DEVELOPMENT OF NEW SEISMIC RETROFIT METHOD FOR STEEL STRUCTURES USING POLY URETHANE MIXED FLY ASH

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ABSTRACT :

This paper establishes superiority of seismic retrofit method using P お ly Urethane mixed Fly Ash (PUFA). Firstly, cyclic loading test of steel pipe column filled with PUFA was performed to grasp its elasto-plastic response characteristic. The results of the steel pipe column filled with PUFA were compared with those of the steel pipe column without PUFA and those of the steel pipe column filled with concrete. Nextly, earthquake resistance of an existing steel arch bridge built in 1955 was evaluated by nonliner dynamic analysis and steel arch ribs were retrofitted by PUFA. We clarified stiffening effect comparing behaviors of the bridge retrofitted by PUFA and concrete. Through these two studies, we confirmed effect of new seismic retrofit method for steel structures using PUFA.

KEYWORDS: seismic retrofit, steel pipe column, Poly Urethane mixed Fly Ash, cyclic loading test, steel arch bridge, nonliner dynamic analysis,

1. INTRODUCTION

Studies on seismic retrofit using concrete filling for steel structures has been reported in large numbers, but we thought that lightweight material was suitable for seismic retrofit and industrial waste was utilized effectively. So we developed PUFA (Poly Urethane mixed Fly Ash) which is lightweight and high-intensity. In the past, fundamental study of PUFA was conducted. And more specifically, we conducted compressive and tension strength tests for models and formulated PUFA's material characteristics and stress-strain curves. We pushed forward our study in order to clarify superiority of seismic retrofit method using PUFA.
Firstly, in order to clarify the rationality of restoring force characteristics of steel pipe column filled with PUFA, the cyclic loading tests were conducted for three types of steel pipe columns, i.e. column filled with PUFA, column with concrete and steel column only. Then, the load displacement curve of steel pipe column filled with PUFA was obtained. Still more, the buckling phenomena was discussed.
Nextly, an existing over-through-type steel arch bridge was checked seismic safety by using elasto-plastic and geometric nonlinear dynamic analysis. Steel arch ribs failed to satisfy seismic performance was reinforced with PUFA coating. Additionally, seismic retrofit by concrete coating was conducted aside from PUFA. Then we compared analytical results of these seismic retrofits, and availability of seismic retrofit method using PUFA was assessed quantitatively.

2. CYCLIC LOADING TEST OF STEEL PIPE COLUMN FILLED WITH PUFA

2.1.Outline Of Specimen

Figure 1 shows the sketch of the specimen and Table 1 shows dimensional dates. Size of this specimen was divided outside dimension of circular steel pipe column which is designed with "Design Specifications for Highway Bridges (1996)" by homothetic ratio (α). Homothetic ratio (α) was configured a one-twelfth from loading machine's capacity and steel plate thickness in the marketplace.

2.2. Materials

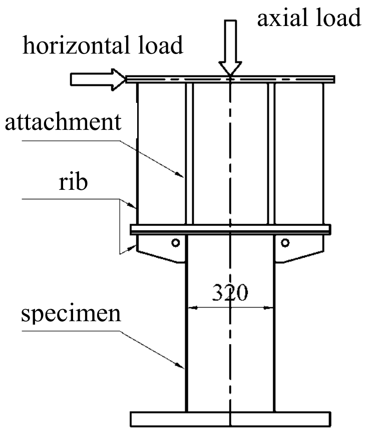


Figure 1 The sketch of the specimen

Table 1 Dimensional dates of the specimen

steel pipe column			specimen height	attachment height	total height
outside diameter D_1 (mm)	inner diameter D_2 (mm)	plate thickness t (mm)	h_1 (mm)	h_2 (mm)	(mm)
320	313.6	3.2	650	550	1200

Table 2 Test results for PUFA

test piece	density t/m^3	compressive strength N/mm^2	young modulus N/mm^2
PUFA-1	1.34	52.8	6200
PUFA-2	1.36	56.6	6500
PUFA-3	1.35	58.2	6300
average	1.35	55.9	6330

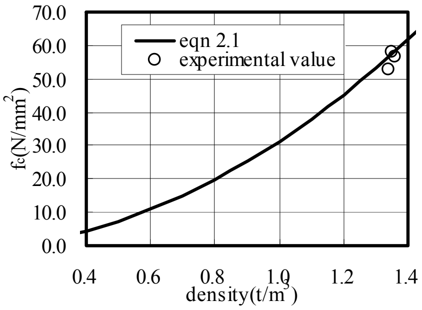


Figure 2 Compressive strength–density curve of PUFA

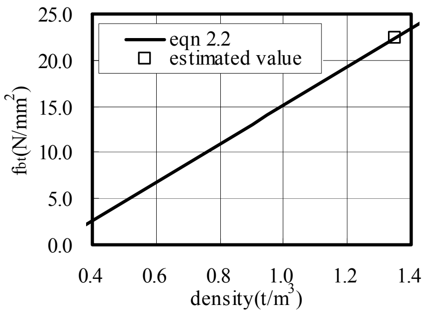


Figure 3 Tension strength–density curve of PUFA

Table 3 Test results for steel pipe

test piece	yield strength N/mm^2	tension strength N/mm^2	young modulus N/mm^2
S-1	250	441	1.84×10^5
S-2	250	441	1.97×10^5
S-3	250	440	1.90×10^5
average	250	441	1.90×10^5

Table 4 Test results for concrete

test piece	compressive strength N/mm^2	young modulus N/mm^2
C-1	35.8	3.2×10^4
C-2	36.1	2.8×10^4
C-3	36.3	3.4×10^4
average	36.1	3.1×10^4

2.2.1 PUFA

Table 2 shows density and compressive strength, elastic modulus. Eqn. 2.1 expressing the relationship between compressive strength and density was formulated by fundamental experiment an equation. Figure 2 shows compressive strength–density curve and test results, and it was found that approximate curve was extremely precise.

$$f_c = 31.5\gamma^2 + 0.644\gamma - 0.857 \tag{2.1}$$

where, f_c is compressive strength in N/mm^2 and γ is density in t/m^3.
The relationship between tension strength and density expressing by Eqn. 2.2 was formulated by fundamental experiment.

$$f_{bt} = 20.8\gamma - 5.67 \tag{2.2}$$

where, f_{bt} is tension strength in N/mm^2 and γ is density in t/m^3.
Figure 3 shows tension strength–density curve. Tension strength was expected to become 22.4 N/mm^2 from γ = 1.35 t/m^3.

2.2.2 Steel

In this study, steel pipe column was made of SS400. Table 3 shows the results of steel strength obtained from steel plate tension tests for SS400.

2.2.3 Concrete

Table 4 shows the results of concrete strength obtained by standard cylinders test. The average value of concrete strength was around 66% of an average value of PUFA strength; this difference had influence for the results of cyclic loading test.

2.3.Outline of Cyclic Loading Test

2.3.1 Loading Step

Loading path consists of two main steps. At first axial load was gradually increased to the designated value, which is 150 kN. This value is 20 % of yield strength of steel pipe column. At the second step steel pipe column was loaded by horizontal load keeping the constant axial load. The yield displacement (δ_y=2.5mm) of steel pipe column which is a control reference value on cyclic loading test was calculated with Bernoulli-Euler equation considering axial load. Column is transversely forced to displaced in $\pm1\delta_y$, $\pm2\delta_y$, $\pm3\delta_y$ … until crack appearance at steel pipe column.

2.3.2 Experiment Case

To investigate influence of with or without filling material and filling material's varieties, the cyclic loading tests were conducted for three types of steel pipe columns, i.e. column filled with PUFA (case1), steel column only (case2) and column with concrete (case3).

2.4.Results And Discussions

2.4.1 Case1-Steel Pipe Column Filled with PUFA

Figure 4 shows the experimental load-displacement relationships of case1. On the graph, the vertical axis represents the ratio between horizontal load and yield load (P_y=40.0kN) of steel pipe column only and horizontal axis represents the ratio between horizontal displacement of the top edge of specimen and yield displacement (δ_y=2.5mm) of steel pipe column only. Moreover, Figure 5 shows the progress of localized buckling at the column base. On the graph, the vertical axis represents the ratio between buckling amplitude (C) and radius of steep pipe (R=160mm) and horizontal axis is the same as that of Figure 4. Figures 4 and 5 give the following discussion.

1. Horizontal load monotonically increases until PUFA inside the steel pipe reaches flexural tension failure after yielding of the steel. Where the yield point of steel is the point which strain at outside surface of pipe reached yield strain of the steel ($\varepsilon=1.175\times10^{-3}$) and the flexural tension failure point of PUFA is the point which outer ledge strain reached flexural tension failure strain of PUFA (ε_{bt}=0.005).
2. After flexural tension failure of PUFA (point A), horizontal load drastically fell down by 20 %. This reason is thought of as tensile prestress applied by expansion of PUFA being released by flexural tension failure of PUFA.

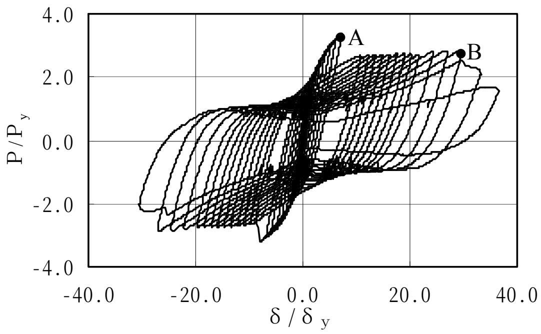


Figure 4 Hysteresis loops (case1)

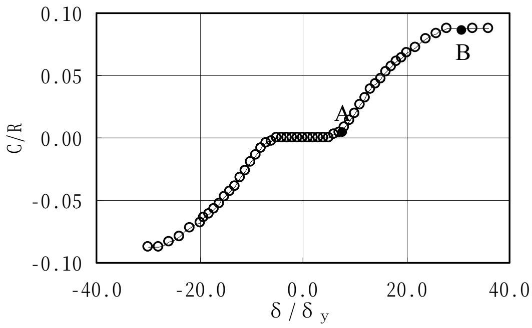


Figure 5 The progress of localized buckling (case1)

3. Localized buckling amplitude increased moderately after reaching point A, and reached point B in which steel pipe column crack while keeping horizontal load constant. The reason why steel pipe column filled with PUFA has toughness is that inside PUFA becomes triaxial compressive stress state by expansion of PUFA and circumferential tensile force is applied to steel pipe, this force curbs localized buckling.
4. Point B crack of steel pipe column was defined as ultimate limit state of steel pipe column filled with PUFA.
5. After reaching ultimate limit state, horizontal load fell down. This reason is because flexural tension failure region of PUFA in steel pipe expanded and cracks in steel pipe column widened.

2.4.2 Case2-Steel Pipe Column Only

Figure 6 shows the experimental load-displacement relationships of case2 and Figure 7 shows the progress of localized buckling at the column base. On these graph, axile parameters are the same those on case1. Figures 6 and 7 give the following discussion.

1. Horizontal load rises until maximum horizontal load (point A) after yielding of the steel.
2. On the maximum horizontal load point A, localized buckling at the column base occurred and then maximum horizontal load drastically decrease.
3. Steel pipe column was cracked at defined as Point B.

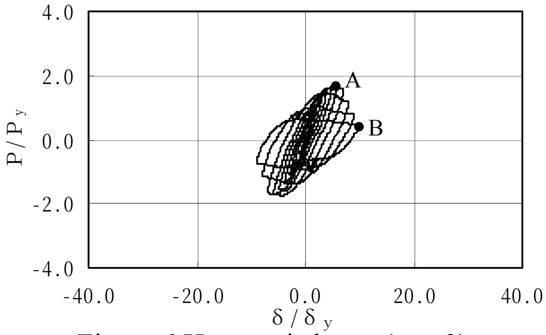


Figure 6 Hysteresis loops (case2)

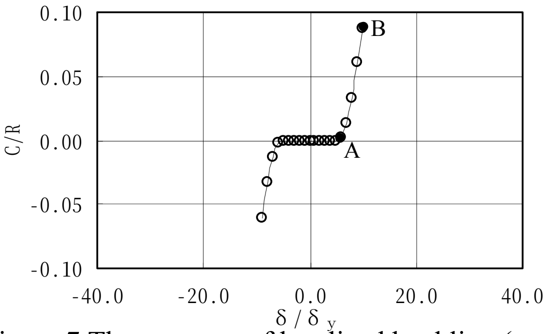


Figure 7 The progress of localized buckling (case2)

2.4.3 Case3-Steel Pipe Column Filled With Concrete

Figure 8 shows the experimental load-displacement relationships of case3 and Figure 9 shows the progress of localized buckling at the column base. On these graph, axile parameters are the same those on case1. Figures 8 and 9 give the following discussion of concrete.

1. Horizontal load smoothly rises until maximum horizontal load (point A) after crack of the concrete and yielding of the steel. Where originating point of concrete crack is the point which measured axial strain at steel pipe column base reaches flexural tension failure strain.
2. After the maximum horizontal load point A, maximum horizontal load in each loop was in a gradual decline. This reason is that localized buckling at the column base occurred. The subsequent decreasing is caused by decrease of effective cross-section area of concrete by repeating flexural tension and compression failure, so destruction property of steel pipe column filled with concrete is not exactly the same as that of steel pipe column filled with PUFA. As compared with case1-steel pipe with PUFA, progress of localized buckling at the column base was a little bit quick.

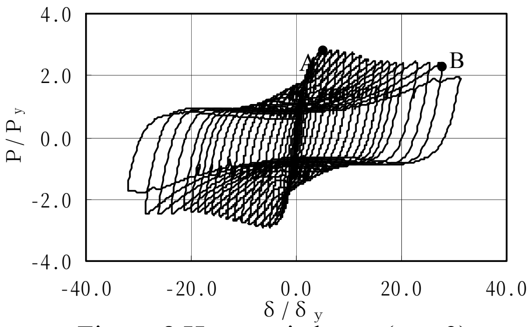


Figure 8 Hysteresis loops (case3)

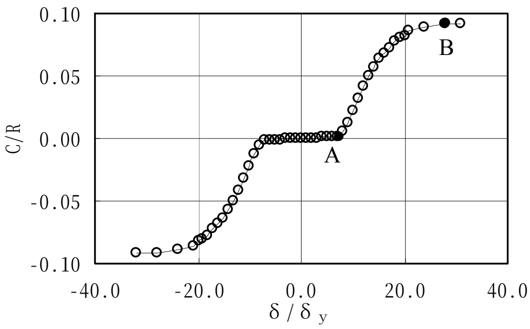


Figure 9 The progress of localized buckling (case3)

2.5. Summary

1. Load bearing ability and toughness of steel pipe column filled with PUFA is improving as compared with steel pipe column only, and stiffening effect of that is the same or more than that of steel pipe column filled with concrete.
2. PUFA can be a bit superior to with concrete in restraint efficacy on localized buckling.

3. NON-LINER DYNAMIC ANALYSIS FOR STEEL ARCH BRIDGE

3.1. Intended Bridge And Analytical Model

3.1.1 Intended Bridge

Intended bridge is a steel arched bridge constructed in 1955. Figure 10 shows the side view. Stiffening girder is supported by pin support on the left end and supported by roller support on the right end.

3.1.2 Analytical Model

Arch rib and stiffening girder, brace members were modeled as three-dimension beam elements and the other members were modeled as truss element. In addition, RC slab and stiffening girder were set as liner member and the other were set as non-liner member by checking damaged member with liner dynamic analysis. Figure 11 shows the three-dimensional frame model.

3.1.3 Stress-Strain Curve

Figure 12 shows the stress-strain curve of steel, PUFA and concrete applied to nonliner dynamic analysis. In the stress-strain curve of steel, allowable axial compressive stress was adopted in view of localized buckling at compression section, but after retrofitting, tensile yield stress was adopted considering the buckling restraining.

3.1.4 N-M-φ Curve Of Arch Rib

In nonliner dynamic analysis, N-M-φ curve of arch rib loaded by both axial force and bending moment was considered. In order to derive the N-M-φ curve, the followings were assumed.

Firstly, strain distribution is proportion to distance from the neutral axis. Nextly, member reaches yield when strain becomes yield strain ($\varepsilon_y=1.7\sigma_{ca}/E=114/(2.0\times10^5)=5.7\times10^{-4}$) at the edge in the section. Moreover, according to reference, in cases where arch rib which is principal member of steel arch bridge become severely damaged, there is a possibility that entire bridge falls down. So allowable

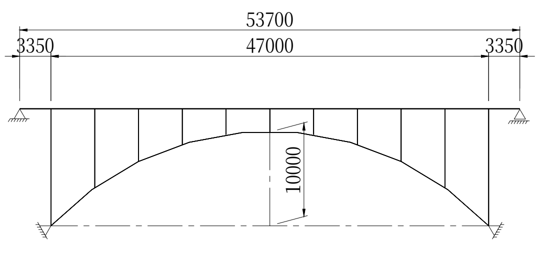


Figure 10 The side view of intended bridge

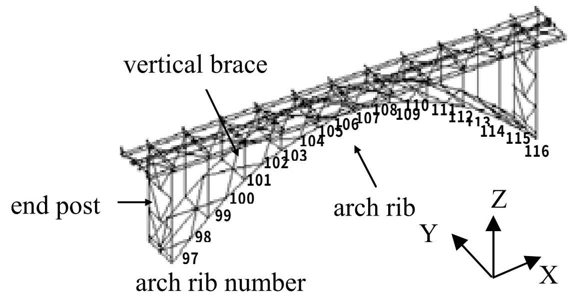
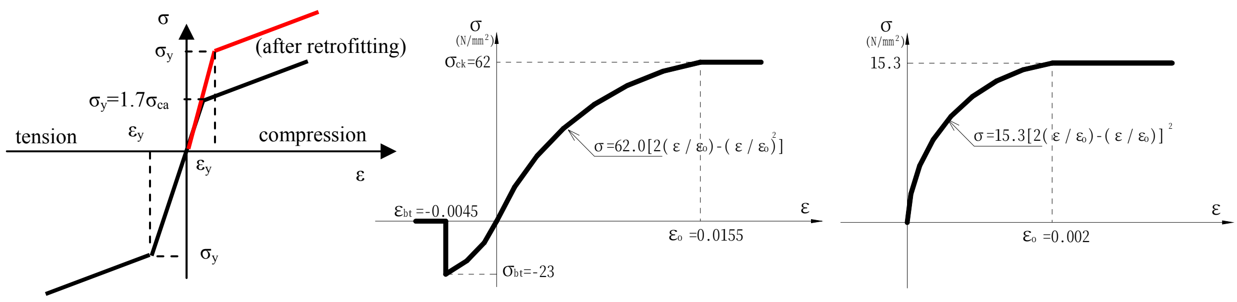


Figure 11 The three-dimensional frame model



(a) Steel (b) PUFA (c) Concrete

Figure 12 The stress- strain curve

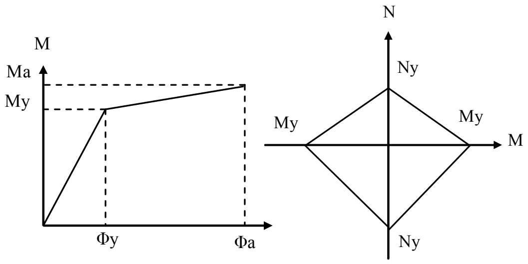


Figure 13 N-M-φ relation

Table 5 Input conditions

analytical approach	direct integration method
integral approach	Newmark- β method β=0.25
integral interval time	0.005 sec
mass matrix	lumped mass matrix
damping type	Rayleigh damping
ground classification	I type
area classification	B type
seismic wave	II - I -3

plastic strain of steel arch rib is $2\varepsilon_y$. From the above-mentioned assumption, yield bending moment of arch rib section loaded by axial force was estimated. Figure 13 shows the image of N-M-φ relation obtained as above.

3.1.5 Input Condition

In this study, we conducted dynamic time history response analysis with direct integration method. Table 5 shows input condition used on numeric analysis.

3.2. Seismic Performance Checking of Existing Bridge and Seismic Retrofit Design

3.2.1 Checking Method

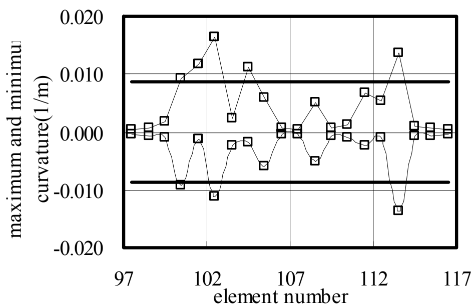
Seismic performance of existing bridge was checked by comparing the maximum response curvature of arch rib members with allowable curvature (Eqn. 3.3).

$$\phi_{\max} \leq \phi_a \cdots OK \qquad \phi_{\max} \geq \phi_a \cdots OUT \tag{3.3}$$

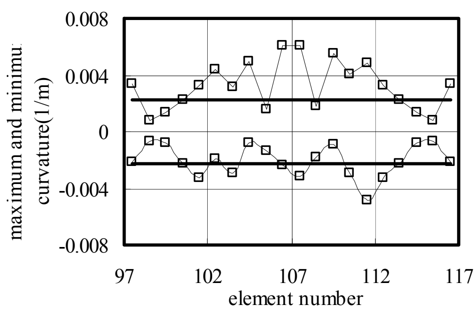
3.2.2 Checking Result

Figure 14 shows the maximum and minimum response curvature of arch rib members by nonliner dynamic analysis when seismic wave (Type2-1-3) was subjected to separately in longitudinal direction and transverse direction. On the graph, horizontal axis represents arch rib element number and the vertical axis represents maximum (tension side) and minimum (compression side) curvature. In addition, solid line means allowable curvature of arch rib. Figure 14 gives the following discussion.

1. In cases of dynamic analysis in longitudinal direction, response curvature (Max 0.0164) is larger than allowable curvature (φ_a=0.0087) at near L/4 and 3L/4 of arch rib, and these members can not satisfy the seismic performance.
2. In cases of dynamic analysis in transverse direction, response curvature (Max 0.0061) is larger than allowable curvature (φ_a=0.0023) at near base and crown of arch rib, and these members can not satisfy the seismic performance.



(a) In longitudinal direction



(b) In transverse direction

Figure 14 The maximum and minimum response curvature

3.2.3 Submissions for Seismic Retrofit Method

Results from non-liner dynamic analysis showed that arch rib members should be retrofitted. Figure 15 shows the distribution of members needed to strengthen shown by wide solid line. Arch rib members were planned to strengthen by adding PUFA or concrete as illustrated in Figure 16. In addition, it must be noted the followings in seismic retrofit design.

1. Studs are necessary inside of top and bottom flanges to promote the integration of PUFA or concrete to steel.
2. Covering (t=30mm) of PUFA or concrete seal steel from the air and shield it from corrosion.Covering is not counted on as a resisting cross-section.

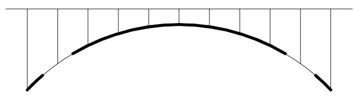


Figure 15 Steel arch ribs failed to satisfy seismic performance

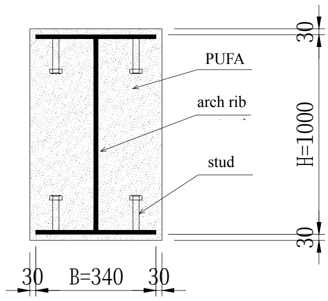


Figure 16 Reinforced section of arch rib (unit in mm)

3.3. Seismic Performance Checking of the Bridge Reinforced with PUFA Coating

The bridge reinforced with PUFA coating was analyzed similarly by nonliner dynamic analysis which was conducted to existing bridge, and seismic performance after retrofit was checked by focusing on the response curvature of arch rib members. Figure 17 shows the maximum and minimum response curvature of arch rib element when seismic wave (Type2-1-3) was subjected to separately in longitudinal direction and transverse direction. Dark and light lines mean yield curvature of arch rib with PUFA and concrete coating respectively. Figure 17 gives the following discussions.

1. In cases of dynamic analysis in longitudinal direction, response curvature (Max 0.0037) was smaller than allowable curvature (φ_a=0.0131), and all arch rib members satisfy the seismic performance.
2. In cases of dynamic analysis in transverse direction, response curvature (Max 0.0018) was smaller than allowable curvature (φ_a=0.0041), and all arch rib members satisfy the seismic performance.

3.4. Two Seismic Retrofit Methods Comparison

3.4.1 Examination Object

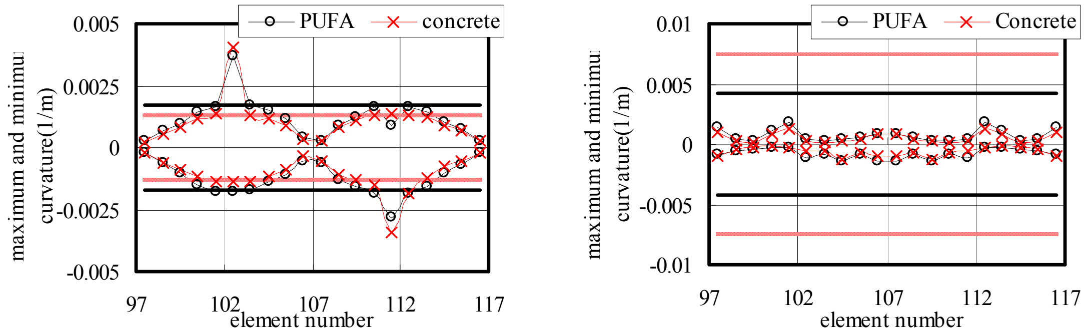
Seismic retrofit methods with PUFA coating and concrete (σ_{ck}=18N/mm^2) coating were conducted separately, and by comparing response curvature and support reaction force, availability of seismic retrofit method with PUFA coating was assessed quantitatively.

3.4.2 Response Curvature

According to Figure 17, there is little difference between response curvatures of two seismic retrofit methods. Therefore, the reductions of the response value by PUFA coating and concrete coating are almost same.

3.4.3 Support Reaction (Support Coupled On Arch Rib No.97)

Table 8 shows the maximum value of support reaction force (T_x, T_y, T_z) by non-liner dynamic analysis and Figure 18 shows T_z time history response. The directions of each support reaction force fit the definition of X, Y, Z axis of entirety coordinate system (Figure 11). Table 6 and Figure 18 give the followings.

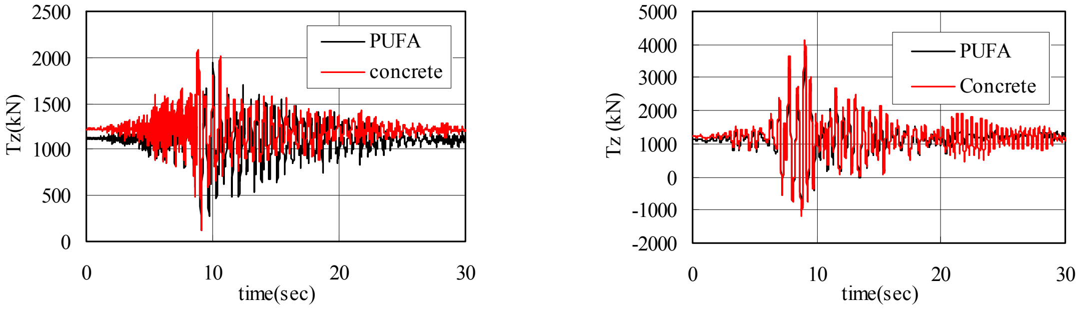


(a) In longitudinal direction (b) In transverse direction

Figure 17 The maximum and minimum response curvature of arch rib

Table 6 The maximum value of support reaction force

	longitudinal direction			transverse direction		
	T_x	T_y	Tz	T_x	T_y	Tz
PUFA (kN)	2042	43	1970	3713	1644	3790
concrete (kN)	2280	47	2080	4425	1994	4112
ratio	0.90	0.91	0.95	0.84	0.82	0.92



(a) In longitudinal direction (b) In transverse direction

Figure 18 time history responses of T_z

1. In each case, maximum support reaction force of steel arch bridge with PUFA coating is about 10% smaller than that of concrete coating.
2. In cases of dynamic analysis in longitudinal direction, there wasn't so much of a difference between steel arch bridge with PUFA coating and concrete coating and uplift didn’t occur.
3. In transverse direction, vertical movement occurred. In Addition, maximum uplift at support of steel arch bridge with PUFA coating was 1020kN and that of concrete coating was 1200kN, so the former was about 15% smaller than the latter.

3.5. Summary

1. Seismic retrofit method with PUFA coating could substantially improve seismic performance of existing bridge.
2. Steel arch bridge with PUFA coating could reduce support reaction force and uplift by up to 15% compared with that of concrete coating.

4. CONCLUSION

In this study, cyclic loading test of steel pipe column filled with PUFA and seismic retrofit design of existing steel arch bridge were conducted in order to develop new seismic retrofit method for steel structures. From a cyclic loading test, it was found that column filled with PUFA improve its strength and toughness in comparison with steel column only and column filled with concrete. Moreover, seismic retrofit method using PUFA was proved beneficial from nonliner dynamic analysis for steel arched bridge. So PUFA has applicability to seismic retrofit for steel structures.

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