Special edition paper

DEVELOPMENT OF RESPONSE-CONTROLLED AND LINKED STRUCTURAL SYSTEM IN OVER-TRACK BUILDINGS

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Application of damage control system is one way to overcome limitations in designing and constructing over-track buildings. In order to actually apply this system, we proposed a response-controlled and linked structural system by connecting multiple independent buildings of different vibration characteristics with dampers or other energy-absorbing components. This system can be implemented with simple devices and make use of structural features of over-track buildings such as the absence of underground beams. It is also a flexible system that enables control of response of each component of the structure by selecting appropriate combinations; so it is effective in easing conditions that limit design and construction and in improving recoverability after disasters. This system also can control torsional response of over-track buildings that can easily cause horizontal eccentricity. I will introduce tests with a shake table for quantitative performance evaluation and confirmation of effects, as well as application to actual buildings.

Keywords: Over-track building, Seismic design, Vibration control structure, Damping, Damper, Test with shake table

1 Introduction

Over-track space is attracting attention as the need for better use of space increases in urban areas. But over-track buildings have many limitations in terms of designing and construction that prevent development in such space; so, there is demand for a new construction system to overcome such limitations. Meanwhile, damage control systems employing vibration isolation and vibration control technologies have been recently developed. Damage control systems alleviate the loads distributed to a structure as a whole by concentrating damage on a specified part and absorbing energy there. If this system can be applied to over-track buildings, we can design more streamlined building frames by easing the load on the parts near tracks and other parts with severe limitations. That will reduce not only materials cost but also construction cost. Also from the perspective of ensuring train operation, concentrating damage on a specified part will lead to higher recoverability after a disaster. But base isolation or other large-scale methods increase construction burden; hence we need a system attainable by devices that are as simple as possible.

In this context, I will introduce our tests with a shake table for quantitative performance evaluation and confirming the effect of the response-controlled and linked structural system we suggested as a solution of those issues in this paper. I will also cover application of that to actual buildings.

2 Overview of Response-controlled and Linked Structural System

As shown in Fig. 1, multiple independent buildings of different vibration characteristics are connected by energy absorbers (dampers etc.) in this system. Dampers activate by relative displacement or relative velocity to absorb energy, and that decreases seismic response of the buildings themselves. Since this system only requires damping components to be set at joints and bearing components that are necessary when vertical loads are transmitted at joints, it can be built with simple devices. This system also can change the response of individual components of the structure by changing the combination of vibration characteristics of structural components and materials connected. Since the limitations on construction differ by component of the over-track building, this flexible system that can control the ratio of external force is effective in alleviating the limitation and in improving recoverability by limiting the parts that are damaged. Usual over-track buildings often have no underground beams for the parts across the tracks[1]; so they tend to have different structural characteristics from that of undeveloped land. That makes it easy to vary vibration characteristics; and this accordingly makes it easy to create the ideal combination of structures for the response-controlled and linked structural system. The system would be able to control torsional response by horizontal eccentricity of buildings. Column placement of over-track buildings is often horizontally eccentric, rather than even, because placement is limited for linearity of tracks; and this eccentricity might cause such torsional response.

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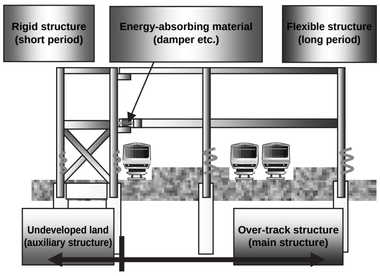


Fig.1: Overview of Response-controlled and Linked Structural System

3 Response Reduction Theory of Response-controlled and Linked Structural System

Now let us look at an example of response reduction of two independent vibration units by connecting them with a damper as shown in Fig. 2.

The optimal vibration control in this case shall be the combination that minimizes the peak rate of the transmission of earthquake vibration of both units. As shown in Fig.3, when controlling vibration by connecting two independent structures, transmission curves of mass points of both structures corresponding to each other pass a fixed point at a specific frequency, regardless the size of the damper. This phenomenon is known as the fixed point theorem. The fixed point can be obtained as the intersection point of the transmission curves of the mass points with an infinitely large damper and without a damper. The selection and design of connecting components to minimize the amplification factor at such an intersection point is the optimal design in this case[2].

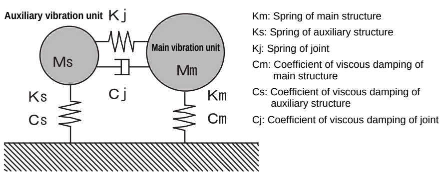


Fig.2:Basic Model of Response-controlled and Linked Structural System

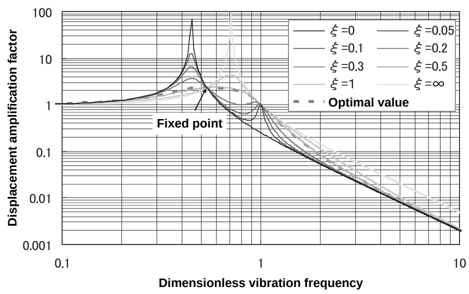


Fig.3: Concept of Optimal Setting of Transmission Rate in Response-controlled and Linked Structural System

4 Shake Table Test of Response-controlled and Linked Structural System

4.1 Overview of Test

4.1.1 Test Model

We carried out shake table tests to identify the effect of application of this system to actual buildings. Fig. 4 shows an illustration of the test model. For simplification, we comprised the model of two structures with four columns each. We specified the scale as approx. 1/2 to make the first mode natural frequency of the model similar to that of actual buildings. Table 1 shows the weight and natural frequency of the model obtained by eigenvalue analysis. We employed hydraulic dampers as connecting components and attached them at the level of the second floor between the two structures in an X-shape to allow them to work in two horizontal directions.

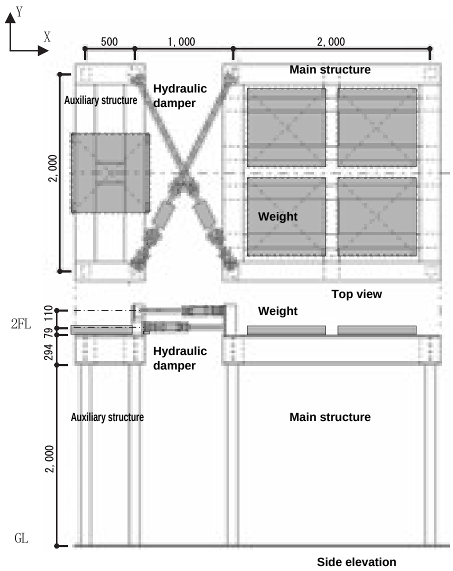


Fig.4: Plan of Test Model

Table.1: Weight and Natural Frequency Obtained by Analysis of Test Model

	Full scale model		Test model		
	Weight (ton)	Natural frequency (Hz)	Weight (ton)	Natural frequency (Hz)	
				X direction	Y direction
Main structure	830	1. 60	8. 57	3. 15	3. 15
Auxiliary structure	121	3. 70	1. 27	8. 18	7. 88
Auxiliary /Main structure	0. 146	2. 31	0. 148	2. 60	2. 60

4.1.2 Excitation Case

We carried out tests to identify the vibration characteristics of the model (measurement of microtremors, square wave excitation, random wave excitation and sine wave excitation) and seismic response tests.

We selected the seismic wave shown in Table 2 as the incident seismic wave for the tests. We set 1/2 of actual time for the time axis and standardized it at the level before the test model frame reaches the yield point.

Table.2: Incident Seismic Wave

Name	Component	Name	Maximum acceleration (+)	Maximum acceleration (-)	Time (sec)
El-Centro	NS	ELNS	341.7	−263.1	0.02
	EW	ELEW	210.1	−178.6	0.02
Taft	NS	TFNS	152.7	−137.7	0.02
	EW	TFEW	175.9	−146.4	0.02
Hachinohe	NS	HANS	225.0	−146.76	0.01
	EW	HAEW	182.9	−177.04	0.01
Tho-30	NS	THNS	258.2	−206.3	0.02
	EW	THEW	202.5	−196.1	0.02

4.1.3 Hydraulic Damper

Fig. 5 shows the results of the preliminary response analysis with the coefficient of viscous damping of the hydraulic damper as parameter. As the coefficient of viscous damping became larger, the natural frequency of the main structure shifted to a higher frequency, and that of the auxiliary structure conversely shifted to lower frequency. The shift of the auxiliary structure of smaller weight was more remarkable. Additionally, the larger the coefficient of viscous damping was, the smaller the amplification ratio was. Those tendencies were relatively evident at the coefficient of viscous damping less than 1,000 N/(cm/s), but no significant difference was seen at the coefficient of viscous damping larger than that. Based on those results, we set the coefficient of viscous damping of the hydraulic damper at 980 N (cm/s).

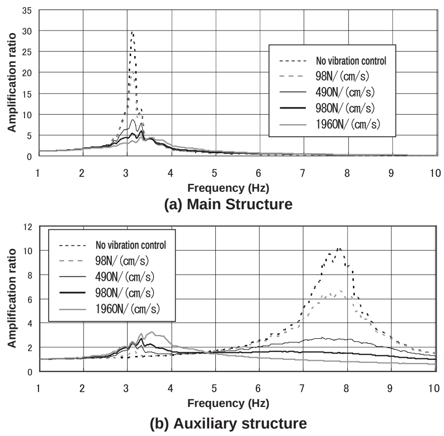


Fig.5: Amplification Factor by Preliminary Analysis

4.2 Test Results

4.2.1 Vibration Characteristics of Test Model

Table 3 shows assumed values obtained by preliminary response analysis of the natural frequency and the damping factor of the model. Analyzed values were higher than measured values in all results. This would be because the evaluation of fixation and rigid zone of the column-beam joint differed from the actual status. There was a new peak between the natural frequencies of main and auxiliary structures with dampers since the structures affected each other. The results of square wave excitation proved that the damping factor with dampers was higher.

Table.3: Natural Frequency and Damping Factor of Test Model by Excitation Test

			Natural frequency (Hz)		Damping factor (%)	
			Microtremor	Random wave	Square wave	Impact test
No vibration control	Main structure	X direction	3. 05	3. 00	2. 1	0. 10
		Y direction	3. 00	2. 98	1. 2	0. 10
	Auxiliary structure	X direction	—	7. 58	2. 6	0. 13
		Y direction	—	7. 10	0. 7	0. 14
Vibration control	Main structure	X direction	6. 51	6. 42	4. 8	—
		Y direction	6. 51	3. 06	4. 7	—
	Auxiliary structure	X direction	6. 51	6. 37	1. 8	—
		Y direction	6. 50	3. 10	2. 8	—

4.2.2 Seismic Response Test

(1) Time History Response

Due to the placement of connecting components, this model had a structure where a translational component is predominant when input in an X direction, and response of a translational component and a torsional component appears when input in a Y direction. Fig. 6 shows the time history response acceleration when input with ELNS waves in an X direction. The effect from early seismic waves was small for the main structure, but the response was clearly reduced. For the auxiliary structure, overall vibration control effect was smaller than for the main structure, while the maximum value was greatly decreased.

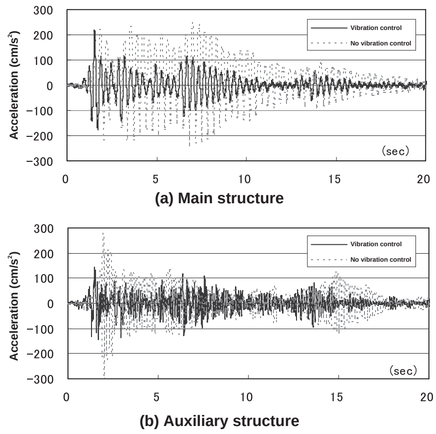


Fig.6: Time History Response Acceleration

(2) Comparison of Maximum Response

Fig. 7 shows the maximum response acceleration when excited in an X direction. When a translational component was predominant, we could find sufficient reduction effect in most cases. Fig. 8 shows the maximum response acceleration upon excitation in a Y direction including a torsional component. The response acceleration in the input direction showed no reduction when input with ELEW waves and with HANS waves.

When input with ELNS waves, vibration was amplified for the main structure, while vibration was much reduced for the auxiliary structure. The causes could be the characteristics of the incident seismic wave, the effect of higher mode components and the effect of torsion. For the purpose of assuming torsional response, we added the response acceleration in the direction orthogonal to input direction (X direction) in the figure. The response acceleration in the orthogonal direction was considerable, proving the significant effect of torsion. This tendency was more remarkable for the auxiliary structure.

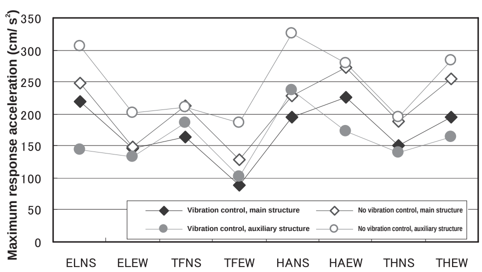


Fig.7: Maximum Response Acceleration (Excitation in X Direction)

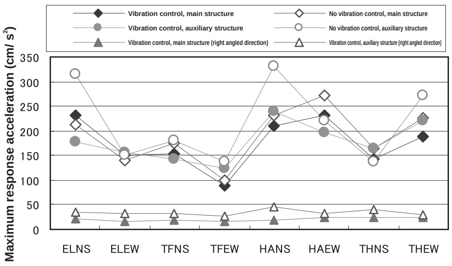


Fig.8: Maximum Response Acceleration (Excitation in Y Direction)

(3) Coefficient of Viscous Damping of Damper

Fig. 9 shows the damping force-velocity relationship of the dampers when input with ELNS waves in an X direction. Linear characteristics were not shown in the lower velocity area, probably because of the friction at the rod of the damper. The coefficient of viscous damping and the velocity were almost proportional in the higher velocity area, but overall coefficient of viscous damping was slightly smaller than the design coefficient of viscous damping (C = 980 N (cm/s)).

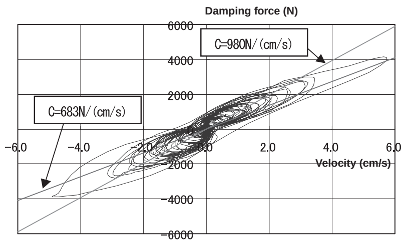


Fig.9: Damping Force-Velocity Relationship of Damper

(4) Comparison with Analysis Results

We simulated the test results using a three-dimensional frame modeling the test. Fig. 10 shows the comparison with the maximum response when excited with ELNS waves in an X direction. The analysis results agreed with the test results relatively well also for the stress at the component level. For the auxiliary structure, only component stress was amplified, while acceleration etc. was reduced. This means that acceleration response and component response are not always correlative to each other.

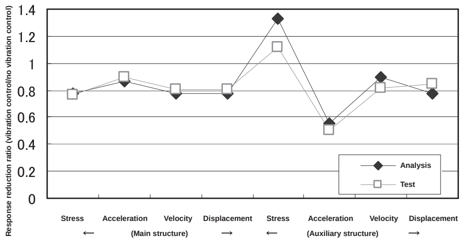


Fig.10: Comparison of Analysis Results and Test Results

5 Application to Over-Track Station Building

5.1 Overview of Building

We applied the response-controlled and linked structural system to the over-track station building of Sekiya station (opened December 2006) on the Echigo line, Niigata Pref. Fig. 11 shows the plan and Fig. 12 shows the appearance. The over-railway passage has three spans right-angled to the tracks and underground beams in the undeveloped land. The over-track station building has a span parallel and a span right-angled to the tracks and no underground beams. The vibration characteristics are combined with rigid structure for the over-railway passage and flexible structure for the station building. The passage and the station building are almost equal in weight.

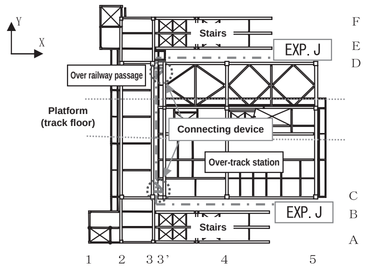


Fig.11: Plan of Concourse Floor

Fig.12: Appearance of Over-Track Station Building of Sekiya Station

5.2 Examination Policy

5.2.1 Design Criteria

As the incident seismic vibration for designing, we used a model seismic wave aiming for the acceleration response spectrum (hereinafter "notification wave") specified in a notice from the Ministry of Land, Infrastructure and Transport (No. 1461).[3)4)]

We aimed for the frame to be within the elastic region for seismic waves occurring rarely (level 1 notification wave) and that the slope by relative story displacement remain less than approx. 1/200.

We also aimed for the slope by relative story displacement of the frame to remain less than approx. 1/100 for seismic waves occurring extremely rarely (level 2 notification wave). The plasticity rate of steel components is to be less than 2.0, and pile components for the parts with a pile per column (without underground beams) are not to be over the yield proof stress.

5.2.2 Damping Device

We placed hydraulic dampers at the joint as shown in Fig. 13 to make them work for both X and Y directions. In order to prevent generation of too much reaction force, we used a relief system to set the damping force of the dampers to be constant in the velocity area greater than a specific velocity. The specifications of the dampers are shown in Table 4. The maximum response displacement of the dampers is kept to around 70% of the allowable stroke. We omitted the columns of the over-railway passage side on the railway floor of the station building. Rubber Bearings are to transmit only vertical loads to the passage side.

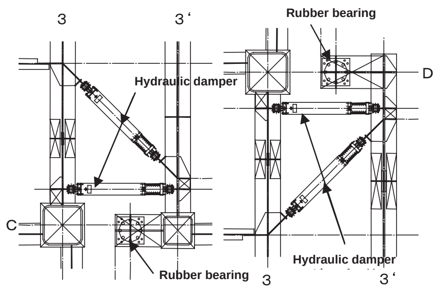


Fig.13: Details of Joint

Table.4: Setting Values of Hydraulic Damper

Damper name	Coefficient of viscous damping α (kN/kine)	Maximum damping force F (kN)	Setting angle
C x	10.0	200	90°
C y	20.0	200	45°
D x	5.0	200	90°
D y	20.0	200	45°

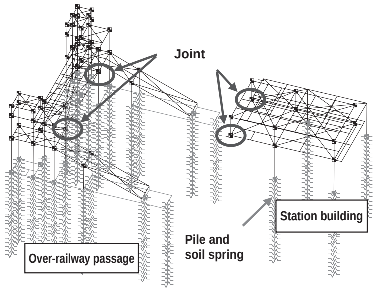


Fig.14: Analysis Model

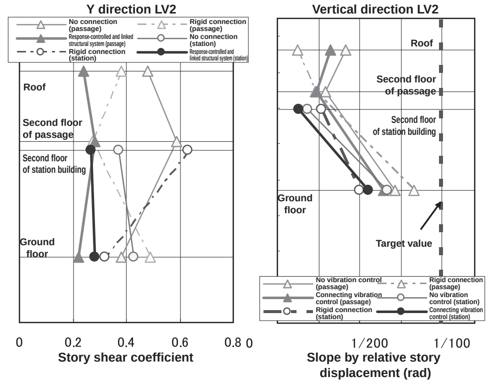
5.2.3 Analysis Policy

We used a three-dimensional frame shown in Fig. 14 for the analysis model. Since the actual building is a structure partially without underground beams in some parts, superstructures and under part of structure of the model were connected. We modeled the hydraulic dampers using Maxwell elements that directly connect springs and dashpots. In load-velocity relationship, we set bilinear resilience characteristics that have a turnoff points at the relief load.

5.3 Examination Results

The results of eigenvalue analysis showed the primary natural frequencies of the over railway passage to be 0.647 sec. and of the station building 1.105 sec. Accordingly, the ratio was approx. 1:1.8.

Fig. 15 shows the maximum response when input with level 2 waves in a Y direction. The maximum slope by relative story displacement of the passage and the station building were 1/257 and 1/168 respectively, which meet the target values. The figure also shows the values in rigid connection and in no connection. Both for the passage and the station building, story shear coefficient of response-controlled and linked structural system was smaller than that of rigid connection and no connection in all stories. Slope by relative story displacement had almost the same tendency. The maximum relative displacement between the two structures was 3.73 cm.



(a) Story shear coefficient (b) Slope by relative story displacement

Fig.15: Maximum Response

The response reduction effect by connecting was reflected at design level in the pile diameter of the station building, the reduction of the column cross-section, and the partial omission of foundation beams of the passage.

6 Conclusion

This concluded my introduction to the development background and the application examples for response-controlled and linked structural system. We are planning to apply the system to aseismatic reinforcement of existing inappropriate buildings as well as to new buildings.

I believe that the application of damage control systems including the system introduced here is indispensable as the direction to head for future architectural structures. Thus, we will further study effective systems with an aim of improving performance of over-track buildings.

Reference:

1) Railway Technical Research Institute "Structural Design Standard of Over-Tracks buildings (Low-Rise)", June 2002

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3) Ministry of Construction, Notification No. 1461, Gazette (Extra No. 106), May 30, 2000

4) Building Research Institute, Ministry of Construction, and The Building Center of Japan "Technical Guideline for Creating Input Seismic Wave for Designing (Draft): Test Commentary", 1992