MAINTENANCE MANAGEMENT OF MULTISPAN MASONRY ARCH BRIDGES BY RELIABILITY CONCEPT

Mitao OHGA*, Kamal KARUNANANDA*, Ranjith DISSANAYKE**, Kazuriro TANIWAKI***
Department of Civil and Environmental Engineering, Ehime University, Japan*
Department of Civil Engineering, University of Peradeniya, Sri Lanka**
Department of Civil and Environmental Engineering, Fukui University of Technology, Japan***

***ABSTRACT*: ** This paper presents a maintenance strategy of multispan masonry arch bridges by reliability concept. Based on axle load of the individual arches, a safety margin (limit state function) is introduced. Then, failure probabilities of each arch are estimated. Since failure of any arch makes the bridge failure, the bridge is treated as a series system with individual arches. Then, failure probabilities of arches are assembled to obtain failure probability of the bridge by Ditlevsen's bounds. Acceptable reliability indices of masonry arch bridges are introduced using Nordic Committee of Building Regulation. From the reliability index variation with time, major maintenance of the bridge is predicted. The introduced maintenance strategy is illustrated with an old multispan brick masonry arch bridge from Sri Lanka.

KEYWORDS: maintenance, reliability index, UIC code MEXE method

1. INTRODUCTION

Masonry arch bridges exist in most of countries in the world. At present, the increase in loading, traffic frequency and age of these structures has resulted in structural decay. Hence, safety estimation has become very important issue in masonry arch bridge management. As pointed out by some researchers, it cannot be predicted in a reliable manner because of time dependent effects, environmental effects, participation of non-structural elements and etc (Ng & Fairfield 2002). Thus, bridge authorities of most of countries still consider masonry arch bridges as difficult to rate due to many uncertainties.

Many researchers have attempted to study remaining load carrying capacity of masonry arch bridges in recent years. Marefat and his research group (Marefat et al. 2004) studied the load carrying capacity of a decommissioned plain concrete arch railway bridge in Iran by a static and a dynamic load test. Then, they concluded that all parts of the bridge such as arch, spandrel wall, fill layer, pier, and foundation contribute to the structural resistance of the bridge. Boothby and his research group (Boothby et al. 1998) carried out full scale load testing on five masonry arch bridges in the USA. Using the obtained load-deflection behavior of arch bridges, significant observations regarding masonry arch bridge behavior were concluded. Fanning and his research group (Fanning et al. 2000) presented load carrying capacities of single and multispan masonry arch bridges. In the study, they carried out service load testing and ultimate strength testing for selected bridges in the UK and the USA. They showed the importance of longitudinal and transverse effects in loading of arch bridge assessment. All of the previous researches indicate that the assessment procedures of load carrying capacity of masonry arch bridges is not straight forward and easy task as many factors contribute for load carrying capacity.

At present, there are several methods in estimation of axle load of masonry arch bridges. But none of them attempts to model uncertainties of masonry arch bridges. Hence, the objective of the paper is to model uncertainties of axle load capacity and applied axle load for introducing a maintenance management of multispan masonry arch bridges.

2. METHODOLOGY

The introduced maintenance strategy of multispan masonry arch bridges has three sections. These sections are explained in following sub sections in sequence.

2.1Estimation of failure probability of an arch

Masonry arch bridges are generally rated based on axle load capacities. Therefore, a safety margin which is based on axle load capacity is introduced for an arch as

$$M = ln(PAL) - ln(AAL) \qquad (2.1)$$

where *M* is the safety margin. *PAL* is the Provisional Axle Load and *AAL* is the Actual Axle Load of the arch. Both *PAL* and *AAL* are treated as log normal distributed random variables since negative values of *PAL* and *AAL* can be precluded. Then, the failure probability of the arch can be estimated as

$$P(F_i) = \emptyset\left[\frac{ln\left(\frac{\mu_{AAL}}{\mu_{PAL}}\times\sqrt{\frac{COV_{PAL}^2+1}{COV_{AAL}^2+1}}\right)}{\sqrt{ln\left((COV_{PAL}^2+1)\times(COV_{AAL}^2+1)\right)}}\right] \qquad (2.2)$$

where $P(F_i)$ is the failure probability of the arch, $\emptyset$ is the standard unit normal distribution, μ_{PAL} and μ_{AAL} represent the means of *PAL* and *AAL*, respectively. Similarly, COV_{PAL} and COV_{AAL} represent coefficient of variations of *PAL* and *AAL*, respectively,

2.2 Estimation of failure probability and reliability index of the bridge

In reliability modeling, each arch can be considered separately and then safety margins are introduced similar to Eq. (2.1). Then, their failure probabilities can be estimated as in Eq. (2.2). When any arch of the bridge fails, the bridge must be considered as broken. Hence, the failure modes of each arch of the bridge can be represented with a series system to obtain the reliability of the whole bridge (Frangopol 1999).

The failure probability of the bridge (P_{FS}) can be estimated from the failure probabilities of arches as expressed. In estimation of the failure probability of the bridge, Ditlevsen's bounds are used (Frangopol 1999). This bound is useful as it gives better results than simple bounds. Lower and upper bounds of the failure probability of the bridge (P_{FS}) are given as

$$P_{FS} \le \sum_{i}^{n} P(F_i) - \sum_{i=2\; j,<i}^{n} max\left[P\left(F_j \cap F_i\right)\right] \qquad (2.3)$$

$$P_{FS} \ge P(F_i) + \sum_{i=2}^{n} max\left\{\left[P(F_i) - \sum_{j=1}^{i-1} P\left(F_j \cap F_i\right)\right]; 0\right\}$$

where n is number of arches of the bridge. Then, the reliability index of the bridge (β_{FS}) can be expressed as

$$\beta_{FS} = -\emptyset^{-1}(P_{FS}) \qquad (2.4)$$

where $\emptyset^{-1}$ is the inverse standard unit normal distribution.

2.3 Maintenance management of the bridge

In order to determine the latest time to repair interventions for bridges, it is necessary to establish an acceptance level of reliability below which the bridge may be considered unsafe to operate. In most of countries, there are no criteria specified in the bridge codes and standards and no guidelines for establishing such acceptance levels.

In this study, a more direct approach for establishing acceptance probability levels based on economic optimization recommended by the Nordic committee on building regulation is used (Sarveswaran and Roberts 1999). This approach is based on the type of failure consequences and the nature of the failure as shown in Table 1. As failure consequences are strongly site specific, bridge engineers should use their own expert knowledge and judgments to find consequences of bridge failures on a selected bridge site.

Table 1 Acceptable reliability indices (Sarveswaran and Roberts 1999)

Failure consequences	Ductile failure with reserve strength	Ductile failure without reserve strength	Brittle failure
Not serious	3.09	3.71	4.26
Serious	3.71	4.26	4.75
Very serious	4.26	4.75	5.20

The reliability index of the bridge can be plotted with different time intervals. The current value of the reliability index of the bridge is compared with the acceptable reliability index. If the current reliability index is higher than the acceptable reliability index, then time required from current reliability index to acceptable reliability index is predicted. This time is defined as the time of major maintenance (essential maintenance) of the bridge.

3. ESTIMATION OF VARIABLES

Statistical parameters of variables (*PAL* and *AAL*) should be estimated to apply the introduced procedure. Hence, following sub sections explain the estimations of variables.

3.1 Provisional axle load

Several assessment methods are available to estimate the mean of *PAL* (μ_{PAL}). They are mechanism method, finite element methods, energy method, MEXE method and non-destructive testing methods. Some of the methods such as mechanism method, finite element methods and energy method are based on analytical approach whereas non-destructive methods are based on purely experimental approach. The MEXE method is a semi-empirical method (Hulet et al. 2004).

In the study, MEXE method is used to estimate the provisional axle load. The procedure was developed during World War II (1939-1945) at the military engineering experimental establishment in the UK, and it has subsequently been widely used throughout the world. The method was initially designed to provide army officers with a quick and simple means of assessing the abilities of bridges to carry out abnormal loadings.

The MEXE method was developed from a permissible stress analysis of a centrally loaded two pinned parabolic arch. Various modifying factors are applied to account for differing geometries, materials, conditions, etc. Further, the method should be used only when the fill is compacted well and it should not be used for open spandrel arch bridges. This is fast and easy to use and it should be tried before using a more sophisticated method (HA 2004) for estimation of *PAL*. There are two versions of MEXE method: Modified MEXE method and UIC code MEXE method. In this study, UIC code MEXE method (UIC 1995) is used for estimation of μ_{PAL}.

UIC code MEXE method is used more usual than other method (Modified MEXE) in estimation of axle load capacity of the bridge. The procedure of UIC code MEXE method is given below.

(i) Estimate the initial value of provisional axle load ($Initial\ PAL$) of the arch by referring relevant figures of UIC code (UIC 1995). It depends on the span, arch thickness and the height of compacted fill of the masonry arch.

(ii) Estimate modified provisional axle load ($Modified\ PAL$) of the arch as

$$Modified\ PAL = Initial\ PAL \times f \tag{3.1}$$

where *f* is the global strength adjustment factor. It is expressed as

$$f = f_S \cdot f_M \cdot f_J \cdot f_C \cdot f_N \cdot \frac{1}{f_\emptyset} \tag{3.2}$$

where f_S is arch shape factor, f_M is material factor, f_J is joint factor, f_C is condition factor, f_N is number of spans factor, and $f_\emptyset$ is dynamic factor, respectively. These modifying factors can be found from relevant tables and figures in the UIC code (UIC 1995).

(iii) Estimate mean of *PAL* (μ_{PAL}) as

$$\mu_{PAL} = Modified\ PAL \tag{3.3}$$

3.2 Actual axle load

Several methods are available to estimate statistical parameters of *AAL*. Among them, two important methods which are based on axle load measurements (weigh-in-motion measurements) and traffic survey data are briefly explained. In axle load measurement method, information such as axle load values, axle spacing and number of axles is measured through an instrument set out on the bridge wearing surface. From the measured axle load values, parameters of *AAL* can be estimated. In the traffic survey data methods, number of vehicles, type of vehicles is counted during a selected time period (TRL 2004). These measurements can then be converted to axle load values. From the converted axle load values, parameters of *AAL* can be estimated.

In this study, axle load measurement method is used to estimate the parameters of *AAL* as such data can easily be found from bridge databases. Further, axle load measurements are more reliable than traffic survey data method as there are higher uncertainties in the latter method. Thus, from axle load measurements of the bridge, statistical parameters of *AAL* are estimated.

4. CASE STUDY

Four span brick masonry arch bridge is selected from Sri Lanka to illustrate the introduced maintenance management procedure. The selected bridge (No. 90/1) is located in the route A1 that connects the capital, Colombo, to Kandy in the middle of the country. It was constructed by British military engineers in 1833 AD. Arch barrels and spandrel walls of all four spans were built of brick masonry. Piers and abutments were built of dressed Granite stones. A side view of the bridge is shown in Fig 1. The present condition of the bridge is visually satisfactory, although there are some apparent water seepages in the underneath of arch barrels. The geometric details of the bridge are given in Table 2.

Fig 1 A side view of the A1 90/1 bridge

Table 2 Geometric details of the bridge

Geometric parameter	Value
Bridge length (L_b)	70 m
Clear span of an arch (L)	15 m
Thickness of the barrel (d)	1.4 m
Height of the compacted fill from the crest of the barrel (h)	1.05 m
Rise of the arch at mid span (r_c)	4.20 m
Number of arches (n)	4

Initial value of provisional axle load of each arch is estimated by referring relevant figures in UIC code (UIC 1995) as 1000 kN. Then, six modifying factors are estimated by referring tables and figures in UIC code. Finally, the mean values of *PAL* of outer and inner arches are estimated using Eq. (3.3) as given in Table 3.

Table 3 Modifying factors and mean of *PAL* for route A1 90/1 bridge

Modifying factor	Outer arch	Inner arch
Arch shape factor	1.0	1.0
Material factor	1.0	1.0
Joint factor	1.0	1.0
Condition factor	1.0	1.0
Number of spans factor	0.9	0.8
Dynamic factor	1.25	1.25
Mean of *PAL* (kN)	720	640

A previous study (Frangopol 1999) has proposed that coefficient of variation (*COV*) of strength variables of safety margins vary from 0.0 to 0.20. In this paper, *PAL* of the introduced safety margin stands for the strength variable. Therefore, three values of *COV* of *PAL* (0.0, 0.10, and 0.20) were used.

Axle load measurements were obtained from the Traffic and Planning Division of the Road Development Authority of Sri Lanka. From that, it was estimated that *AAL* has the mean of 85.5 kN. However, *AAL* represents live loading of the bridge. According to same literature (Frangopol 1999), coefficient of variation of *AAL* is considered as 0.3.

5. RESULTS OF RELIABILITY ANALYSIS

Failure probabilities of arches are estimated from the statistical parameters of *PAL* and *AAL*. The estimated values are given in Table 4. In Table 4, outer arch is defined as an arch located either end of the bridge. Thus, inner arch is defined as an arch located between outer arches.

Table 4 Failure probabilities of each arch

Case (*COV* of *PAL*)	Outer arch	Inner arch
0.00	$6.56*10^{-14}$	$1.25*10^{-12}$
0.10	$1.32*10^{-12}$	$1.85*10^{-11}$
0.20	$5.88*10^{-10}$	$4.44*10^{-09}$

Next step is to estimate of reliability index of the bridge. However, in the process of estimation, correlation between arches is required. Hence, it is considered that there is a correlation between adjacent arches only. In fact, this correlation is due to *PAL* and *AAL* sharing among adjacent arches. Three values of correlation coefficient are used: 0.5, 0.75, and 0.90.

Hence, failure probability is estimated using Eq. (2.3). Then, reliability index of the bridge is estimated using Eq. (2.4). The obtained results are given in Table 5. Change of the reliability index with correlation coefficient is given in Fig 2 for three cases of *COV* of *PAL* and three cases of correlation coefficient of arches.

Table 5 Reliability index of bridge

COV of *PAL*	Correlation between arches	Reliability index
0.00	0.90	6.90
0.00	0.75	6.90
0.00	0.50	6.90
0.10	0.90	6.51
0.10	0.75	6.50
0.10	0.50	6.50
0.20	0.90	5.63
0.20	0.75	5.63
0.20	0.50	5.63

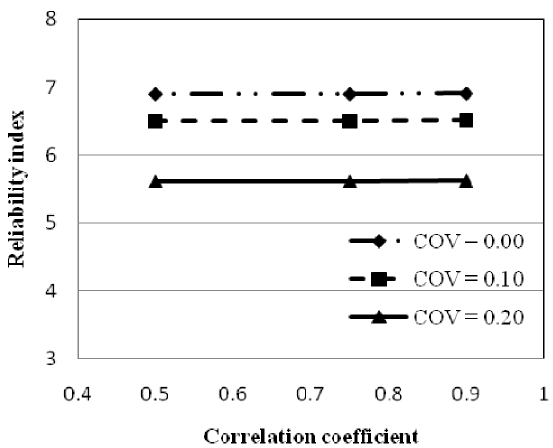

Fig 2. Effect of Correlation coefficient with reliability index

It is clear that with the increase of *COV* value, reliability index of the bridge reduces. Further, it can be concluded that the effect of correlation coefficient on reliability index is insignificant for different values of *COV*s.

With reference Table 1, the accetaptable reliability index of the bridge is estimated as 4.26 which corresponds to failure consequces of not serious and brittle failure situation.

Next step is to estimate the time required for major maintenance of the bridge. A few assumptions had to be made since there was not enough data to draw the reliability index variation curve of the bridge. Firstly, the initial reliability index (β_0) of the bridge is used as 11.0 (Christensen 1998) which corresponds to a failure probability of less than 10^{-22}. Then, a horizontal line is drawn with the constant reliability index of 11.0 ($\beta_0 = 11.0$) up to 125 years as shown in Fig 3. Thus, it is assumed that the reliability index of the bridge had been constant for 125 years after construction. The assumption is based on the facts that heavy vehicles were not started to move through the bridge until end of 1950's and strength deteriorations of masonry arch bridges are small. Then, for plotting the reliability index of the bridge on the Fig 3, the case of *COV* of *PAL* equals to 0.10 and correlation coefficient of 0.90 of Table 5 is used. Then, current reliability index of the bridge ($\beta_c = 6.51$) is plotted on Fig 3. A line was drawn from the end point of line of $\beta_0 = 11.0$ to the present reliability index. This line is extended until the reliability index equals to 4.26 (acceptable reliability index; $\beta_{acc.} = 4.26$), marked with a dotted line in Fig 3. From that, the time taken from the current reliability index to the acceptable reliability index (major maintenance time) is estimated. Hence, major maintenance time of the bridge resulted as 23 years more from the present time (Until 2032). Based on the results, it is better to be prepared for such maintenance activities in future.

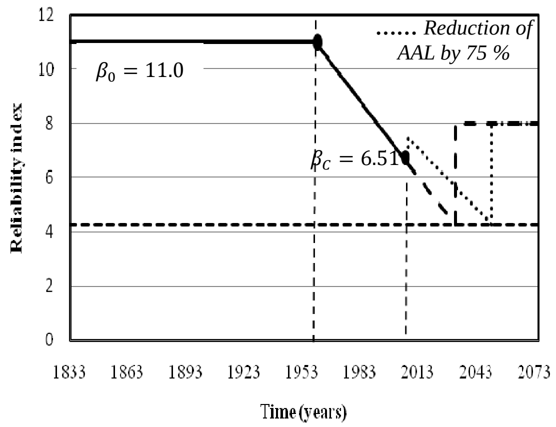

Fig 3. Reliability index variation with time

Further, the effect of reducing axle loading (*AAL*) of the bridge is estimated. For illustration as given in Fig 3, 75% of reduction of *AAL* is considered. This would increase the reliability index up to 7.45 for the same case of *COV* and coefficient of variation. Then, service life is increased further 19 years (Until 2051).

After reaching to the acceptable reliability index, major maintenance of the bridge is carried out. Then, it is possible to upgrade the reliability of the bridge as shown in Fig 3.

6. CONCLUSIONS AND RECOMMENDATIONS

Reliability based procedure was introduced for maintenance management of masonry arch bridges. Major maintenance time can be estimated from the introduced procedure. Practical applicability of the introduced procedure was illustrated with a case study.

Further study should be carried out to find coefficient of variations of *PAL* and *AAL*. Further, the correlation efficient between arches should be established. Similarly, more research studies should be directed on acceptable reliability indices of masonry arch bridges.

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