the proposed device be one of the promising countermeasures to realize the anti-catastrophe properties.
2) Test designs for newly founded RC rigid frame viaducts with DWC structure were conducted. In this test design, examined cases were as follows: 1) a case where DWC columns shear verticla load only after normal columns are totally broken, 2) a case where DWC columns always shear vertical load, and sliding bearings are installed between the capitals of the DWC columns and the superstructure to restrict horizontal reacting force between the DWC column and the slab to the small extent. The test design results showed that realistic specifications are realized in the latter case because there were few modifications of design from the original viaduct.
3) A test design for existing RC rigid frame viaducts with DWC structure was conducted. In this study DWC columns were installed adjacent to the normal columns. DWC columns were portal type and assumed to be made of steel to simplify construction. The lower end of the DWC columns were connected rigidly to the vertical underground beam. According to the test design results, test design results showed that it is possible to apply the DWC structure without reinforcing the existing superstructure or footing beams by installing steel columns adjacent to the existing columns.

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8. References

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