A STUDY ON SEISMIC PERFORMANCE OF BALANCED ARCH BRIDGE USING ARTIFICIAL LIGHTWEIGHT AGGREGATE CONCRETE

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This study investigates the seismic performance of a balanced arch bridge using artificial lightweight aggregate (LWA) concrete. Dynamic analysis has been carried out for the bridge using stress-strain model, and properties of LWA concrete obtained from our previous study. In order to compare the use of LWA with normal concrete, different cases were considered. Results showed that a bridge with LWA concrete had seismic performance level 2, while the same bridge with normal concrete failed to perform even level 4. There was a 10% reduction in the number of piles when LWA concrete was used for the whole structure than when used only for arch and piers. It is also possible to reduce the size of bridge members by using the concrete.

Key words: *Balanced Arch Bridge, Lightweight Aggregate Concrete, Seismic performance, Dynamic Analysis*

1. INTRODUCTION

LWA concrete has been used in American and European highway bridges for many years. Experience in Japan where LWA concrete was applied extensively in elevated expressways, where space has been restricted and ground conditions are poor, suggests the advantages of the concrete[1)]. However, due to the oil shock in 1974 the production of LWA dramatically decreased in Japan[2)]. In 1999, there were only three companies producing a total of 48000m^3 LWA, which was just 1/3 of the peak production in 1973.

Recently, as a measure to reduce the construction cost, LWA concrete is again taken into consideration. The use of this concrete could bring several advantages such as: (1) posibility of lower inertia forces acting on a structure;(2)possibility of lower member forces acting on a foundation, therefore its dimension could be reduced;(3) the size of structure in general could also be reduced;(4) since smaller, lighter transportation and machineries can be used, shorter construction time is expected.In general, proper use of the LWA concrete could lower total construction cost. There were, however, problems encountered with its application such as: (1) high water absorption of LWA concrete mixture causing poor pumpability; (2) low elastic modulus and tension strength might cause larger deformation and lower shear capacity; (3) unknown seismic properties of members using LWA concrete.

Recent tests on real scale structures showed that the pumpability problem has been solved; OTSUKA H., et al.[3)] conducted a large scale test on a real bridge structure. The experiment showed the feasibility of pumping artificial LWA concrete mixtures up to a 70m elevation and 80m distances. Similar tests[4)] conducted on a 3-span PC frame bridge also indicated that with proper mixture of the concrete, up to150m horizontal pumping is possible.

Shear tests by Watanabe H., et al.[5] suggested that application of pre-stressed LWA concrete can improve shear capacity of LWA concrete members.

Results of new researches and developments enable application of LWA concrete to many bridges. In Japan alone, RC bridge at Inafune river, PC frame bridge at Sendai Hokubu Road, and Northeast Shinkansen Numamiyanai railway bridge have been constructed recently using LWA concrete.

In general, a large portion of masses concentrate in superstructure, therefore it is reasonable to apply LWA concrete for girders as in the cases of all bridges mentioned above. However, it is still felt that the full benefit of the material can only appreciated if LWA concrete also be applied for substructure. This is especially true for bridges in Japan where piers or arches have considerably large sizes. The application of LWA concrete for these members, which are expected to undergo large inelastic deformation during an earthquake, requires further knowledge of seismic properties of structural members using the concrete.

For this purpose the authors conducted an experimental research on "Seismic performance of structural members using artificial LWA concrete" (Interested readers may find a full version of this study, which is a master's thesis under the same title above, at Kyushu University library, or the future publication of a paper [6] in the JSCE journal). The study found that LWA concrete members have almost the same flexural strength and displacement capacity as normal concrete members. The stress-strain model for confined LWA concrete members was also proposed. Further more it has also been found that the current definition of the ultimate strain of concrete under compression as defined by the current Japan Specifications for Highway Bridges[7] (hereafter referred as the specification) is still applicable for LWA concrete.

Based on results of the previous study, this research aims to investigate seismic performance of the balanced arch bridge using artificial LWA concrete. The main objectives of this study are:1) to find out overall seismic performance of bridge using LWA concrete and compare with application of normal weight concrete and 2) to confirm technical benefits of application of LWA concrete.

It may seem obvious that lower weights of LWA concrete lead to lower inertial forces acting on a structure, however, this expected result can only be thoroughly verified by analysis. The fact is that LWA concrete has not only lower weights, but also lower modulus of elasticity. These two factors may change the natural properties of a bridge depend on distribution of masses and stiffness, and therefore the level of inertial force. Moreover, lower weights will lead to lower axial forces of members, and consequently reduce flexural capacity . Shear and displacement responses are also of interest because LWA concrete shows its weaknesses for these two design criteria. Therefore, dynamic analysis conducted in this study is necessary for complete understanding of seismic performance of the bridge using LWA concrete.

2. STUDIED BRIDGE

2.1 Bridge outline

Figure 1 shows the elevation view of the bridge under consideration. The symmetrical balanced arch bridge consists of 3 spans of PC continuous girder, 105-210-105m, total length of 420m. Girder ends rest on sliding bearings of abutments A1 and A2. Eight vertical elements on each side of axis of symmetry, namely C1 to C8 are equally spaced at 14m distance, transferring vertical loads from the girder to the arch rib. The two identical piers P1 and P2 are divided into 2 parts, upper pier and lower pier.

A typical cross section of girder is shown in **Figure 2**. The girder has a 3.114m height at the arch crown section , its highest section is 6.7m. Arch ribs have typical height of 2.8m, the height gradually increases up to 3.5m when reaching the springing section. **Figure 3** shows bars arrangement of the arch springing section. In the longitudinal direction, the top and bottom flanges have 1 inner and 1 outer rows of D38 ctc125 bars; D22 ctc125 rebars are used for lateral reinforcement. **Figure 4** shows the cross section of the lower pier

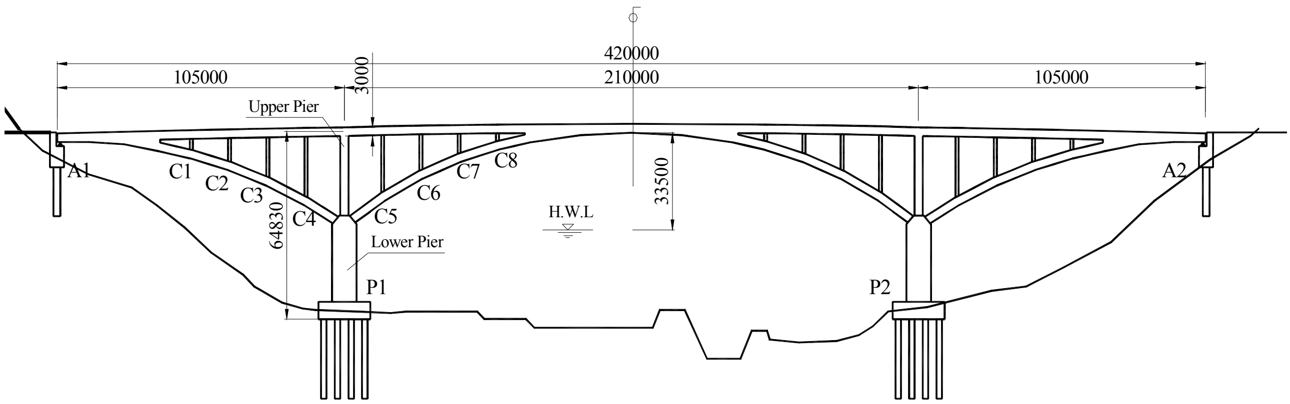


Figure 1. Elevation view of the bridge (unit mm)

with D51ctc150 rebars used as main reinforcement. The lower pier is hollow but, the upper pier has a solid 3x8.5m cross section.

2.2 Materials

For comparison purposes, two types of concrete were used, namely, artificial LWA (coarse aggregate only, produced by Taiheyo Materials Corporation with commercial name Asanolite) and normal weight concrete. The former has density of 1900kg/m^3 which is about 75% of the latter. Full description of LWA concrete properties including the designed mix can be found in reference (6. **Table 1** shows modulus of elasticity of various concrete strengths applied to different structural members of the bridge and the type of rebars and PC bars used for reinforcement.

2.3 Analytical model

The bridge has been independently analyzed in two directions. the frame elements were used for structural members in both models, namely longitudinal and transverse models. **Figure 5** shows the longitudinal model of the bridge. In this direction, vertical displacement of abutments is not significant, therefore girder end supports were considered as rollers, whereas in the transverse direction, abutments were fully taken into account. The girder was designed so that its behavior will be linear elastic. Arch rib, vertical elements, and piers are supposed to enter inelastic range and therefore modeled as nonlinear elements. Interaction of piers and foundation was modeled using spring elements. Connections of vertical elements with

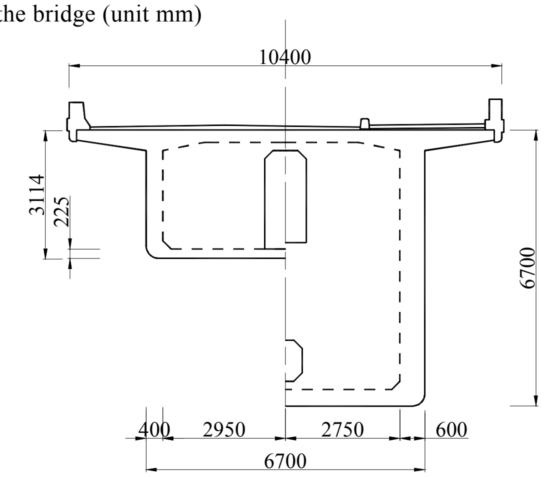


Figure 2. Cross section of girder (unit mm)

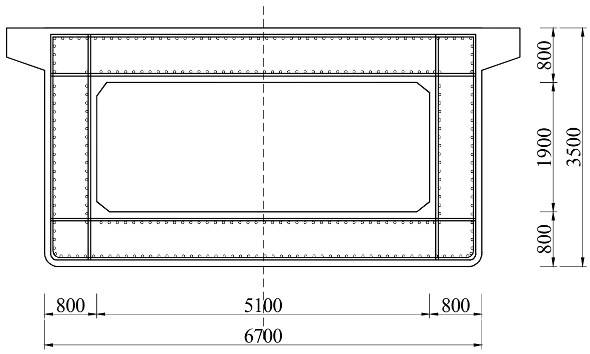


Figure 3. Reinforcement of arch springing (unit mm)

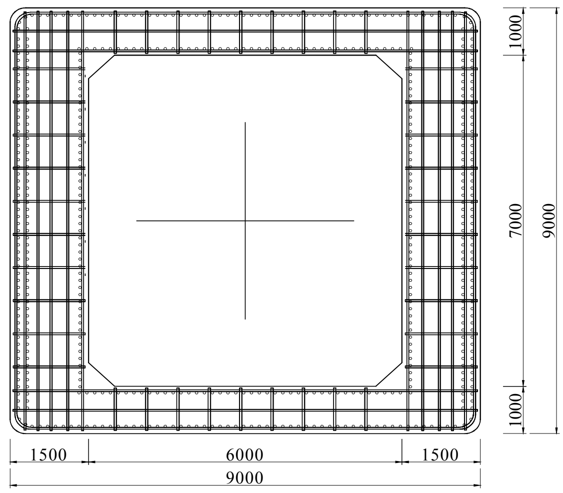


Figure 4. Reinforcement of lower pier (unit mm)

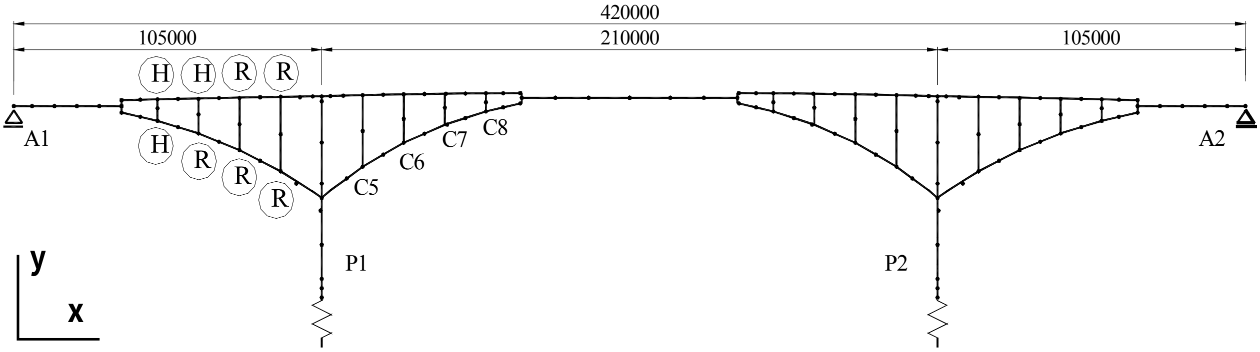


Figure 5. Analytical model (longitudinal direction) (unit mm)

Table 1. Material data

Material	Strength	Young's Modulus (kN/mm^2)		Structural members
	(N/mm^2)	Normal	LWA	
Concrete	24	25	-	Footing
	30	28	17.2	(Upper, lower) Piers
	40	31	18.8	Arch rib, posts
	50	33	20.3	Girder
Rebars	SD345			All
PC bars	SBPR930/1180			Girder

girder and arch ribs are shown in **Figure 5**, where H and R denote for hinge and rigid connections.

2.4 Analytical conditions

According to the specification[7)], the bridge under consideration has the following design characteristics:

- Class of importance : Class B
- Regional class : Class B
- Ground Type for seismic design : Type II

Level 2 (L2) earthquake was considered and the standard ground motion of type II-II-1 was used as the input acceleration wave for the time domain dynamic analysis. The analysis has been conducted using Newmark direct integration method with β =0.25, time interval Δ t=0.001. Actual time length of the wave is 40 seconds, however analysis has been conducted for 50 seconds to obtain residual displacements.

2.5 Analytical cases

Basically, 4 cases have been analyzed. As shown in **Table 2** each case has different use of concretes for structural members. LW1 and LW2 denote for cases where LWA concrete is applied to all structural members but not to the footing. LW&N denotes a case where normal

Table 2. The use of concrete in analytical cases

Case Name	Concrete type	
	Normal	LWA
LW1, LW2	footing	girder, ach rib, posts, piers
LW&N	footing, girder	ach rib, posts, piers
N	all	-

concrete is used on the girder and footing, whereas other members use LWA concrete. In the case N, normal concrete is used on all structural members. LW1, LW&N and N are 3 cases of the same bridge geometry, while LW2 is the case with reduced sizes of cross sections of the arch rib and lower pier.

2.6 Concrete stress-strain models

Material nonlinearity of members is presented using the moment-curvature model. For normal concrete, the members' skeleton curves were obtained using concrete stress-strain model available in the specification[7)]. For LWA concrete, however, a modified stress-strain model proposed by the authors based on experimental study[6)] was used since it is not yet available in the specification. This model use the same functions for ascending and descending branches as in the specification, but has the following modified equations for maximum stress, strain at maximum stress, and gradient of descending branch:

$$f_{cc} = 0.945 \; f_{c0} + 0.43 \; \rho_s \sigma_{sy} \qquad (1)$$

$$\varepsilon_{cc} = 6.229x10^{-5} f_{c0} + \frac{0.00453\rho_s\sigma_{sy}}{f_{c0}} \qquad (2)$$

$$E_{des} = 15839 - 694034 \frac{\rho_s\sigma_{sy}}{f_{c0}^2} \qquad (3a)$$

$$E_{des} = 8022 - 558175 \frac{\rho_s\sigma_{sy}}{f_{c0}} \qquad (3b)$$

where,

f_{cc} : Maximum stress (N/mm^2)

f_{c0} : Concrete strength (N/mm^2)

ε_{cc}: Strain at maximum stress

ρ_s : the volumetric ratio(%).

σ_{sy} : yield strength of lateral ties (N/mm^2).

E_{des}: Gradient of descending branch (N/mm^2)

Note that the model can only be applied for rectangular sections, and equations (3a) and (3b) represent descending gradients for concrete with strengths 40N/mm^2 and 30N/mm^2, respectively.

Figure 6 shows skeleton curves of the arch springing sections in different analytical cases. From experimental study[6], it was found that the LWA concrete member has almost the same flexural strength as normal concrete. However, it can be seen from **Figure 6** that the larger the weights the higher the flexural capacity. This is because of higher axial forces acting on the arch springing sections. In this case, self-weight analysis gave axial force values of 87.8, 77.7, 62.2MN for N, LW&N and LW cases, respectively.

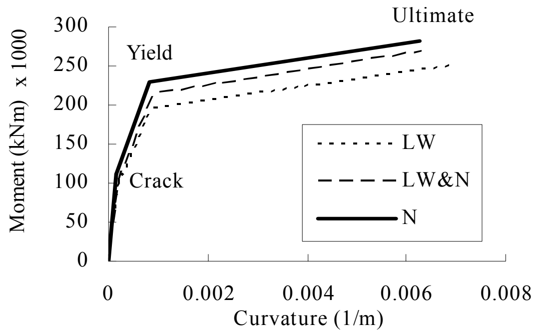


Figure 6. Arch springing skeleton curve

3. EIGENVALUE ANALYSIS

Eigenvalue analysis has been conducted considering 40 modes of vibration. **Tables 3a** and **3b** show the results of the analysis for the first 10 modes of longitudinal and transverse models, respectively. The fundamental periods of the bridge did not change much for all of the three analytical cases (LW1, LW&N and N) although bridge masses are different. The bridge in the N and LW&N cases has larger masses compared to the LW1 case, however the natural period is not longer as one might expect since in the LW1 case the girder's stiffness also reduces due to a lower modulus of elasticity of LWA concrete.

The results above suggest that the use of LWA concrete reduces inertial forces acting on the bridge structure. Since the bridge in all three cases had close fundamental periods, approximately the same level of response acceleration is expected. Therefore, the bridge with lower masses will be acted upon by lower inertial forces.

Figure 7 shows the shape of the first and third modes of vibration of the bridge in the longitudinal direction. These are two dominant modes of vibration of the bridge. Notable displaced forms of the girder suggest that its stiffness considerably affects the total stiffness of the bridge.

4. DYNAMIC ANALYSIS RESULTS

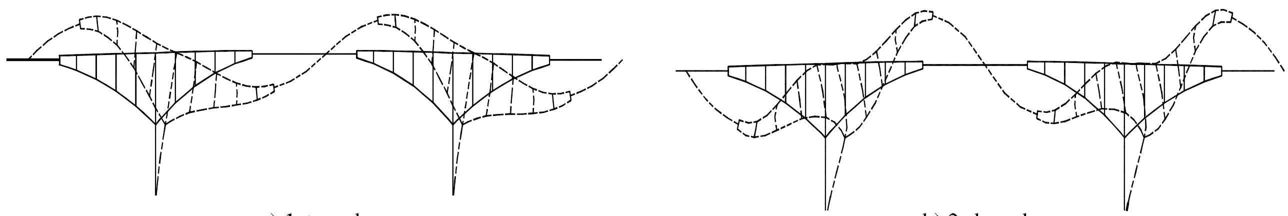
Table 3a. Eigenvalue analysis results (longitudinal)

Mode	LW1		LW&N		N	
	Period (s)	βx	Period (s)	βx	Period (s)	βx
1	2.05	130.50	1.98	139.02	2.09	166.98
2	1.11	0.00	1.07	0.00	1.05	0.00
3	0.81	117.27	0.79	115.80	0.83	131.24
4	0.77	0.00	0.71	0.00	0.72	0.00
5	0.67	-2.69	0.63	3.29	0.64	1.40
6	0.63	0.00	0.58	0.00	0.60	0.00
7	0.42	0.00	0.39	0.00	0.41	0.00
8	0.42	-23.85	0.38	-23.78	0.40	-25.36
9	0.41	13.45	0.37	-8.87	0.38	18.17
10	0.39	0.00	0.35	0.00	0.37	0.00

βx, βy: Coefficient of modal participation in x, y direction

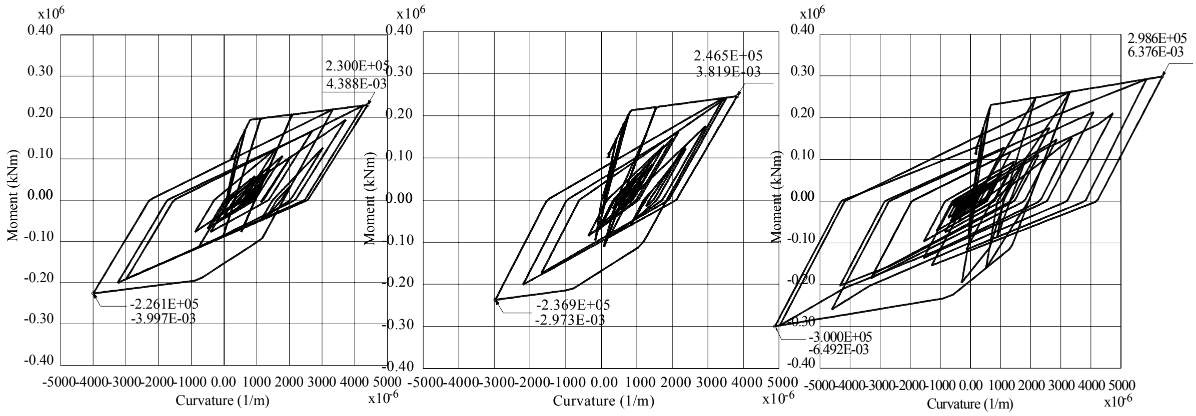
Table 3b. Eigenvalue analysis results (transverse)

Mode	LW1		LW&N		N	
	Period (s)	βy	Period (s)	βy	Period (s)	βy
1	2.60	117.75	2.55	131.12	2.61	155.12
2	1.69	1.08	1.69	0.66	1.77	0.90
3	1.42	-108.49	1.31	99.96	1.41	127.10
4	1.05	-1.24	0.91	-0.81	1.00	-1.24
5	0.68	13.01	0.59	-7.23	0.64	-20.44
6	0.52	-0.09	0.44	0.08	0.50	0.25
7	0.38	30.57	0.35	60.22	0.36	71.50
8	0.36	31.12	0.35	-1.43	0.36	-19.95
9	0.35	-27.13	0.34	-9.86	0.36	-13.36
10	0.33	1.68	0.32	1.00	0.33	2.98



a) 1st mode b) 3rd mode

Figure 7. Mode shapes (longitudinal)



a. LW1 case b. LW&N case c. N case

Figure 8. Response M-φ of the arch springing section (longitudinal direction)

Dynamic analysis results showed that in the longitudinal direction, the arch rib had remarkable responses compared to other members, whereas in the transverse direction the upper pier did. **Figure 8** shows hysteresis loops of the arch springing section. The largest response of moment and curvature belong to the N case. The LW&N case had greater bending moment compared to the LW1 case but, had a lower response curvature at maximum moment. This implies that the dynamic forces acting on the bridge were larger in the LW&N case. However, in this case the flexural capacity of the arch rib was also larger, therefore the resulting curvature was smaller. Upper piers in the transverse direction generally showed the same behavior where maximum moment and curvature are largest in the N case and smallest in the LW1 case.

Table 4 shows the maximum member forces (M, N, Q) acting on the foundation for different analytical cases. It is clear that the LW1 case has the lowest values of maximum responses. They are approximately 70% of those in the N case. The LW&N case had intermediate response values that were very close to the LW1 analysis case. From these results, the following two conclusions could be drawn: 1) the use of LWA concrete could lead to lighter foundation and 2) proper combination of normal and LWA concrete could give responses as lower as those in the case of LWA concrete only.

5. SEISMIC PERFORMANCE ASSESSMENT

5.1 Shear strength assessment

Shear strengths of normal and LWA members were calculated using the equations for shear strength in the specification [7], provided that shear strength by concrete mechanism of LWA concrete is 70% of that of normal concrete . **Table 5** shows the maximum shear forces in 3 analytical cases and the shear strengths of critical sections, i.e. arch ribs and piers. For all three

Table 4. Maximum member forces acting on the foundation

LW1			LW&N			N		
M (kNm)	N (kN)	Q (kN)	M (kNm)	N (kN)	Q (kN)	M (kNm)	N (kN)	Q (kN)
1.5×10^6	2.1×10^5	1.0×10^5	1.6×10^6	2.2×10^5	1.0×10^5	2.1×10^6	2.9×10^5	1.3×10^5

Table 5. Maximum shear forces and shear strength of members

Members	Maximum shear (kN)			Shear strength (kN)	
	LW case	LW&N case	N case	ALWA concrete	Normal concrete
Arch springing	2.10E+04	2.50E+04	2.90E+04	3.58E+04	3.73E+04
Lower pier bottom	1.00E+05	1.00E+05	1.30E+05	1.28E+05	1.31E+05
Upper pier top	1.70E+04	1.70E+04	2.10E+04	3.62E+04	3.86E+04

cases, the maximum response shear forces of members are lower than their shear strengths.

5.2 Flexural strength assessment

The assessment for moment responses was done according to the performance criteria proposed by JSCE Earthquake Engineering Committee[8]). **Table 6** shows the ratios of response (ϕ) and allowable (ϕ_a) curvatures of members in three cases. As can be seen from the table, the springing section is the most critical one among the members. With safety coefficient α=1.5 (performance level 2), the response curvatures of springing in the LW1 and LW&N cases are smaller than the allowable values. However, in the N cases the curvature ratio is greater than 1, which means that the bridge failed to perform level 2. The bridge in the N case failed to perform even performance level 4 because with α=1 the curvature ratio of springing is still greater than 1.

Table 6. Curvature ratios of members ϕ/ϕ_a

Direction	Member	Performance level			
		Level 2	Level 2	Level 2	Level 4
		LW1	LW&N	N	
Longitudinal	Arch springing	0.9	0.97	1.47	1.04
	Upper Pier top	0.26	0.20	0.22	0.15
	Lower Pier bottom	0.27	0.24	0.39	0.27
Transverse	Arch springing	0.17	0.16	0.22	0.15
	Upper Pier top	0.4	0.46	0.13	0.10
	Lower Pier bottom	0.22	0.14	0.27	0.18

Table 7. Residual displacements

Direction	Location	Residual Displacement δ_r (mm)			Limit value δ_{ra} (mm)
		LW1	LW&N	N	
Longitudinal	P1 top	1.13	1.29	5.41	327
	P2 top	3.77	3.80	0.14	
Transverse	P1 top	34.07	4.91	57.55	
	P2 top	37.90	4.58	60.53	

5.3 Residual displacement assessment

Table 7 shows the residual displacements of the tops of the P1 and P2 piers and their allowable values. Residual displacements were displacements obtained at 50 second of dynamic analysis, which is 10 seconds after the earthquake ended. The limit value of the residual displacement is h/100, where h is the height of the pier, and in this case the length of the upper pier. In all cases, the residual displacements of piers were smaller than their limit values. These values were larger in the transverse direction than in the longitudinal direction, particularly in the LW1 and N cases. However, all values were smaller than the limit values. The LW&N case had the smallest values of residual displacements.

From the results above the following points were drawn: 1) target performance level 2 is satisfied in LW1 and LW&N cases, where LWA concrete was used. However, the bridge failed to perform even seismic performance level 4 in the case of normal weight concrete; 2) as much as 30% of member forces values acting on the foundation were reduced when LWA concrete was used; 3) in general, residual displacements were largest in the case of normal weight concrete.

It is also noticeable from the **Tables 5** and **6** that there is a possibility to reduce members' size in the the LW1 case since the response values are not so close to the allowable values compared to other cases.

Table 8. Members sizes in different cases

Member	LW1	LW2
Arch springing	6.7x3.5m	6.7x3.0m
Upper pier	8.5x3.0m	8.0x3.0m
Lower pier	9.0x9.0m	8.0x8.0m

6. MEMBERS SIZE REDUCTION

The members' sizes were reduced for the original bridge since there was a relatively large margin of safety for members in the LW1 case. The bridge with reduced members' sizes was analyzed in LW2 case. The differences in members' sizes of LW1 and LW2 cases were summarized as shown in **Table 8** :

The thickness of concrete wall of all sections and total area of rebars were kept unchanged. The only change in reinforcement was the lateral hoop diameter of arch springing, where D22-ctc125 bars were changed to D29-ctc125 bars for further proof of shear. Dynamic analysis results of the LW2 case showed that the bridge was satisfied with seismic performance level 2. As shown in **Table 9,** the response curvature of members, especially arch springing section in longitudinal direction, is much closer to the allowable values after size reduction but the bridge was still satisfied by the target performance level 2. **Table 10** shows members' forces acting on the foundation of the bridge in the LW2 and LW&N cases. It was found that larger moment occurred after the size reduction by comparing the forces in the LW1 case (see **Table 4**) with the LW2 case. This is because the part of the moment , which was not carried further by the springing due to the reduction of flexural strength of arch springing section at arch-pier joint was transferred to the foundation. However, there was an obvious reduction of axial and shear forces. The LW2 case had as much as 10% reduction of axial forces and 75% reduction of shear while the moment responses remained almost unchanged compared to the LW&N case. Members' forces on the foundation of the LW2 case were then taken for the design of bridge foundation. The solution in this case was then compared to that of LW&N case to assess the benefits of using the LWA concrete in the foundation.

Table 9. Curvature ratios of members ϕ/ϕ_a

Direction	Member	Performance level 2	
		LW1	LW2
Longitudinal	Arch springing	0.9	0.99
	Upper Pier top	0.26	0.37
	Lower Pier bottom	0.27	0.46
Transverse	Arch springing	0.17	0.18
	Upper Pier top	0.4	0.29
	Lower Pier bottom	0.22	0.18

Table 10. Maximum member forces acting on the foundation

LW2			LW&N		
M (kNm)	N (kN)	Q (kN)	M (kNm)	N (kN)	Q (kN)
$1.6x10^6$	$2.0x10^5$	$8.5x10^4$	$1.6x10^6$	$2.2x10^5$	$1.0x10^5$

7. COMPARISON OF FOUNDATION DESIGN SOLUTIONS

This section compares the difference in the foundation designs of the LW2 and LW&N cases, in which the bridge satisfied with seismic performance level 2. The foundation was checked for all load conditions (i.e. service load, earthquake level 1 and level 2). The soil conditions data were of real design case.

7.1 Soil conditions and foundation solution

Table 11 shows the soil conditions at the bridge construction site. The ground consists of sand and clay soils, which can be divided into 4 layers with different values of soil module E_0. The 4th soil layer was selected as the bearing layer for piles.

Cast-in-place RC piles foundation was selected as a design solution. The pile was 2m in diameter and reinforced with single layer of 28 D51 bars . The pile's length was 23.4m. **Figure 9** shows arrangement of the piles with the cap. In the LW&N case there were 18 piles under each bridge pier. As shown in **Figure 9**, the piles are numbered from 1 to 18. In the LW2 case, however, the number of piles was reduced by 2. There

Table 11. Soil conditions

No	Layer	Thickness (m)	Depth (m)	N value	γ_t (kN/m^3)	$\alpha\times E_0$(kN/m^2) Service load (α=4)	Earthq. (α=8)
—	Sand	6.00	—	5	18	68000	136000
1	Sand	4.80	4.80	5	19	68000	136000
2	Clay	4.00	8.80	50	24	1364000	2728000
3		12.50	21.30	50	24	2320000	4640000
4		2.10	23.40	50	24	3668000	7336000

were no piles at positions 6, 7, 12, and 13, instead, two piles were set at positions 19 and 20.

7.2 Modelling and analytical method

2D frame-spring model was used for the foundation analysis. Piles cap was modeled by rigid element, while piles were modeled using trilinear frame element. Soil stiffness properties were modeled using bilinear spring element and the "distributed springs" model was considered. In this model, the pile's surface friction and pile's tip resistance were modeled using different springs. The schematic of the distributed spring model is shown in **Figure 10**. Stiffnesses of springs were obtained from soil data. Member forces acting on the foundation were applied as external loads and model was analyzed using nonlinear pushover analytical method. Three load conditions were taken into consideration: service load, earthquake level 1 and earthquke level 2.

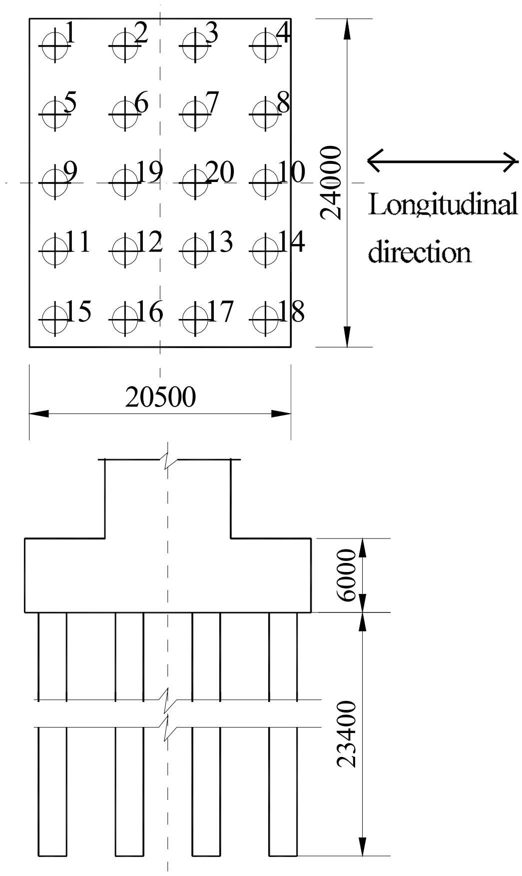


Figure 9. Piles arrangement

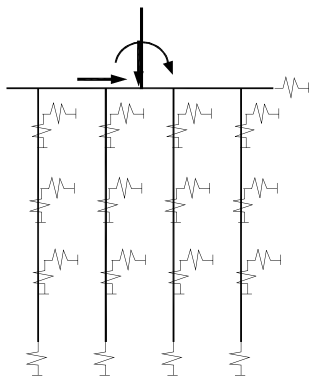


Figure 10. Distributed spring model

7.3 Analysis results

Analysis results showed that the selected design solutions for the foundation in both LW&N and LW2 cases satisfied the design requirements for all load conditions. It was also found that the design of foundation in both cases was governed by the service load case.

Table 11a and **Table 11 b** show the checked results for single pile against normal load condition and earthquake level 1 for both cases. It can be seen from these tables that for the checked items such as uplift force, horizontal displacement and pile's moment, the response values are much lower than the allowable values. However, considerable axial force responses are found. For example, the response axial force is much closer to the allowable pile vertical bearing capacity in the service load condition. This value is decisive for the design of foundation in both cases. Pile check against earthquake level 2 was also implemented for 4 items: axial force, uplift force, shear force and footing (piles cap) rotation. Response values for all checked items above were lower than their

Table 11a. Pile check for LW&N case

Items	Sub-items	Service load	Level 1 Earthqu.	
			Longitud.	Transv.
Members' forces (total)	Axial force (kN)	216245	217690	216275
	Shear (kN)	4207	29296	20076
	Moment (kNm)	118297	604528	603706
Vertical bearing	Response (kN)	14099	16099	18000
	Allowable (kN)	14846	22326	22326
Uplift	Response (kN)	7689	4299	2839
	Allowable (kN)	0	-7897	-7897
Horizontal disp.	Response (mm)	0	1	1
	Allowable (mm)	20	20	20
Moment	Response(kNm)	29	594	285
	Allowable(kNm)	4171	4171	4171

Table 11b. Pile check for LW2 case

Items	Sub-items	Service load	Level 1 Earthqu.	
			Longitud.	Transv.
Members' forces (total)	Axial force (kN)	203406	205155	194733
	Shear (kN)	4361	28603	16957
	Moment (kNm)	139622	620249	445615
Vertical bearing	Response (kN)	13950	17950	16750
	Allowable (kN)	14846	22326	22326
Uplift	Response (kN)	7550	1750	2000
	Allowable (kN)	0	-7897	-7897
Horizontal disp.	Response (mm)	0	2	1
	Allowable (mm)	20	20	20
Moment	Response(kNm)	114	2400	355
	Allowable(kNm)	4038	4038	4038

allowable values. For the reason of the paper volume these check results are not included here.

From the results of foundation analysis the following points were deduced: 1)more than 10% of number of piles could be reduced in LW2 case and 2) the design of the foundation in both LW&N and LW2 cases was governed by the service load condition.

8. CONCLUSIONS

Artificial LWA concrete with density of about 1.9t/m^3 was used for the design of concrete balanced arch bridge. To investigate the seismic performance of the bridge, dynamic analysis has been implemented. The following conclusions were drawn from this study:

1. The fundamental period of vibration of the bridge using LWA concrete was almost the same as that of normal concrete.
2. The bridge performed seimic level 2 in both LW1 and LW&N cases, where LWA concrete was used but failed to perform even level 4 in the case of normal concrete (N case).
3. It is possible to reduce members' size in the case of total use of LWA concrete.
4. As many as 10% of the number of piles could be reduced in the case of total use of LWA concrete compared to the case of partially using it for arch and piers.

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