STUDY ON CORROSION ANALYSIS WITH FIBER MODEL OF LONG-SPAN TRUSS BRIDGE

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Abstract

According to maintenance of long span bridges, especially inspection, it is required rational and feasible inspection programs, because these are large in scale and have numerous inspection points. To assist rationalization for long-span bridge inspection, the authors have considered features of structure and environment, and examined the maintenance plan in this study. We created the whole bridge system model of truss bridge and quantitatively identified the vulnerability to simulate in detail considered vertical load and horizontal load, those are earthquake and wind. Also we analyzed in case of member fracture. Furthermore, extracting significant inspection location considered engineering judgment was discussed.

Introduction

The Hanshin Expressway network, especially the Wangan Route located in the bay areas (FIGURE 1), has many long-span bridges. Inspection schedules for these long-span bridges are prepared individually, apart from the other bridges which are included in general inspections performed per route every 5 to 8 years (Hanshin Expressway Company Limited, 2005). Long-span bridges are extremely large in scale and composed of many members. Numerous locations need to be inspected, and inspection practices tend to be complicated. Many of them are also marine bridges with poor accessibility, which results in further increased costs of inspection and other maintenance activities. Because of these, it has been practically impossible to carry out inspections as initially intended.

FIGURE 1 HANSHIN EXPRESSWAY NETWORK

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Recently more reducing the maintenance cost, it is required rational inspection with significant level of members. The authors simulated to determine limit states of the whole bridge system and to quantitatively identify impacts of corrosion-induced sectional loss on the safety performance of the whole bridge system, with a new modeling method considering effects of local buckling (Sugioka et al., 2011a, b). To pick up some weak members with quantitative evaluation, we think very useful means, in case of settling on optimum inspection plan of frequency or criteria. In this paper, assuming member corrosion of the Gerber truss bridge, we analyzed using the whole bridge system model with local buckling or initial imperfection, where was defined as corrosion analysis, and also examined maintenance rationalization for long-span bridges because of containing complicated structure and high redundancies.

Summary of Examination

The bridge under analysis is the long-span Gerber truss bridge of 980m length with the center span of 510m and the side spans of 235m each. The general view of bridge is shown in FIGURE 2. Analysis was conducted assuming corrosion by aged deteriorated member, so weak members were extracted as significant inspected one.

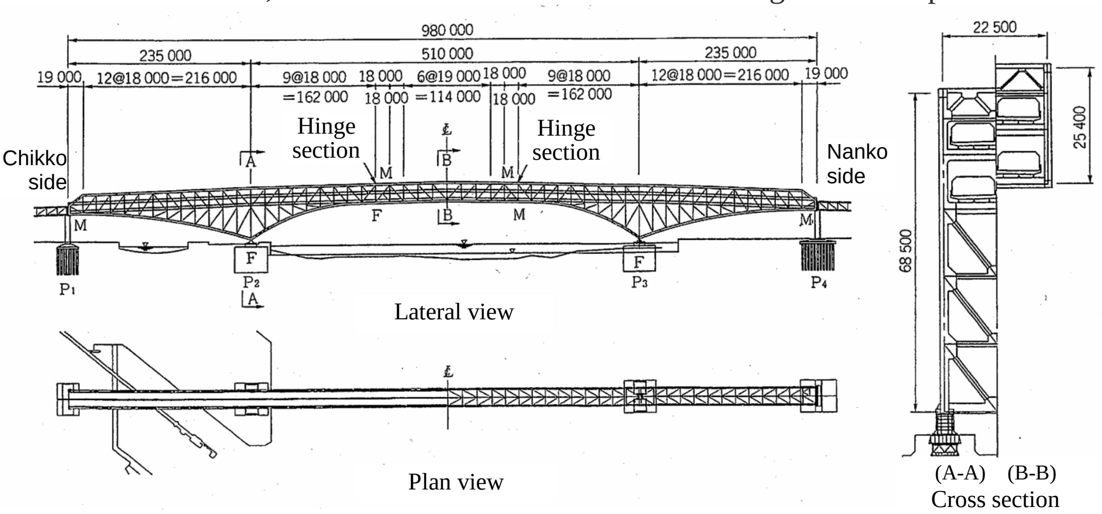

FIGURE 2 GENERAL VIEW OF LONG-SPAN GERBER TRUSS BRIDGE

The whole bridge system was modeled as shown in FIGURE 3, using the Fiber model for all members. The Fiber model that has Bernoulli-Euler theory cannot consider local buckling, so approximately evaluated axial compressive stress was equal to local buckling. The initial imperfection effecting global and local buckling stress

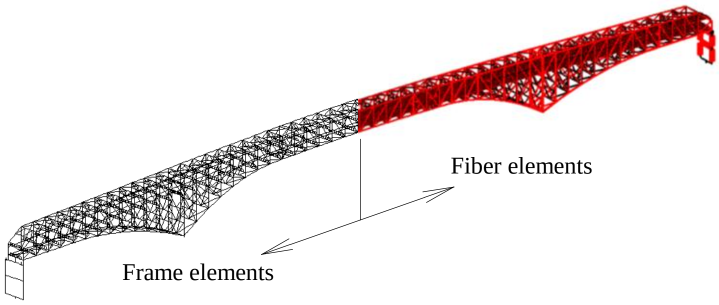

FIGURE 3 ANALYSIS MODEL OF WHOLE BRIDGE

was the initial deflection and residual stress. The initial deflection contained the deformation mode by loading D+L (for horizontal D+W, where D is dead load, L is live load and W is wind load) in advance and the deformation adjusted as L/1,000 where occurring the maximum displacement for each member. The residual stress, shown in FIGURE 4, was set up as initial stress at each integral point of fiber element. FIGURE 5 shows the contour figure of the initial imperfection. The created analysis model was proved the validation compared with the natural period, the cable tension and the support reaction of the original design.

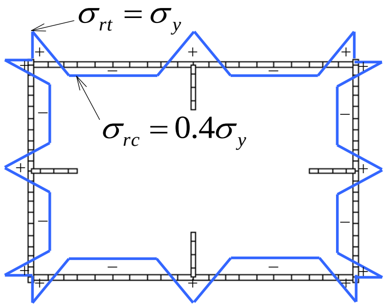

FIGURE 4 RESIDUAL STRESS

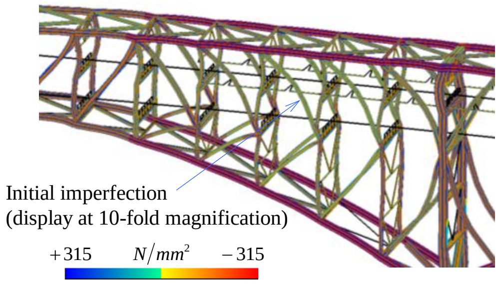

FIGURE 5 INITIAL IMPERFECTION

As corrosion was increased, the analysis stopping point emerged in structure instability. This is named the analytical limit. For steel members, the ultimate limit was judged automatically by the analytical limit through elastic-plastic finite displacement analysis. On the other hand, for the other members such as bearings did not judge the ultimate limit automatically, and adopted as the ultimate limit whether came first the analytical limit or the threshold to determine separately.

After loading dead load D, design load X was loading incrementally up to the ultimate limit of the whole bridge, which is D+βX. Design load X was applied dead load D and wind load W with loading fully, and seismic load set maximum acceleration distribution for dominant mode as load vector which simulated Level 1 earthquake response analysis in advance, as shown in FIGURE 6. In case of existing several dominant mode, seismic load was used for the central span. The bridge ultimate strength was reached at the ultimate limit on the minimum safety rate β, where magnification divided total load by design load. Also, for reference, we have calculated load ratio β' converted magnification of dead load D and design load X.

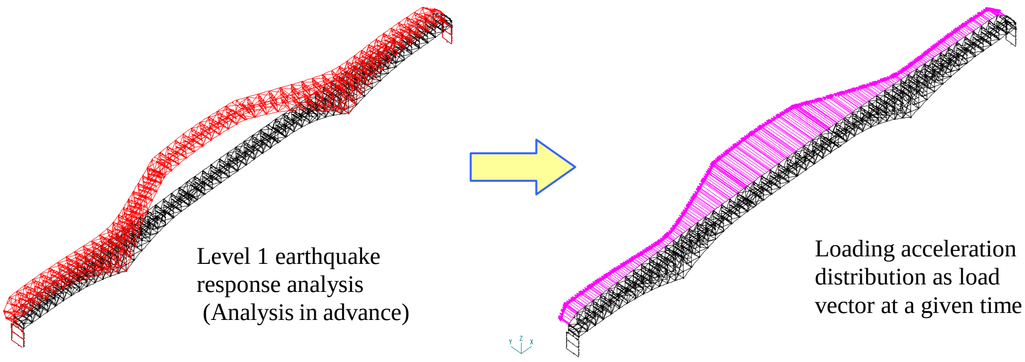

FIGURE 6 SETTING SEISMIC LOAD

Then, in the condition of loading design load that was live load, wind load and seismic load, section corrosion of the focused member increased, amount of displacement change for the focused position examined, and corrosion mass calculated when reached at the ultimate limit. The bridge under analysis is a symmetric structure, so 182 major members, quarter of whole bridge structure, were examination objects; where were 52 upper and lower chord members, 41 diagonal members, 27 vertical members, 57 cross beams, 1 eye bar support, 4 substructures. Analysis pattern was that the base was one member, without grouping corrosion members, and corroded at entire member for length and all around, as shown in FIGURE 7. Corrosion reduced to 10% through 90% of the base metal thickness in 10% increments (9 steps), it was organized the average thickness converted the base metal thickness of target members. Buckling strength was calculated for metal thickness of each corrosion step, the local buckling strength reduction was evaluated considering reduction of the yield stress. Design load was defined live load, wind load and seismic load (plane or out plane), and load vector was switched for each member of examine pattern if there were several dominant modes by dynamic response analysis of Level 1 earthquake.

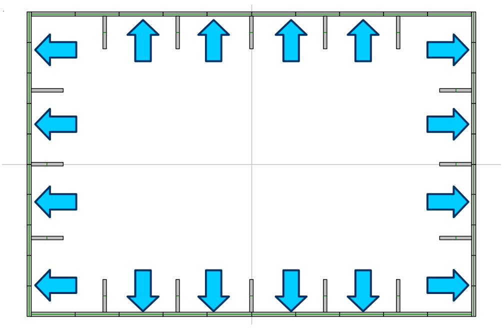

FIGURE 7 CORROSION DIRECTION

By the way, some steel truss bridges in the country and overseas has been occurred damages, therefore, either to collapse whole bridge in a moment or avoid such a disaster. These are assumed to depend on differences of damage occurring position, structural redundancy and so on. To reveal the mechanism will be able to prevent an accident. In this study, live load (D+L) was applied to the whole bridge system model and elastic-plastic finite displacement analysis was performed in case of member fracture (buckling). Analysis case was based on one member without grouping corrosion members, and as shown in FIGURE 8 (URS Corporation, 2006.), some tension members were considered fracture impact force in D+L loaded condition. Then, the member reached the analytical limit by this simulation was extracted as the significant member due to sudden fracture.

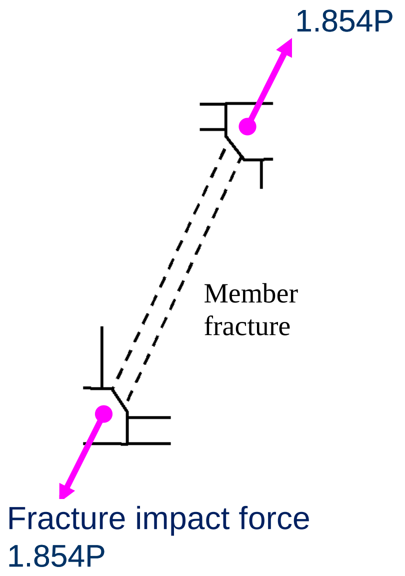

FIGURE 8 CONCEPT OF FRACTURE ANALYSIS

Analysis Results

Analysis results are shown in TABLE 1. The minimum ultimate strength is wind load (D+βW) and load ratio β is 1.57.

TABLE 1 ANALYSIS RESULTS OF ULTIMATE LIMIT

Judgment Value		Limit Value ●				Ultimate Limit
Examination Case	Monitoring Point	End Support	Intermediate Support	Pin of Eye Bar	Gerber Section	
D+L	β' [β'(D+L)]	-	-	-	-	β'=1.54 β=5.46
	β [D+βL]	-	-	-	-	
	Response/Allowable Value	0.14	0.59	0.90	0.00	
	Location	SH-P4L	SH-P2L	IB-P3L	SH-HP3U	
D+W	β' [β'(D+W)]	-	-	-	-	β'=1.08 β=1.57
	β [D+βW]	-	-	-	-	
	Response/Allowable Value	0.24	0.75	0.43	0.33	
	Location	SH-P4R	SH-P2R	IB-P3R	SH-HP2D	

FIGURE 9 shows one of analysis results in relationship between vertical displacement of the eye bar support bottom and corrosion mass of upper chord members in case of loading D+L. Here the vertical axis is the former and the horizontal axis is the latter that converted the average thickness of corrosion section. In this figure, the location named “01UB-17” was displaced drastically, the maximum vertical displacement 530mm at the eye bar support bottom, when average corrosion become 25mm. Analysis was stopped at the next step of 90% corrosion because of structural instability, so 80% corrosion was defined as the ultimate limit. FIGURE 10 shows corrosion mass of each upper chord member at the ultimate limit. Also the ultimate limits were organized for every analytical pattern of the lower chord members, the diagonal members and the vertical members; corrosion mass was made by different colors for each member as shown in FIGURE 11. Many diagonal members and vertical members were made the ultimate limit with corrosion mass of 20mm. Also the upper and lower chord

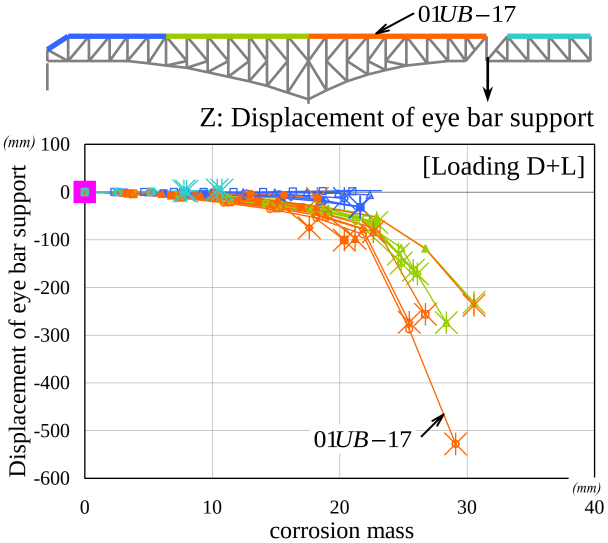

FIGURE 9 CORROSION AND DISPLACEMENT CURVE

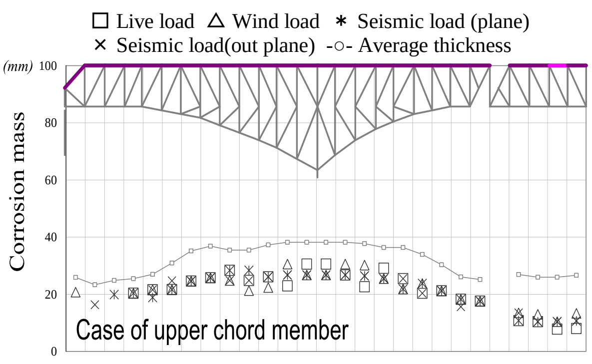

FIGURE 10 CORROSION AT ULTIMATE LIMIT

members that initial metal was thick were reached the ultimate limit with corrosion mass of 20 through 30mm. Otherwise, the eye bar support was very thick (average thickness was 75mm), so not to reach the ultimate limit but to displace much with progression of corrosion. If a less corrosion member, 0 through 10mm when its ultimate limit was reached, was regarded as weakness, some upper chord members of the suspended girder and diagonal members of the side span, displayed in red, were found weak points.

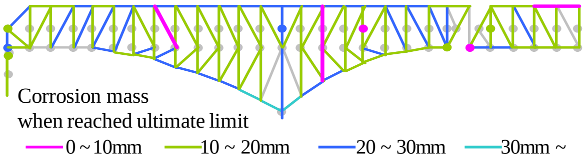

FIGURE 11 WEAK POINTS FROM ANALYSIS

Then, the results of local fracture analysis are mentioned. The fracture cases were 121 in total; where were 26 upper chord members, 26 lower chord members, 41 diagonal members, 27 vertical members and 1 eye bar support. In a condition of live load (D+L), compression members and tension members were crassified, and the latters were considered fracture impact force 1.854P. FIGURE 12 shows fracture member at each axial force. From the results of fracture analysis, members that reached its ultimate limit were extracted as significant ones, shown in FIGURE 13. In each case, there were significant members at the same degree. The results were not reached to collapse in any case.

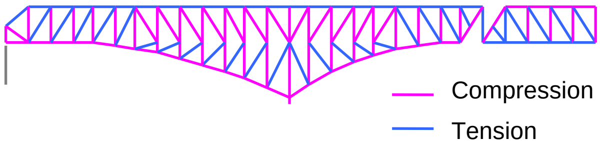

FIGURE 12 CLASSIFICATION OF MEMBERS

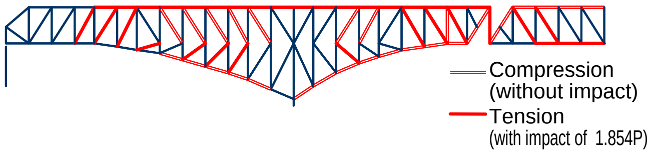

FIGURE 13 SIGNIFICANT MEMBERS

Here we examined the structure didn't make unstable even if the lower chord member near the intermediate bearing broken. The change of response axial force after fracture of the lower chord member was found that the force redistributed to the bottom laterals, the support members, the diagonal members, the main tower and the vertical members. TABLE 2 shows the increase-decrease rate of axial force share around the lower chord member before or after fracture. The lower chord member labeled No.(1) as shown in FIGURE 14 was broken, the bottom lateral No.(3) increased axial force

TABLE 2 INCREASE-DECREASE RATE OF AXIAL FORCE

			Before fracture: A	After fracture: B	Rate B/A
Lower chord member	L	①	-95,000 kN (0.27 Ny)	0 kN (0.00 Ny)	0.0
	R	②	-96,000 kN (0.27 Ny)	-161,000 kN (0.45 Ny)	1.7
Bottom lateral (side span)	L	③	-4,000 kN (0.05 Ny)	-36,000 kN (0.48 Ny)	9.0
	R	④	-4,000 kN (0.05 Ny)	-5,000 kN (0.06 Ny)	1.3
Bottom lateral (center span)	L	⑤	-5,000 kN (0.06 Ny)	-13,000 kN (0.17 Ny)	2.6
	R	⑥	-5,000 kN (0.06 Ny)	-14,000 kN (0.18 Ny)	2.8

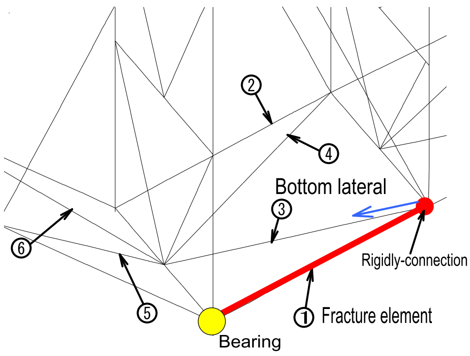

FIGURE 14 FRACTURE ELEMENT AROUND BEARING

mainly (9 times before fracture) had good margin: $0.48N_y$, where N_y was the yield axial force at full section. This was thought the force transmitted to (3) because every joint at panel point simulate as rigidly-connect. When (1) was actually broken, the gusset joint between (1) and (3) collapsed and the axial force was likely not to transmit to (3) normally. As mentioned above, this result of analysis had problem about the difference between analytical model and real structure. As it is now, however, significant part for each member was distributed by compositing structural significant member selection.

Optimum Inspection Policy

Since long-span bridges tend to have complicated structure with high redundancies, it is not necessarily reasonable to weigh all members equally in inspections. Therefore, in order to suggest priority of inspection frequency for each location, the flow diagram considered engineering judgment was made as shown in FIGURE 15 (Hanshin Expressway Company Limited, 2011). Inspection frequency was set 3 stages as “LOW”, “MIDDLE” or “HIGH”, and the location that cannot inspect with existing facilities was examined other inspection method. According to FIGURE 15, FIGURE 16 shows to suggest proper prioritization of structural members in inspection.

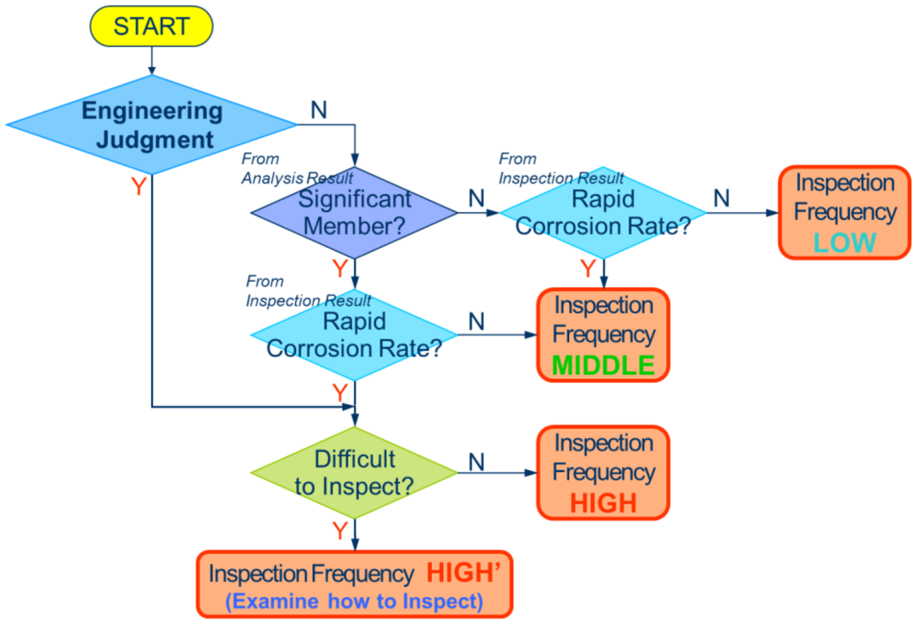

FIGURE 15 FLOW DIAGRAM OF INSPECTION FREQUENCY WEIGHT BY ENGINEERING JUDGMENT

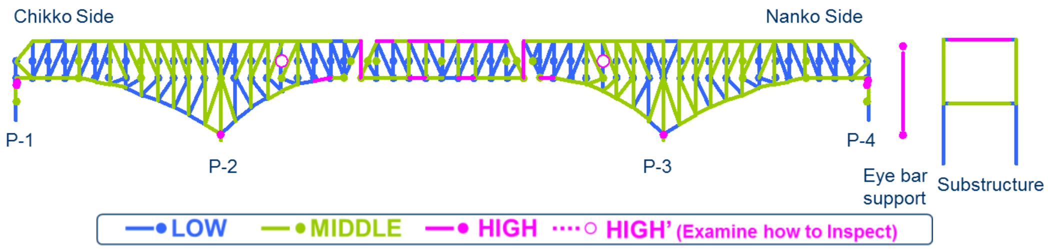

FIGURE 16 INSPECTION FREQUENCY OF LONG-SPAN TRUSS BRIDGE

The Hanshin Expressway manages, including the truss bridge under consideration, 15 long-span bridges such as the cable stayed bridge, the arch bridge and so in the Wangan Route. The inspection program for each long-span bridge was determined at the same time of initial design, so the contents of program have to be updated to reflect the condition of damages when the time comes to need a full-scale maintenance. In conjunction with the long-span bridge management techniques by Fault Tree Analysis have developed so far (Mashima et al., 2010), these techniques will be expended to rational and optimum inspection methods in the future.

Conclusion

This study was examined aging corrosion analysis with the sophisticated analytical fiber model of whole bridge system considered local buckling and initial imperfection, and presented a method to extract quantitatively weak members as significant inspection ones. Corrosion analysis was carried out for the long-span Gerber truss bridge using an whole bridge model and it was found that the some upper chord members of the suspended girder and diagonal members of the side span were the weak members. The selection of some significant inspection members from not only this analysis result but also engineering judgment enable to plan a rational inspection program of every long-span bridge.

Acknowledgments

This study has been reviewed at the Long-Span Bridge Inspection Rationalization Review Committee of the Hanshin Expressway. The authors would like to express their appreciation to Dr. Osamu Yoshikawa, a chairperson of the Committee and former professor at Osaka Institute of Technology, Dr. Masatsugu Nagai, a vice chairperson of the Committee and professor at Nagaoka University of Technology, and Dr. Kiyoyuki Kaito, a member of the Committee and associate professor at Osaka University. Appreciation of the authors is also extended to Mr. Kazuya Magoshi of Seismic Analysis Research Inc. who conducted the analysis.

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