INSPECTION, ASSESSMENT, REPAIRS AND RETROFITTING OF MASONRY ARCH BRIDGES

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INDIAN RAILWAYS INSTITUTE OF CIVIL ENGINEERING
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PREFACE

The advent of RCC, a versatile material which resists both compressive and tensile forces with equal ease, is only recent. Most of the structures prior to World War II were constructed either in steel or in masonry using local materials like stones or baked mud bricks. While the steel was used for larger spans of bridges, most of the smaller openings in bridges as well as in buildings were constructed using arch structure in masonry. The arch shape resists the forces in compression transferring them through the piers abutments. Old monuments dating back to 11th century AD in stone domes and arches still survive, masonry being good material to resist compression and no tension is developed in such structures. Even in the Indian Railways, large number of small water way bridges and even some longer spans are constructed as masonry arch. They have withstood the test of time and have been serving the railways for almost 100 years. However, with the present trend of increased axle loads and braking/tractive forces, it is essential that the arch bridges are examined in detail in regard to their load carrying capacity. The behaviour of arch structures under moving loads is somewhat different than under static loads and tension is produced in masonry arches due to moving loads.

Most of the engineers would have noticed some cracks on masonry arch bridges during their inspection. The first reaction normally in such cases could be that masonry has crushed, due to age or leaching out of cement from joints etc. But, many times the reasons for such cracks may be very different. The experience however suggests that arch bridges get distressed due to development of tension in the masonry and not due to crushing in compression.

There are no exact analytical methods and calculations to check the residual strength of the bridges under railway moving loads. Railway Board had accordingly formed a core group and associated with UIC (International Union of Railways) to collect the practice being followed by other countries. Shri Ajay Goyal, Senior Professor/IRICEN was a member of this group and the information gathered by this group to assess the strength of the masonry arch bridges has been compiled in this book. The book will be useful for the railway engineers in understanding the behaviour of the masonry arch bridges under moving train loads and also to assess the strength and methods to repair and strengthen the arch bridges.

A. K. GOEL
Director/IRICEN, Pune

ACKNOWLEDGEMENT

About 15% of bridges on Indian Railways are arch bridges, 80-100 years old. Their upkeep is a challenge for civil engineers. As bridges are old, there is always some deficiency in the arch bridge, an engineer has to decide whether with existing deficiencies, bridge is in safe condition, requires repair or strengthening and if yes, what type of strengthening. Secondly, engineer has to decide whether to permit higher axle loads on the bridge. Knowledge in Indian Railways in this subject has been limited. UIC had taken up a comprehensive project on 'Improving Assessment, Optimization of Maintenance & Development of data base for Masonry Arch Bridges in 2002/03. Keeping in view large number of Masonry Arch Bridges on Indian Railways, IR also joined this project in 2004. There are total of 14 participating Railways in this project. Officers from Indian Railways attended 5 meetings from 2004 to 2007. Out of these 5, author attended three meetings. During these meetings, experts who were engaged by UIC gave presentations, which gave valuable insight. Whatever is written in the book is primarily based in UIC documents.

The main objective of this book is to summarise basics of arch bridges, best practice currently known in the field of inspection, assessment and maintenance of masonry arch railway bridges without detailed theory. The book aims at requirements of supervisors and assistant engineers doing yearly inspections in field. Complicated issues like skew bridges, 2D-3D modelling etc. has been kept out of preview of this book.

In bringing out this book, valuable technical inputs were given by my colleagues in UIC working group, Sh. Chahatey Ram, DRM/Lucknow and Sh. V.K. Govil, ED/B&S, Railway Board, for which I thank them profusely. I also thank all other members of UIC group; discussions with them have been very fruitful in understanding various issues of arch bridges. I am also thankful to Sh. V.B. Sood, Prof/Bridges/IRICEN for editing the book and Sh Ganesh

Srinivasan, personal assistant, for helping in secretarial work. I also thank Sh. A.K. Goel, Director/IRICEN, for his encouragement without which it would have not been possible to bring out this book.

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CHAPTER 1

INTRODUCTION

Most of masonry arch bridges, not only in India but world over, are more than 80-100 years old. After advent of concrete, masonry arch bridges are no more being constructed. Contrary to doubts, masonry arch bridges are proving durable with life-cycle costs significantly more economical than for the majority of other type of structure. Masonry arch bridges are part of our heritage. To preserve our heritage and to utilize these bridges to their maximum capacity, understanding of arch bridges is vital. As no more masonry arch bridges are being constructed in modern times, engineers have forgotten the old thumb rule methods of their design and also there are no common softwares available which can analyse the arch bridges correctly taking into account effect of fill, spandrel walls etc. The design of these structures was based on empirical rules which were too conservative, this has resulted in structures with an inherent ability to withstand the applied loads and extreme weathering conditions. Masonry arch bridges form an integral part of the railway infrastructure. They are the oldest structure type of the railway bridge population with thousands still in service despite their age and the significant changes in loading conditions that have occurred since their construction. Today many masonry arches carry a load that is radically different from that when they were constructed.

The condition of masonry arch bridges can vary from good to very poor, but nevertheless they have proven durability with life-cycle costs significantly lower than the majority of other structure types. Total replacement of deteriorated masonry bridges is generally unnecessary and is also expensive and therefore maintenance strategies should promote solutions that preserve and restore arch structures. In order that the railways may accommodate increased axle loads, train speeds and a greater volume of freight traffic, it is necessary to assess the load carrying capacity of existing masonry arch bridges. Assessment of masonry arch bridges is fraught with difficulties as there is little knowledge or experience of the design of arch structures to modern standards, and such structures may have parts hidden from view. The hidden parts, however, influence the structural behavior and have a major effect on the load carrying capacity of these bridges. To provide confidence in the assessment result, reliable input parameters are required. Accordingly effective inspection and measuring methods to establish the parameters are necessary. In addition to visual inspections, there has been a tendency in recent years towards the use of non-destructive testing techniques and destructive investigation to establish the necessary dimensional and material parameters.

1.1 DEMOGRAPHY OF MASONRY ARCH BRIDGES ON INDIAN RAILWAYS

Total no. of bridges on IR = 1,27,768
Arch bridges out of above = 19,647 (15.38%)

Largest span of masonry arch	27.43m (90')	Bridge No.13 on Tundla - Agra section of Agra division
Longest arch bridge	954.9m (3133') – 254x3.6m (12')	Bridge No. 208 on Howrah - Burdawan main line
Multi layer masonry arches	Arch gallery built of arches of 3.35m (11') and 3.81 m (12'6'') span in a 4 storied structure	On Kalka – Shimla line of Ambala division

15% of bridges on IR are arch bridges. Such a huge population of bridges cannot and need not be replaced purely on age basis without any detailed analytical study.

1.2 OBJECTIVES AND SCOPE OF THIS BOOK

The main objectives are to:

- provide basics of arch bridges without detailed theory

- summarize the best practice currently known in the field of inspection, assessment and maintenance of masonry arch railway bridges
- provide guidance for the routine inspection, assessment and maintenance methods bring in uniformity in inspection and maintenance procedure on Indian railways
- introduction to NDT methods available for assessment of a arch bridge

This book deliberately ignores

- Special situations like skew bridges, multi layer bridges, 3D modeling and special load testing etc. for which specialist literature may be referred to.

1.3 UIC PROJECT ON MASONRY ARCH BRIDGES

UIC had taken up a comprehensive project on 'Improving Assessment, Optimization of Maintenance & Development of data base for Masonry Arch Bridges in 2002/ 03. Keeping in view large number of Masonry Arch Bridges on Indian Railways, IR joined this project in 2004. Officers from Indian Railways attended 5 meetings from 2004 to 2007.

The principle objective of this UIC project on masonry arch bridges is to collect and develop tools that help optimizing the life-cycle management of masonry arch bridges, help reducing the maintenance costs and promote an effective exchange of good practices among various world railways.

The project is also aimed at:

- Disseminating the best practice and enhancing the propensity to turn masonry arch bridge related research into useful and commercially valuable innovations.
- Helping implement new technologies and innovations in order to reduce maintenance costs and expand the life-span of masonry arch bridges.

- Optimizing maintenance processes through improved understanding of the structural behavior, and through increased knowledge of current levels of safety and the effects of maintenance measures.
- Helping the life-cycle management of masonry bridges through effective policy making and by providing easy access to the appropriate information, literature, software and a problem-solving platform.
- Revising and extending the existing UIC Code 778-3R “Recommendations for the assessment of the load carrying capacity of existing masonry and mass concrete arch bridges”

Participating Railways - 14
Working Group members - 24

Members	**Country**	**Railway/Organisation**
Zoltan Orban (Chairman)	Hungary	MAV
Bernard Plu	France	SNCF
Bohuslav Stecinsky	Czech Rep.	CD
Fernado Martins	Portugal	REFER
Gaetano Pitisci	Italy	RFI
Dr. Antonio Brencich (Consultant)	Italy	University of Genoa
Hans-Ulrich Knaack	Germany	DB
M. Gutermann (Consultant)	Germany	DB
Ivar Ness	Norway	JBV
Keith Ross	UK	NR
Prof. W. Harvey (Cons)	UK	Obvis Ltd.
Dr. Matthew Gilbert (Consultant)	UK	Sheffield University
Manfred Mautner	Austria	OBB
M. Moser (Consultant)	Austria	OBB
Rafael Garcia Ozaeta	Spain	RENFE
Dr. Jose A. Martin-Caro (Consultant)	Spain	INES (Const)
Members	**Country**	**Railway/Organisation**
Dr. Seiichi Tottori	Japan	RTRI
Hansulrich Remensberger	Switzerland	SBB
Dr. Jan Bien	Poland	PKP
Tomasz Kaminski	Poland	PKP

P Rawa	Poland	PKP
Shri V.K.Govil	India	I R
Shri Chahatey Ram	India	I R
Shri Ajay Goyal	India	I R

Following institutions of repute were engaged/ associated in the project

University of Pecs, ORISOFT Ltd. (HUN)
Obvis Ltd., University of Sheffield (UK)
Hochshule Bremen (GER)
Brno University of Technology (CZ)
Ingenieurburo Pauser (AUT)
INES Consultores (SP)
Wroclaw University of Technology (POL)
University of Genoa (ITA)

During these meetings, experts who were engaged by UIC delivered lectures, which gave valuable insight. The project gave better understanding of behavior of arch bridges, how to inspect and assess load carrying capacity of such bridges and what methods of repairs can be adopted economically. It gave exposure to modern methods available for the above purpose in world railway systems. Revision of UIC code 778-3R, which deals with masonry arch bridges, is also one of the objectives of the project.

Another objective of UIC project was dissemination of knowledge. Effort has been made to achieve this objective through this book.

1.4 TERMINOLOGY

Abutment

A masonry mass supporting the arch barrel and provides resistance to the thrust and vertical loads from the arch, normally on other side is earth.

Admissible Load

The maximum load capacity of an arch determined using appropriate methods of assessment

Annual inspection

Visual examination carried out at approximately 12 month intervals to identify changes in damage patterns and monitor their progress.

Arch

A structure curved in a vertical plane spanning an obstruction and capable of supporting vertical loads, and transferring these loads to the abutments or piers.

Arch Barrel

The load bearing part of an arch consisting of a single thickness of voussior stones or multiple rings of brickwork spanning between abutments and/or piers.

Assessment

The determination of the safe load capacity of a structure taking into account the physical condition and location of the structure. The term includes site inspection with site measurements and the carrying out of any calculations and checks

Backfill

Low strength fill material abutting the structure providing lateral support

Backing

Masonry or concrete constructed over an arch barrel to increase dead load and load distribution. Backing is usually level with the extrados at the crown of the arch.

Bond

An arrangement of masonry components in a regular pattern so that joints are not coincident between adjacent courses of blocks.

Brick

A kiln fired block of clay usually rectangular in shape used in masonry construction.

Circular arch

An arch with an intrados of constant radius.

Crown

The centre and highest point on an arch barrel.

Damage

A loss of integrity of a masonry arch which may impair the load carrying capacity or a change in appearance of a masonry arch which results from human actions, vegetative growth or other factors.

Deep arch

An arch with a rise to span ratio equal to or greater than 0.5.

Defect

A lack of completeness or failure of a component of a masonry arch.

Durability

The quality of maintaining a satisfactory appearance and load carrying ability.

Efflorescence

Crystalline deposits on surface of masonry after evaporation of water carrying soluble salts from within the masonry.

Extrados

The outer (convex) curved surface of an arch barrel.

Fill

Material, usually low strength, placed above the extrados and retained between the spandrel walls which support the railway.

Foundation

The part of the masonry arch below ground level, that transfers and distributes the load from the abutment or piers into the surrounding ground.

Haunching

A thickening of the lower section of an arch barrel towards the springing.

Intrados

The inner (concave) curved surface of an arch barrel.

Masonry

Stone, brick or concrete block construction with joints cemented with mortar.

MEXE method

Military Engineering Experimental Establishment method for assessment of masonry arches, based on the work of Pippard.

Mortar

A mix of inorganic binders (lime or cement), sand aggregate and water forming the joint between stone or bricks in masonry, and which may be bedding mortar providing structural load transfer or pointing mortar for the outer finish of the masonry.

Parapet

Wall at the edge of an arch bridge, usually a vertical continuation of the spandrel wall constructed to prevent persons on the bridge from falling over the edge.

Pier

An intermediate support between two adjoining arch spans.

Pointing

The process of removing loose and defective mortar from joints in masonry construction and filling and finishing the joint with new mortar.

Rehabilitation

The restoration of a damaged masonry arch to a satisfactory functioning condition. Rehabilitation may include repair and strengthening.

Repair

To restore a defective part of an arch to an acceptable condition.

Ring

A layer of single masonry elements across the width of an arch to form the arch barrel. In brick arches the arch barrel is formed from multiple adjacent rings.

Rise

The vertical height from the springing point of an arch to the crown of the intrados.

Segmental arch

A circular arch with a rise to span ratio less than 0.5.

Semi-circular arch

An arch with an intrados of constant radius forming an arc of 180 degrees (rise/span ratio = 0.5).

Scour

The removal of material from around a foundation by flowing water.

Shallow arch

Arch with a rise to span ratio less than 0.25.

Skewback

The inclined masonry surface located at the extremity of an arch barrel which transmits the thrust to an abutment or pier.

Slender pier

A pier for which the interaction of adjacent spans can lead to partial or total collapse of the structure. If the ratio of the height above foundation level to the pier thickness in the direction of the span exceeds two, it is considered a slender pier.

Spalling

Flaking of material from the surface of masonry.

Span

The distance between springings measured at the intrados.

Spandrel

The space above the extrados to the arch barrel and below the track formation level, and may contain compacted fill, internal spandrel walls, voided arches, backing or haunching.

Spandrel wall

A wall built on the extrados of an arch filling the almost triangular space bounded a horizontal line at the level of the crown and a vertical line from the abutment.

Springing

The junction of the vertical face of an abutment and the arch barrel, and from where an arch is supported.

Strengthening

A method of providing a designed increase in the load capacity of a masonry arch from that determined in an assessment. This may be to restore the load capacity of the bridge, or to permit an increase of the traffic loads including increased train speeds.

Thrust

The force due to vertical applied loads exerted by and within an arch.

Ultimate load

The maximum load capacity of an arch determined using the ultimate limit state theorem.

Valut

Arch barrel is also called valut.

Viaduct

A bridge comprising multiple spans (generally 5 or more).

Voussoir

Wedged shaped blocks usually brick/stone forming an arch.

Waterproofing

A membrane provided over the extrados of the arch to prevent water from reaching the arch barrel.

Width

The transverse dimension between edges of the arch barrel perpendicular to the spandrel walls

Wing Wall

A wall adjacent to the abutment that retains backfill to the abutment and which supports the embankment.

1.5 DOCUMENTS REFFERED TO IN WRITING THE BOOK

Following documents produced by UIC working group were referred

1. Harvey, W.J. Guide to Assessment: International Union of Railways, January 2008
2. Assessment, Reliability and Maintenance of Masonry Arch Bridges (Eds. Orbán, Z., UIC Masonry Arch Bridges Working Group). State-of-the-Art Study 2003. International Union of Railways, January 2004.
3. Martin-Caro, J. A; R.O. García-Catalán, Catalogue of Damages for Masonry Arch Bridges: International Union of Railways, January 2008
4. Harvey, W.J. Review of the Military Engineering Experimental Establishment (MEXE) method: International Union of Railways, January 2008
5. Gilbert, M. Guide to use of ring2.0 for the assessment of railway masonry arches: theory & modelling, International Union of Railways, January 2008
6. Brencich, A. Guide to High Level Assessment. International Union of Railways, January 2008
7. Bien, J, Kaminski, T, Rawa, P: Technology and pilot version of expert tool supporting the evaluation of the degradation level for masonry bridges with damages: International Union of Railways, January 2008
8. Steffens, K. Gutermann, M. Institute of Experimental Static University of Applied Sciences Bremen: Guide for load tests of masonry arch bridges : International Union of Railways, January 2008
9. Martin-Caro, J. A. Guide for execution and control of repairs - cleaning manual, International Union of Railways, January 2008
10. Martin-Caro, J. A. Guide to execution and control of

repairs: Part I: Maintenance and replacement of existing masonry, International Union of Railways, January 2008

11. Brencich, A. B. Guide to execution and control of repairs: Part 2: Guide to Strengthening of masonry arch bridges, International Union of Railways, January 2008
12. Harvey, W.J. Railway Empirical Assessment Method (REAM) – Simple arch assessment, International Union of Railways, January 2008

In addition, following documents published by RDSO were also referred

1. Code of practice for the design and construction of masonry arch bridges
2. Investigation on strength of masonry arch bridges, progress report no. 5
3. Investigation on strength of masonry arch bridges, progress report no. 6
4. Minutes of various items related to arch bridges in 'Bridge and Structures standards committee' meetings

CHAPTER 2

HOW AN ARCH BRIDGE WORKS

Before going to behavior of arch bridge, knowledge of basic elements of arch is necessary, which is described below;

2.1 ELEMENTS OF A MASONRY ARCH BRIDGE

The principle components of a masonry arch are shown in Fig. 2.1.

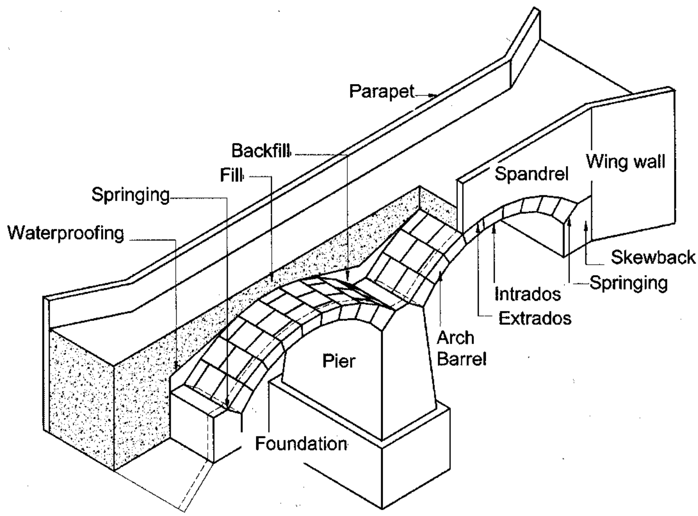


Fig. 2.1: Elements of a masonry arch

2.1.1 Arch barrel

The arch barrel is the primary element of an arch bridge. The barrel may be constructed from dressed stone or bricks. When constructed with brick, the barrel will be formed in several layers or rings which are usually bonded together and are therefore inherently less robust than those with stone voussoirs. For this reason, brick rings are usually thicker than stone for the same span and rise.

The arch barrel thickness across a width may be reduced locally under the external spandrel walls. The shape of the barrel in a railway arch is governed by the clearance requirements and economics at time of construction. Bridges on the same route were therefore usually constructed with arch barrels with identical or similar geometries.

2.1.2 Abutment and Pier

The abutment provides the vertical and horizontal resistance to the spread of the arch. Although rigidly constructed, to resist the thrust from the arch barrel, an abutment moves imperceptibly towards the supported embankments. This movement may normally be less than 0.5 mm in spans up to 8 meters.

Piers transmit vertical load and little horizontal load as horizontal loads due to dead load gets balanced. Horizontal load has little effect on the behavior of an individual span, if height of pier is short but effect may be significant in case of slender long piers

2.1.3 Spandrel Walls

Spandrel walls are provided on the edge of the arch barrel to contain the fill. Spandrel walls are generally built as a continuation of the edge voussoirs, but normally they are built on top of the arch without any form of key. Wall above the level of bottom of sleeper is termed as parapet.

2.1.4 Fill

The primary purpose of the compacted fill in the spandrel is to provide a level formation. The fill also distributes the live load over a larger area of the arch and provides a reaction to movement of the arch barrel thereby increasing the stability of the arch. The usual practice is to use local material for the fill.

2.1.5 Backing and haunching

Additional masonry with a horizontal upper surface is known as backing whilst additional masonry with a sloping surface is known as haunching. This additional masonry may be rough masonry or formally bonded, but is however rarely visible at the face of the arch barrel. This masonry provides a path for the thrust with substantial distribution before it reached the soil fill behind the abutment, and provides an increase in capacity and durability

2.1.6 Wing walls

Wing walls, either continuing the line of the spandrel walls, or turned at an angle contain the fill behind the abutment. They also increase the stability of the abutment, although less so as the angle increases. Wing walls may be founded at the base of an embankment allowing the arch to be stable before the embankment was built. If founded within the embankment the typical lack of compaction of an embankment make settlement of the structure a certainty.

2.1.7 Track system

The track system provides a substantial measure of distribution between the wheel rail contact and the base of the sleepers. The degree of distribution of the load from the wheel through sleepers and through the ballast to the supporting arch depends on the relative stiffness of the various track components.

2.2 FORCE FLOW IN ARCH AND EFFECT OF VARIOUS PARAMETERS

2.2.1 Force flow and load paths

More than any other structure, the key to understanding arch bridge behavior is conceptualising the flow of force. Force is attracted by stiffness. Stiff load paths will always be preferred over more flexible ones, but the more flexible ones will come into play if the stiff ones fail. It is worth appreciating that there are large differences in stiffness which may be present within an arch system, which is, in any case, hugely stiff. Before a structure collapses, it will exhaust all possible load paths. Where the possible paths are few and similar, that may be easy to imagine. Where they are many and quite different, the concepts are more difficult. It is very rare for the forces in an arch to be so high that crushing failure precedes the formation of a mechanism (para 2.3). Larger arches, though, may excessively deflect before reaching the ultimate load; in that case, deflection may be important. Shear is very rarely a problem. This concept will become clear after reading following paragraphs.

2.2.2 Line of thrust

Since 1695, when Robert Hooke first understood the action of an arch, it has been common to use a line of thrust to describe the behavior. Hooke noted that an arch is essentially the mirror image of a hanging chain (un-bonded links). For understanding, it is best to think of a masonry arch as constructed from un-bonded blocks. It is then clear that stability depends entirely on the balance of forces and not on the bending strength of the material.

Fig: 2.2 shows an arch made up of un-bonded blocks. Only inclined force/thrust (R_1) come from the abutment in a particular direction (say A) depending upon geometry of the arch. First block is acted upon by this thrust R_1, vertical load due to dead load/superimposed dead load/live load if any (V_1) and earth pressure of fill (E_1). Resultant of forces is vector sum of R_1, V_1 and E_1. This resultant force has different magnitude (R_2) and direction (B) than that of R_1. Similarly when it passes through next block, it's magnitude and direction change. Line joining the direction of thrust is called line of thrust. As long as this line of trust remains within middle half of arch ring, arch remains safe.

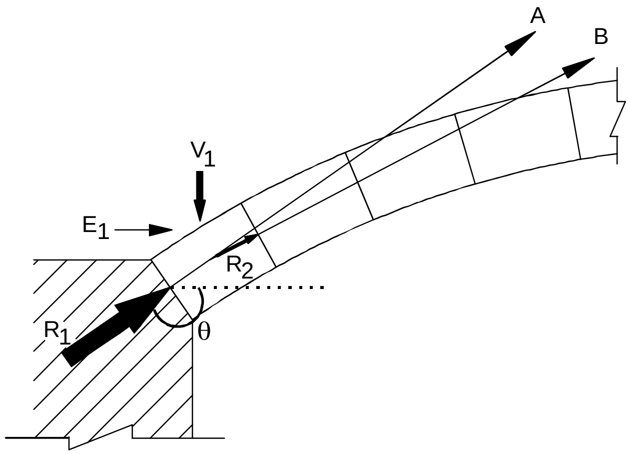


Fig. 2.2 : Showing force flow

Initial angle which R_1 makes with springing (θ) is very important. θ in case of semicircular arch is 90^{O}. For semicircular arch, minimum thickness required for stability is 12% of radius of arch. If θ is 120^{O}, minimum thickness required is only 3% of radius of arch.

A thin bare arch having only self weight is shown in Fig: 2.3, red line shows line of thrust.

Fig. 2.3 : A Bare arch. Red line show "line of thrust"

A concentrated live load will disturb the reactions. If the weight of the arch is very small in relation to the concentrated live load, the load must be supported by two direct forces passing through the arch to the abutments (Fig. 2.4). The optimum form of the arch is when two straight pieces meeting at a point where the load is applied. Fig. 2.4 shows two slightly curved lines (curved due to small dead weight) passing from abutment through arch barrel and meet at concentrated load location. If there is sufficient self weight, the distributed self load must also be carried and the forces trace a curve towards the abutments (Fig. 2.5). If the live load itself is distributed, the flow of force curves yet more gently (Fig. 2.6).

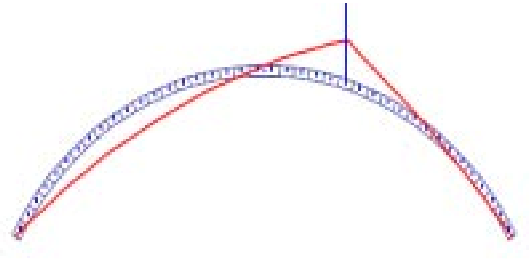


Fig. 2.4 : A thin arch with concentrated load. Red line shows "line of thrust"

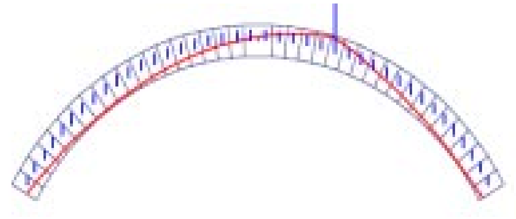


Fig. 2.5 : A thick arch with concentrated load. Red line show "line of thrust"

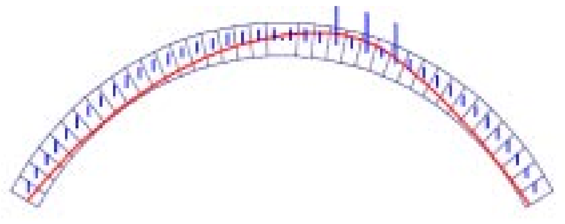


Fig. 2.6 : A thick arch with distributed load. Red line show "line of thrust"

Above simplified description ignores issues of strength and a number of these must now be addressed. Even old mortar has some tensile strength. This is unlikely to be sufficient to prevent the arch cracking at points where the thrust deviates greatly from the center of the arch. Very little strength may, however, be enough to ensure the integrity of the remaining sections, holding the rings together and preventing other radial cracks from forming, even preventing the formation of new cracks under live load. This last issue is very important, because formation of new cracks and the closing of existing ones is one mechanism by which an arch may deteriorate.

Fig. 2.7 shows the zones which would theoretically crack under the thrust of Fig. 2.6. In practice, multiple cracks would only form where the thrust is close to the boundary of the arch for some distance. Another issue of strength is that of sustaining compression, especially when the thrust comes close to the edge of the arch. This compressive force is unlikely to cause a problem unless the stress which results is particularly concentrated, which will happen if substantial rotation takes place locally.

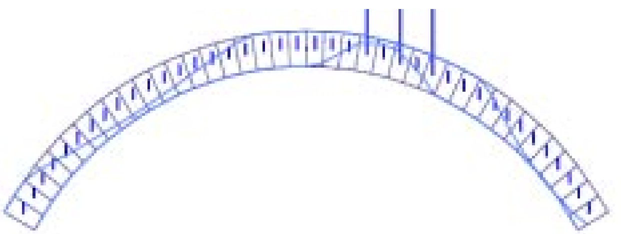


Fig. 2.7 : Areas which will crack due to loads as shown in Fig. 2.6

As far as shear strength is conerened. Two directions of shear force i.e through the depth of the ring and tangential to it are of significance. Through the depth of the ring, shear is rarely a problem. If the thrust is transmitted by the arch, it is unlikely that there will be sufficient shear at a section to overcome the clamping force generated.

The tangential shear is important particularly to brick rings. It raises the question of how a brick ring made of independent layers bedded together with mortar can be regarded as a contiguous structural element. In fact, the transmission of thrust from ring to ring normally takes place over a very great length. As a result, the shear stress imposed on the mortar bed is very small and easily accommodated.

2.2.3 Effect of fill

Soil fill over an arch modifies behaviour of arch in a number of ways.

- The most obvious is in dramatically altering the scale and distribution of dead load. Also important is that the soil does not press vertically downwards, but generates a component of horizontal force too (Fig. 2.8).

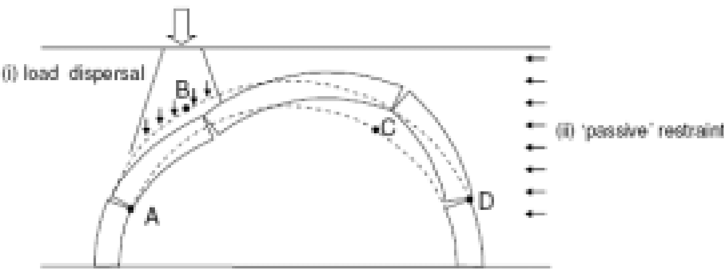


Fig. 2.8 : Effect of fill

- There is a further effect of fill which could be quite dramatic. If the arch distorts under load, it is likely to do so by deflecting downward and away from the near spring at the load point (point B in Fig. 2.8), and upwards and towards the near springing at the opposite side (point C in Fig. 2.8). However small are these movements, there will be some reduction in soil pressure on the falling side, and some increase on the rising side. Thus the fill will generate forces tending to resist the movement.

- There will also be distribution of load through the fill in the transverse direction.

2.2.4 Effect of arch width

The arch itself is clearly capable of distributing the effect of a load. If a live load is applied to the surface, the line of thrust will kink locally and flatten away from the load as in Fig. 2.5. In a wide arch, this is not the complete picture. The live load component of thrust must be concentrated at the point of application of load, but will not remain so all the way to the abutments. In masonry walls, a concentrated load is often assumed to distribute at 1:1 each side of the line of application. In this sense, an arch is just a wall rolled over. There are neither tests nor analytical results to demonstrate this behaviour but it is clear that the effect of the applied load cannot remain concentrated all the way to the abutments.

2.2.5 Effect of arch edges (ends parallel to track)

Edges can be free or fixed. In most cases of railway bridges, edges are fixed. The edges of an arch will become important if the width is less than about four times the span. If the edges of an arch are prevented from moving up or down (a rotation fixity is not feasible), the flow of force will be modified near the edges. Referring to Fig. 2.8, It will be clear that the arch would have to bow upwards to the right and downwards to the left. More significantly, it would need to sway sideways at the crown. This last movement is applied to a hugely stiff flat panel of masonry and will be resisted much more than either of the out of plane actions.

Spandrel walls are usually not bonded to the arch edge so that the resistance to movement they provide is strictly limited. Movement away from the spandrel (the left hand side of Fig. 2.8) meets negligible resistance The sideways movement of the crown is resisted at the edges by the combined action of thrust round the arch and horizontal or inclined force from the spandrel. At the right of Fig. 2.8, the upward movement will be resisted and the arch will tend to bow upwards across its width, transferring force back into the swaying element. All the movements will thus be resisted in some degree. One result of this resistance to movement is that the arch becomes much stiffer in relation to the soil. Distribution of live load will be reduced and the modification of soil pressures towards active and passive will also be constrained. Until the arch cracks, most of the stabilising force will come from the spandrels. Since cracks do sometimes form under the spandrel walls, it is necessary to assume that the soil does the work, because, in the limit, it may have to.

2.2.6 Effect of skew

Skew arches are very complex structures. All the discussion above has assumed that it is possible to regard the abutments as fixed. In a skew bridge, this may not be true. The assumption of rigid abutments does, however, provide a useful starting point for discussion. Force will always take the stiffest route to the foundations. Forces applied near the centre of a skew arch will flows by the shortest (normal) route to the abutment. Near the edges, that ceases to be possible and the arch forces must span on the skew (Fig: 2.9).

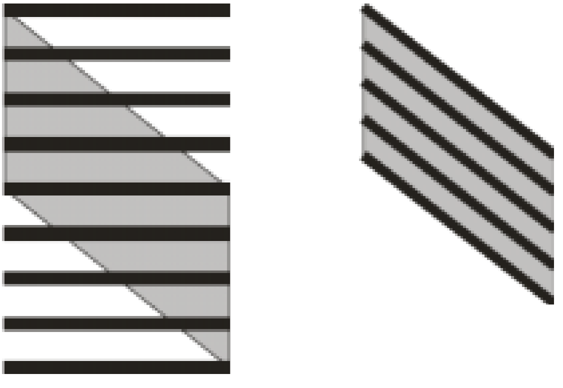


Fig. 2.9 : Showing square and skew load paths

The horizontal forces applied to arch abutments are usually greater than the vertical. An arch can twist with relatively little resistance, so slight settlement of one end of an abutment will have little effect. If the abutment tilts back slightly, (which is likely at the obtuse, heavily loaded corner than at the acute) arch force will start spanning skew. In a 10m span arch with 2.5m rise and 45^{o} skew, a differential movement of 1mm at the springing is sufficient to ensure that the thrust operates on the skew rather than square line. Realistic analysis of this complex behavior is beyond scope of this book.

2.2.7 Effect of Abutments

The actions required of abutments vary considerably. An abutment supporting a shallow, large span arch will be pushed back into the surrounding ground. It is therefore required to distribute the thrust over as large a contact area as possible. In a tall abutment carrying a high rise arch, the thrust at the top may be largely vertical and the wall must act as a retaining wall, with some support from the arch. Live loads modify these actions as they move. A load approaching an arch compresses the soil and generates horizontal pressure which pushes the abutment forward and shortens the span. The farther abutment will usually move rather less and the crown of the arch will be forced to rise. With a load near the centre of the span, the thrust in the arch is increased and both abutments will move back. Finally, as the load leaves the bridge, the increased pressure it generates will push the farther abutment into the span, thus shortning the span. These movements may be small, but are readily measurable.

2.2.8 Effect of Piers

Piers clearly support the two adjacent spans. It is also clear that they can offer little resistance to horizontal movement. All the thrust is passed from span to span. If the effect of a live load distributes in a fan across the width of the arch as it approaches the springing, it follows that the full width of an adjacent span will be called into play to resist toppling of the pier. The spandrel walls which are very stiff, also come into action.

2.2.9 Effect of Backing or Haunching

Any arch with an included angle greater than 60 degrees is likely to have solid masonry extending above the extrados at the springing. If this is brought to a horizontal top as in Fig. 2.10, it is called backing. An alternative form has an inclined top, tangential to the arch ring which will be referred to as haunching. Haunching is more normal on shallower arches, or on single spans and end spans of viaducts. In many railway viaducts, there is a solid layer with internal walls above. Whatever form these details take, the effect is to greatly increase the stiffness and load capacity without increasing the over all depth of the structure. Indeed, it is possible to reduce the ring thickness if there is firm haunching or backing.

Fig. 2.10: Showing "Backing"

2.2.10 Effect of ring thickening

Stone rings are commonly thicker at the springing than at the crown. This is often achieved simply by choosing deeper stones near the springing. An equivalent effect was sought in brick rings where it is normal to find additional rings of brick within the main arch. These are rarely expressed at the exposed edge, being present only between the spandrel walls.

2.2.11 Effect of internal spandrel walls

Internal spandrel walls are normally set above a block of solid backing. The walls may typically (but not necessarily) correspond with the rail positions in a railway viaduct. These walls provide yet further stiffness and strength and also reduce weight if the void between them is not filled. To maintain flexibility in the ring, most engineers truncated the internal spandrel walls at the 1/4 or 1/3 span position. There are viaducts, though where the internal spandrel walls pass completely over the crown. The deck may be formed from stone slabs or brick jack arches. Photo 2.11 shows the spandrel walls exposed in a viaduct following the destruction of a pier. The viaduct was held up by tension in the railway tracks back over the remaining deck. The arches have fallen away from below the spandrels, but the vertical end is also clearly visible.

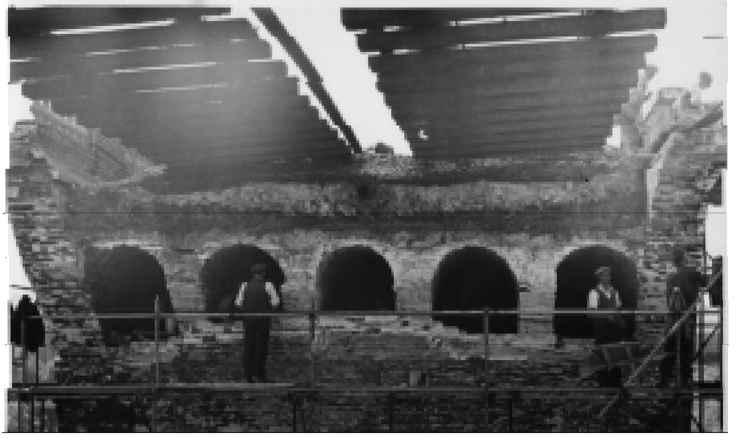


Photo 2.11 Showing internal spandrel walls

2.2.12 Effect of strength of material

Effect of material strength on arch depends upon how arch ultimately fails. Normally, a masonry arch will not fail due to crushing of masonry, it will fail due to development of tension and subsequent formation of a collapse mechanism. Therefore strength of masonry is not very important (beyond some minimum required). Existing arches with bad looking brick cannot just be assumed to have significantly less strength, a proper analysis is important. Example given in para 3.6.3 makes it more clear.

2.3 MODES OF FAILURE OF ARCH

Collapse failure of an arch can be either after formation of sufficient number of hinges or sliding of blocks.

Number of hinges (locations where rotation/sliding takes place after development of tension) required to be formed at the time of collapse in a structure is equal to number of indeterminacy of structure plus one. For example, a simply supported beam is determinate structure as such indeterminacy of structure is zero, so number of hinges required are 0+1=1. A single span two hinged arch has indeterminacy of structure of 1, so number of hinges required for collapse is two. Ends of the arch, though

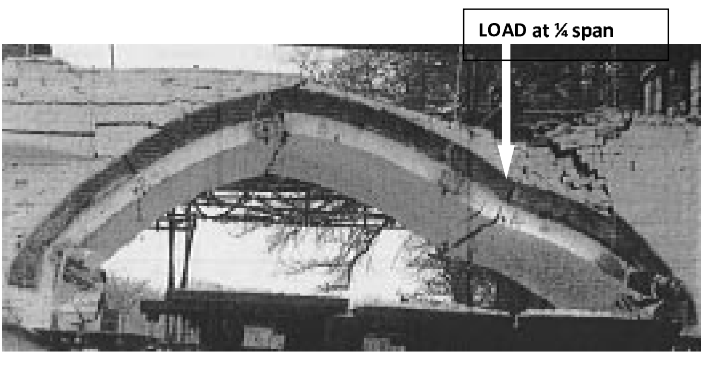


Photo 2.12 Showing collapse mechanism

assumed as hinged, are not so in actual practice, these are partially fixed. So actually four hinges are formed (two near the ends) before a single span arch collapses (see Photo 2.12, where four hinges can be clearly seen, hinge means a clear wide transverse crack). Critical load position, which causes mechanism in single span arch, is quarter of the span not centre of the span.

For two span arch, number of hinges required are eight (normally seven as one hinge is common).

Typical failure mechanisms are shown in Fig. 2.13 a to e.

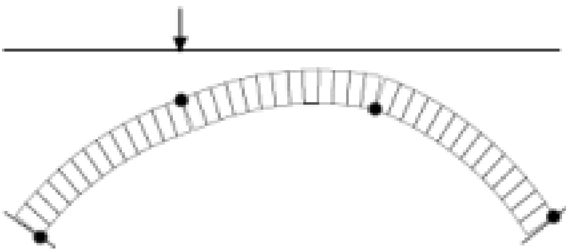


Fig. 2.13 (a) single span: 4 hinges

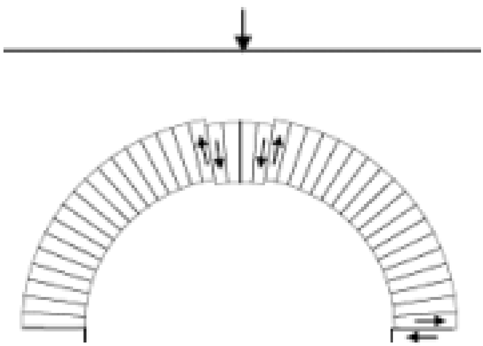


Fig. 2.13 (b) single span: sliding only

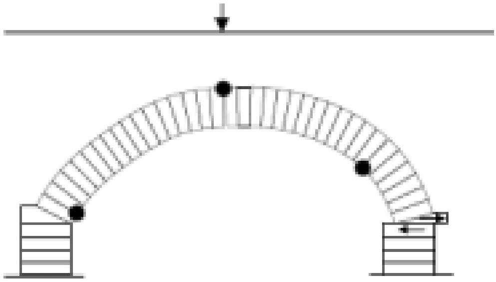


Fig. 2.13 (c) single span: hinges & sliding

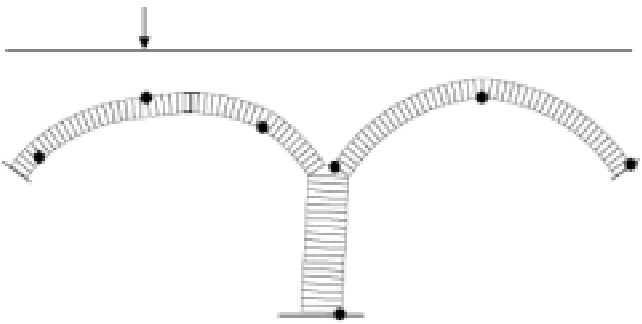


Fig. 2.13 (d) multi-span: 8 hinges

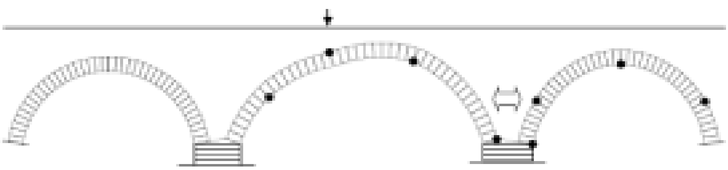


Fig. 2.13 (e) multi-span: 8 hinges (interaction between spans through spandrel zone)

Hinges in arch form successively with increase in load. There can be considerable difference in load at formation of first hinge and last hinge. Results of one of the tests done on arch are as under: (Fig. 2.14)

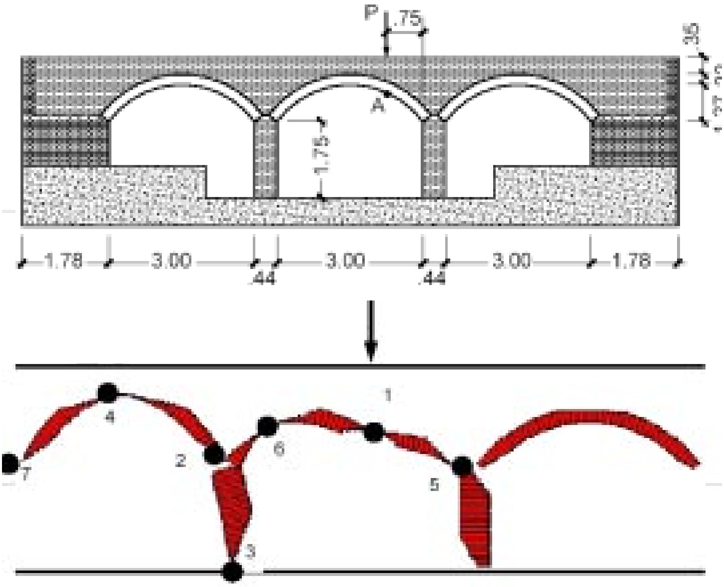


Fig: 2.14

Hinge no.	**1**	**2**	**3**	**4**	**5**	**6**	**7**
Load (KN)	**128**	**202**	**202**	**234**	**245**	**245**	**256**

It can be seen that load at the time of formation of last hinge (just at collapse) is about twice the load at the time of formation of first hinge.

2.4 DEFORMATIONS OF ARCH BRIDGE

Deformation of arch bridge is normally measured as vertical deflection at crown and spread of arch i.e increase of distance between supports. When train approaches arch, span is compressed and reduces by a small value and at the same time crown is lifted up by small value. When load comes on the bridge span spreads and crown moves down by significant values. Arch is considered good if deformations are low.

A typical change of span and vertical deflection of crown with train movement is shown in Fig. 2.15 and 2.16.

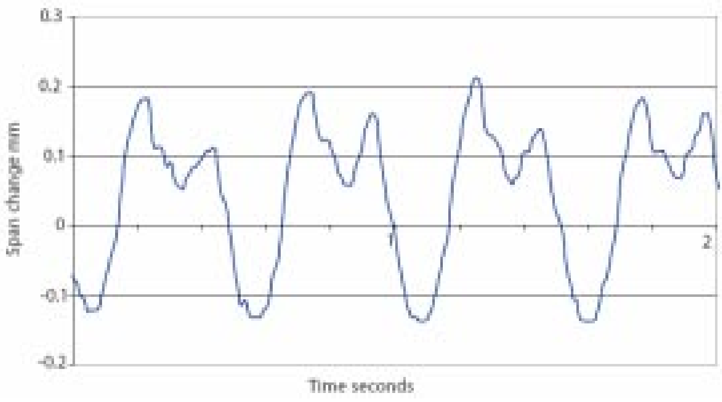


Fig: 2.15 Showing change in span

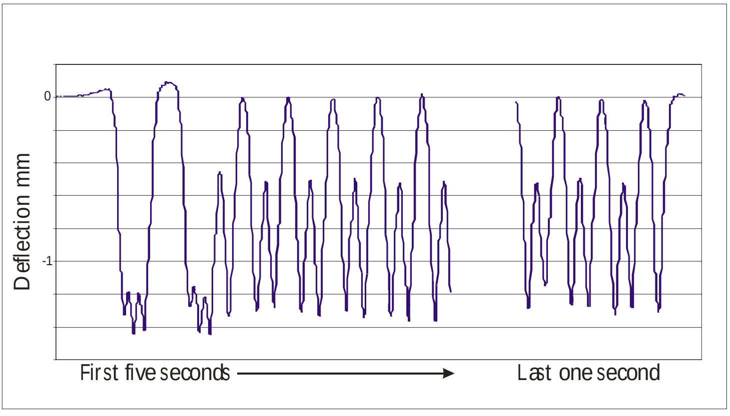


Fig: 2.16 Showing vertical deflection of crown

Load testing criteria of arch bridges is based on these deformations as per "Indian Railways code of practice for design and construction of arch bridges", which is discussed in para 3.5.

2.5 SHAPE AND THICKNESS OF ARCHES

Ideal shape of an arch for a fixed load can be theoretically derived, but actual shape of an arch has to be such that it can be easily constructed, inspected and maintained. Parabolic shape is perhaps more close to an ideal shape. In Indian Railways, most of the arches are segmental arches which means arch barrel is segment of a circle. This type of shape is very easy to construct and measure.

Thickness of arch barrel can be made up of different layers of pieces which together form total thickness as is in the case of brick masonry arches. These can be in separate layers or interlocked. Interlocked condition is better as sliding of layers cannot take place, but common practice is to have seperate layers due to ease of construction. In case of stone arches, normally there is only one stone thickness. Various configurations are shown in Fig. 2.17 (a) to (d).

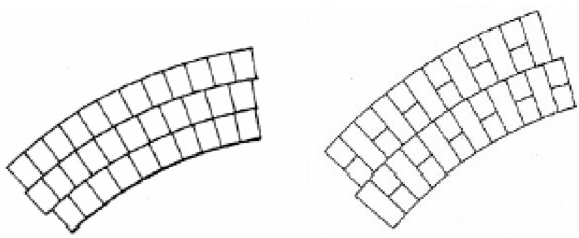


Fig. 2.17 (a) and (b) showing three and two brick layers

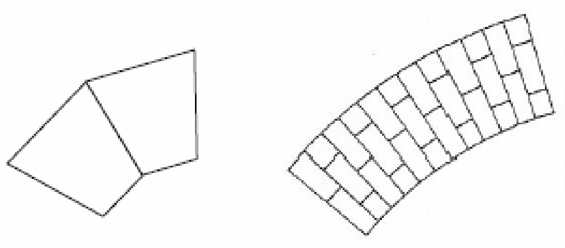


Fig. 2.17 : (c) and (d) showing single stone elements and interlocked brick arch

CHAPTER 3

ASSESSMENT OF LOAD CARRYING CAPACITY

Assessment of load carrying capacity of masonry arches is necessary for a several reasons. (a) Arch bridges deteriorate with time, and accordingly their capacity to carry load also declines. To maintain the safety of the railway it is thus necessary to confirm that load capacity of the arch is sufficient for the current and foreseeable applied loads without accelerated deterioration, and therefore that arches remain serviceable. (b)The loading to which bridges are exposed may change. Typically, axle loads, numbers of axles and vehicle speeds may increase, and it is necessary to confirm when new train movements are proposed that the arches on the proposed routes have sufficient capacity without eroding safety margins.

Assessment of load carrying capacity may be quantitative or qualitative. In the context of arch bridges the load capacity of an arch barrel is determined quantitatively and abutments and piers are generally assessed qualitatively. The assessed load capacity of arch barrel will be dependent on the geometric form, the materials used in the construction, the structural interaction of the parts and the condition of the structure.

3.1 ASSESSMENT PROCEDURE

Steps for assessing strength of masonry arch bridge are as under:

- Collect drawings, old inspection reports, past history of repairs etc.
- Take measurement of arch to assess any deformity in shape.
- Take note of all defects

- Determine masonry and soil properties as required.
- Use appropriate assessment methods.

3.2 DATA COLLECTION

All available information relevant to the structure, including record drawings, inspection and maintenance records, details of past performance and previous assessments, and any available ground investigation data should be collected and examined. The documents should be verified for correctness where ever possible. Fresh inspection for assessment should be carried out, which should include a dimensional survey and close inspection of the structure generally within touching distance. The form and materials of construction, dimensions of the arch and the condition of the arch and any deterioration or defects be observed and recorded. Photographs should be taken to assist in the subsequent determination or investigation of the implications of observed defects and deterioration.

Photo 3.1 : Thickness is more inside than at face. Situation can be reverse in some cases

Critical dimensions such as span at the springings, rise from a level survey of the springings and crown, width between parapets, thickness of the arch ring carrying rail traffic (this may not be the same as the number of rings visible on the face as shown in Photo 3.1), depth of fill between sleepers and ring, top of parapet to soffit of arch and to rail level sufficient to locate track positions on the arch.

Above data is sufficient when only empirical methods (discussed in subsequent paras) are employed for assessment. Where other mathematical modeling methods are used, following data is also necessary:

- hidden details such as saddling, haunching or internal spandrel walls
- material properties
- soil and fill properties

3.2.1 Defects in Arch Barrel, Piers and Abutments

The bridge should be inspected and the location and extent of any observed defects, such as cracks, settlement, crushed or spalling or otherwise defective masonry, external damage etc. including following should be recorded:

- thickness of the joints and the depth of any mortar loss
- presence of cracks, their width, length, position, number, and the degree of any displacement across the crack
- the presence and extent of any ring separation in a brick arch barrel such that a ring is not acting integrally with the rest of the arch
- location and extent of any loss of section due to spalling
- location of any displaced voussoirs and whether the displacement has taken place along one edge of the block or evenly across its full width;
- deformation of the arch barrel from its original shape
- damage by vehicles strikes
- the extent and location of water seepage as this can

provide information about the details of internal construction. The colour and nature of any leachates should be closely examined for signs of brick or stone slurry that may indicate internal movement.

3.2.2 Parapets and Spandrel Walls

Parapets and spandrel walls should be inspected for evidence of any defects and their extent recorded, including, but not limited to:

- tilting, bulging or sagging;
- lateral movement of parapet or spandrel wall relative to the face of the arch barrel;
- lateral movement of parapet or spandrel wall accompanied by longitudinal cracking of the arch barrel;
- weathering and lack of pointing;
- cracking, splitting and spalling;
- loosening of any coping stones;

3.2.3 Determination of Masonry, soil and backfill Properties

Properties of masonry, soil and backfill should be determined by non- destructive or partially destructive methods as described in para 3.7 and 3.8. Properties should be determined only to the extent required for a particular type of assessment.

3.3 LEVELS OF ASSESSMENT

Method of assessment can be simple based on empirical methods to complicated 3-D models. In order to determine the adequacy of a particular arch structure with the minimum degree of effort, the assessment should be carried out in levels of increasing refinement and complexity, with the initial level (Level 1) being based on the most conservative distributions of loads and analytical assumptions. If the structure is shown to be inadequate in relation to the required load carrying capacity at this level, assessment work should continue, with subsequent levels seeking to remove conservatism in the assessment where this can be justified. Judgment will be necessary at many points in the process to arrive at a sensible result. Levels of assessment according to the methods of analysis are;

Level 1	Simplest level using assumptions known to be conservative and, where appropriate, consideration of loading by real trains.
Level 2	Use of more refined analysis and better structural idealisation with approximate material properties.
Level 3	Same method as for level 2 but with reasonably accurate data in regard to materials strengths and structural details
Level 4	Use of sophisticated 2-D, 3-D modeling

3.3.1 Level 1 assessment

Method used for assessment	➢ MEXE, modified MEXE or simplified method
Input Parameters are based on	➢ Visual inspection ➢ Bridge record ➢ Former assessments
Results used for	➢ First ranking of bridges
Decision making	➢ Structures passing convincingly need not be assessed by higher levels ➢ Structures that fail or the adequacy is not convincing, or have damage pattern that cannot be assessed by simplified methods, the assessment procedure should be continued by higher level.

3.3.2 Level 2 assessment

Range of structures	➢ Structures failed at Level 1 ➢ Structures precluded from MEXE or other methods used for preliminary assessment (e.g. complex, oversize structures or structures with unique characteristics).
Method used for assessment	➢ Thrust line analysis, 2D frame models like RING.
Results used for	➢ Verification of preliminary assessment results. ➢ Determination of structural parts and parameters where further investigation should be focused.
Decision making	➢ For structures that are found to be of adequate strength convincingly (even with worst possible assumptions for the unknown parameters) the assessment by higher levels need not be done. ➢ Structures that fail at assessment or the adequacy is not convincing, the location and mode of further inspection should be determined. The rigorousness of further analysis must depend on the importance and complexity of the structure.
Input Parameters are based on	➢ Bridge records, visual inspection. ➢ Simple measurements (geometry, material composition and quality, approximate strength measurements etc.) ➢ Default assumptions by looking at other similar structures.

3.3.3 Level 3 assessment

Range of structures	➢ Structures failed at Level 2 assessment.
Method used for assessment	➢ Same as for Level 2 assessment, but the use of more than one method is advised in order to mitigate the effect of model uncertainty
Input Parameters are based on	➢ Destructive, slightly destructive and non-destructive testing carried out in-situ and in laboratory ➢ Default assumptions for some other parameters.
Results used for	➢ Verification of Level 2 assessment, ➢ Gaining better understanding of arch behavior ➢ Verifying serviceability ➢ Ranking of damaged structures ➢ Decision making on the mode of intervention

3.3.4 Level 4 assessment

Range of structures	➢ Structures failed at Level 3 assessment with a moderate degree ➢ Structures where the damage pattern requires more sophisticated methods ➢ Structures with unique characteristics
Method used for assessment	➢ 2D and 3D finite element (ANSYS, ABACUS) and discrete element models (ELFEN) which can accommodate the unique

	features of masonry arch behavior (e.g. non-linearity, very limited tensile strength, interaction between the arch and fill, transverse effects, etc.)
Input Parameters are based on	➢ Based on NDT or other tests on bridges ➢ Default assumptions
Results used for	➢ Verification of previous assessments ➢ Better understanding of arch behaviour ➢ Verifying serviceability or loss of serviceability ➢ Ranking of damaged structures ➢ Decision making on necessary intervention ➢ Planning of strengthening measures

3.4 METHODS OF ASSESSMENT

(a) Military Engineering Experimental Establishment (MEXE) method (Level 1)
(c) RING 2.0 method (Level 2 and 3)
(d) Archie method (Level 2 and 3)
(e) 2-D, 3-D FEM (Level 4)

In this book we shall limit our discussion to MEXE, Ring and Archie methods. For other higher level assessment methods, expert advice is needed.

3.4.1 MEXE method

The MEXE method was originally developed by Pippard for use in war times, and is a simple empirical procedure method which uses critical dimensions and observations of condition to determine the load capacity of the arch. There is no requirement to determine parameters of the material of construction. The equations involved in the MEXE method do not represent the behavior of a real arch, but represent the best approximation that was achievable without computers when the method was developed.

This method assumes arch as parabolic with a span-rise ratio of 4. The ring arch is considered as elastic but with pinned supports. Fill and masonry are considered to be of equal density. Assessment of single and multi-span span structures may be carried out using the MEXE method. The MEXE method is approximate and should only be used where:

- the clear span is less than 20 m
- the span/rise ratio does not exceed 8
- there is no evidence of significant ring separation for the rings under consideration
- the arch barrel is not severely deformed
- the arch does not support internal spandrel walls with vaulted construction
- the fill depth measured below the sleeper is not less than 1 m at the critical load point. Critical point lies at quarter span for semi-circular arches and 1/6 span for arches with 6:1 span/rise ratio.

(a) SCOPE

(i) This method gives the assessment of the strength of the ARCH BARREL ONLY. The strength of the bridge may be affected by the strength of the spandrel walls, wing walls, foundations, etc.

(ii) The MEXE may be used to estimate the carrying capacity of arches spanning up to 20m, but for spans over 12m it becomes increasingly conservative compared to other methods. The method should not be used where the arch is flat or appreciably deformed.

The long term strength of a brick or masonry arch is almost impossible to calculate accurately and recourse has, therefore, been made to an empirical formula based on the arch dimensions. The arch is first assumed to be parabolic in shape with span/rise ratio of 4, soundly built in good quality brickwork/stonework, with well pointed joints, to be free from cracks, and to have adequate abutments. For such an idealised arch, a provisional assessment is made and then modified by factors which allow for the way in which the actual arch differs from the ideal.

(b) GEOMETRIC DATA REQUIRED TO APPLY MEXE METHOD

(i) The span: L (m) (in the case of skew spans, measure parallel to the axis of the arch)
(ii) The rise at the crown: r_c (m)
(iii) The rise at the quarter points: r_q (m)
(iv) The thickness of the arch adjacent to the keystone: d(m)
(v) The depth of fill below sleeper: h (m)

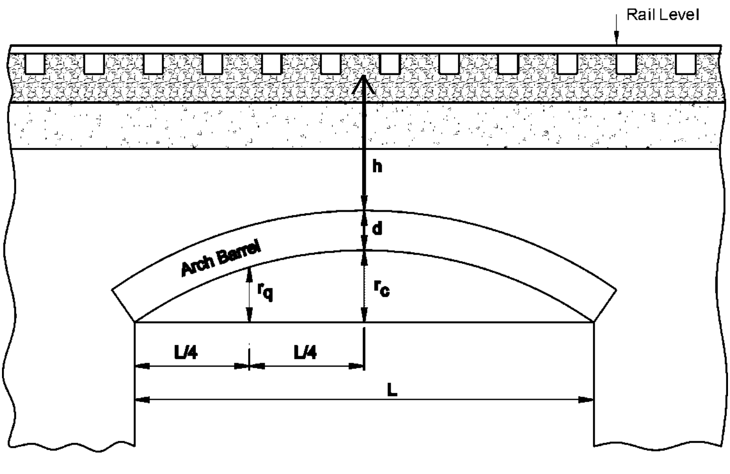


Fig. 3.2 : Dimensions of arch

(c) MATERIAL DATA REQUIRED TO APPLY MEXE METHOD

(i) Type of material used for the arch barrel
(ii) Type of construction of the barrel, ie are the voussoirs in courses or laid at random?
(iii) Condition of materials in the barrel, ie is there a lot of spalling, are the voussoirs sound or are they deteriorating due to weathering?
(iv) Deformation of the arch barrel from its original shap
(v) Positions of dropped voussoirs and the amount of drop
(vi) Width, length, number and positions of cracks
(vii) Type of filling above the arch and its condition
(viii) Position and size of crevices
(ix) Width of mortar joints
(x) Depth of mortar missing from joints
(xi) Condition of joint mortar

The appropriate measurements should be taken for the arch barrel thickness, If there are a number of voussoirs displaced, then this should be taken into account and the thickness of the arch barrel adjusted accordingly.

Note should be taken of any evidence of separation of the arch rings, particularly with regard to any additional rings which have been constructed in later years, and due account should be taken in the value assumed for the arch barrel thickness.

(d) DETERMINATION OF PROVISIONAL AXLE LOAD (PAL)

The provisional axle load is obtained from graphs, which are for different ring thickness as shown in Fig. 3.3 to 3.10. For a particular ring thickness at crown, graph gives span v/s PAL for different cushion depth. e.g. for crown thickness 30 cm, span 6 m and cushion 60 cm gives PAL of 318 KN (Fig. 3.3). The provisional axle load obtained is then modified by the various modifying factors and the condition factor.

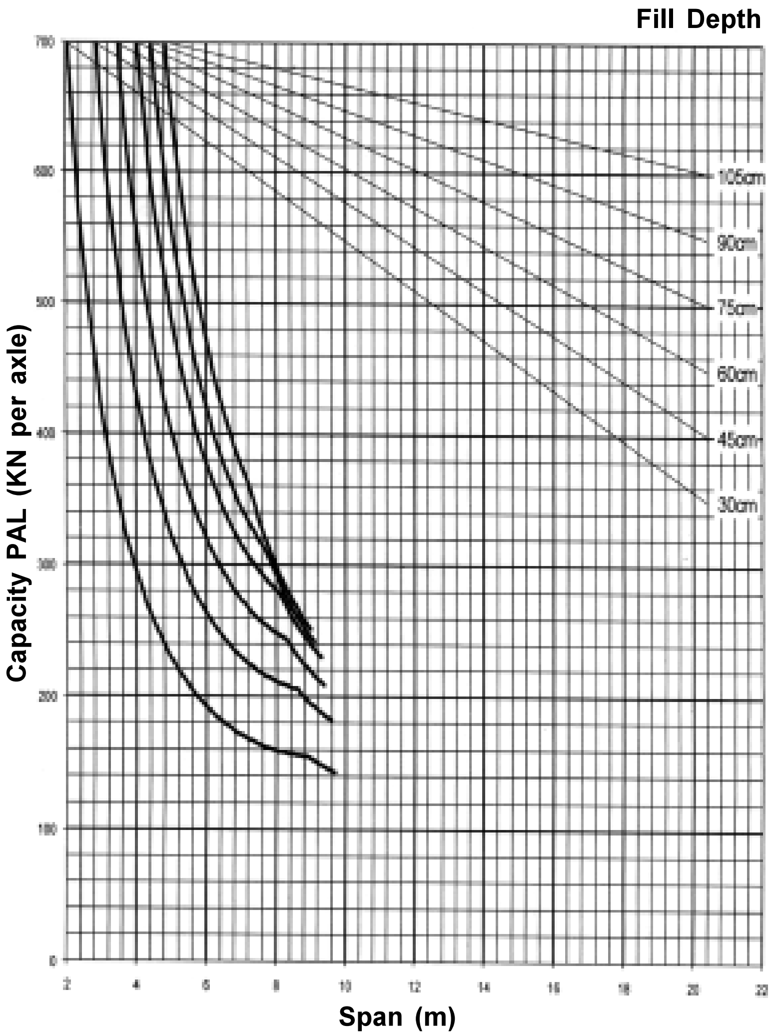


Fig. 3.3: For Ring thickness at crown=30 cm

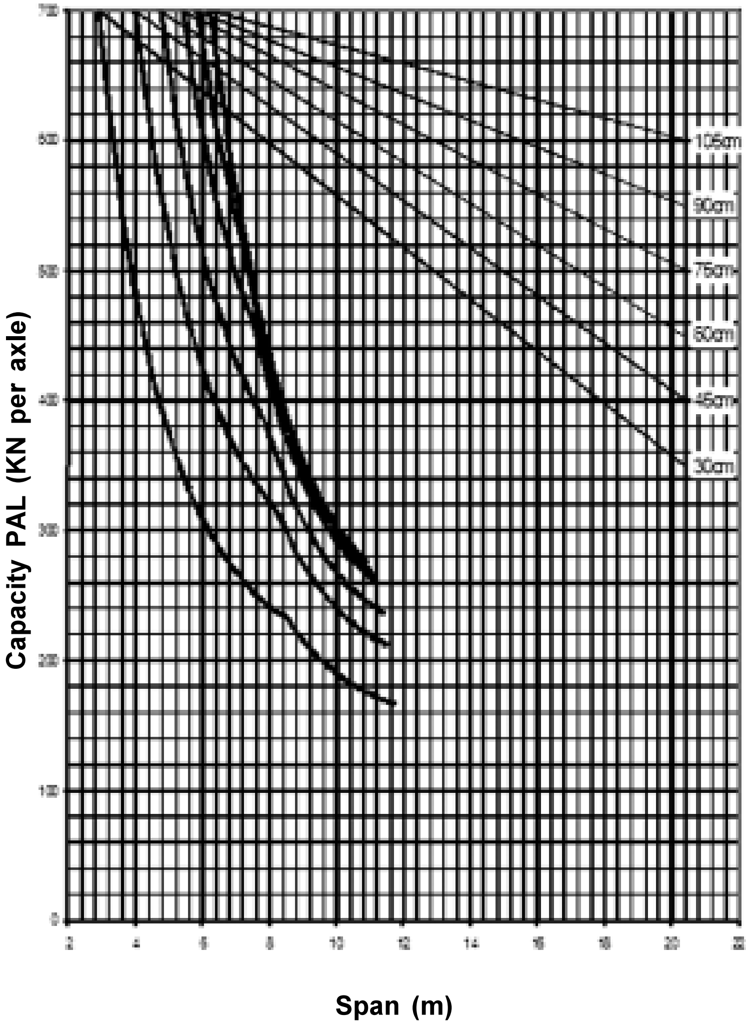


Fig. 3.4: For Ring thickness at crown=40 cm

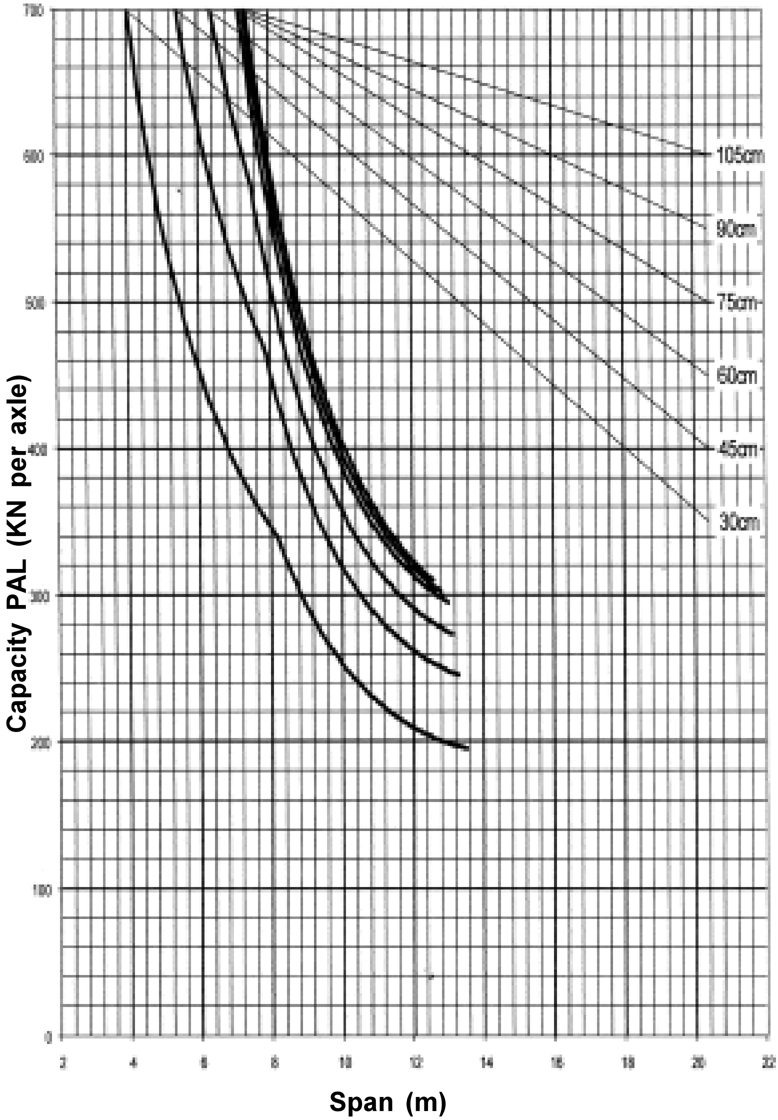


Fig. 3.5: For Ring thickness at crown=50 cm

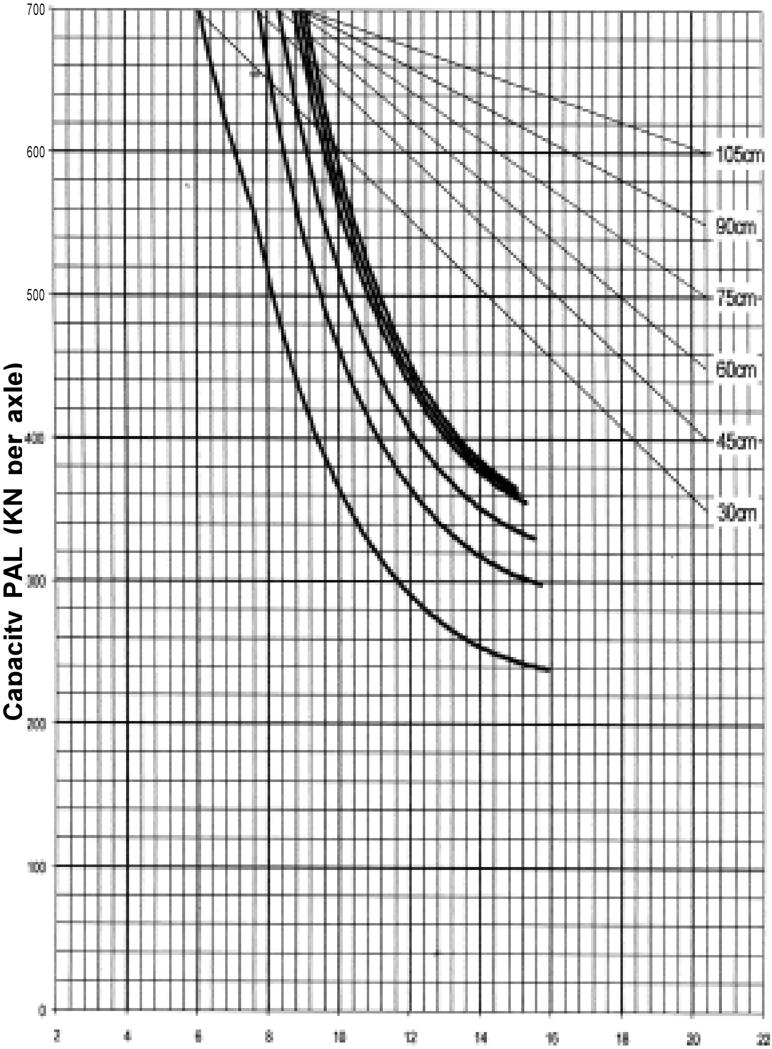


Fig. 3.6: For Ring thickness at crown=65 cm

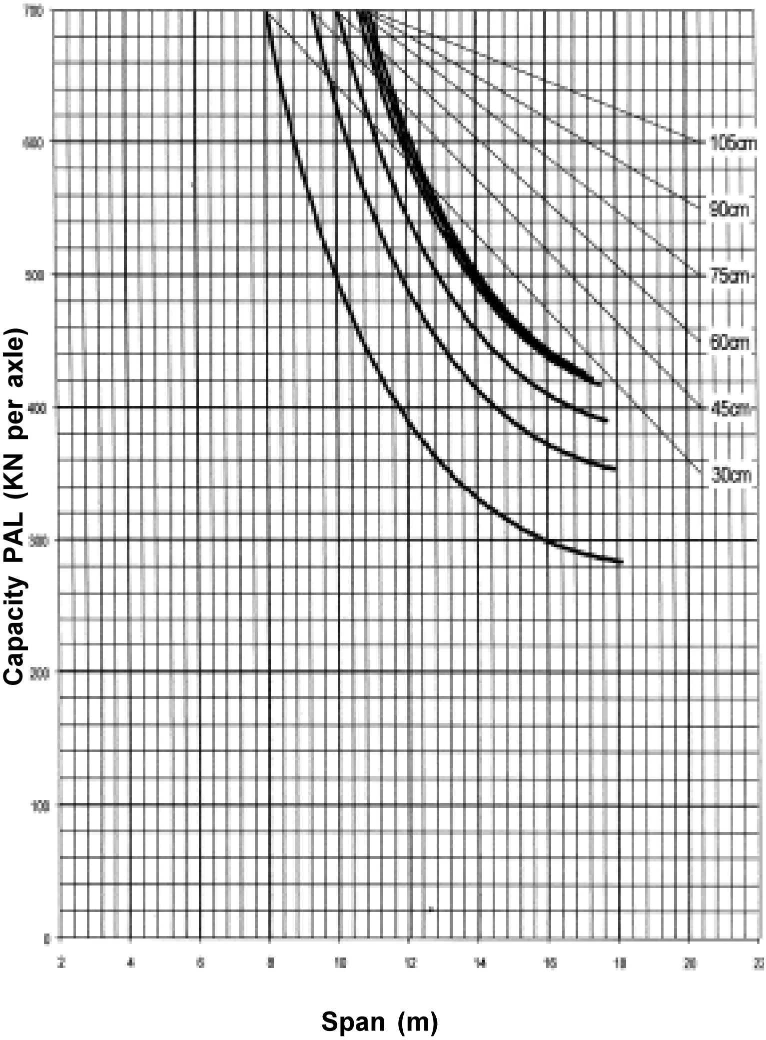


Fig. 3.7: For Ring thickness at crown=80 cm

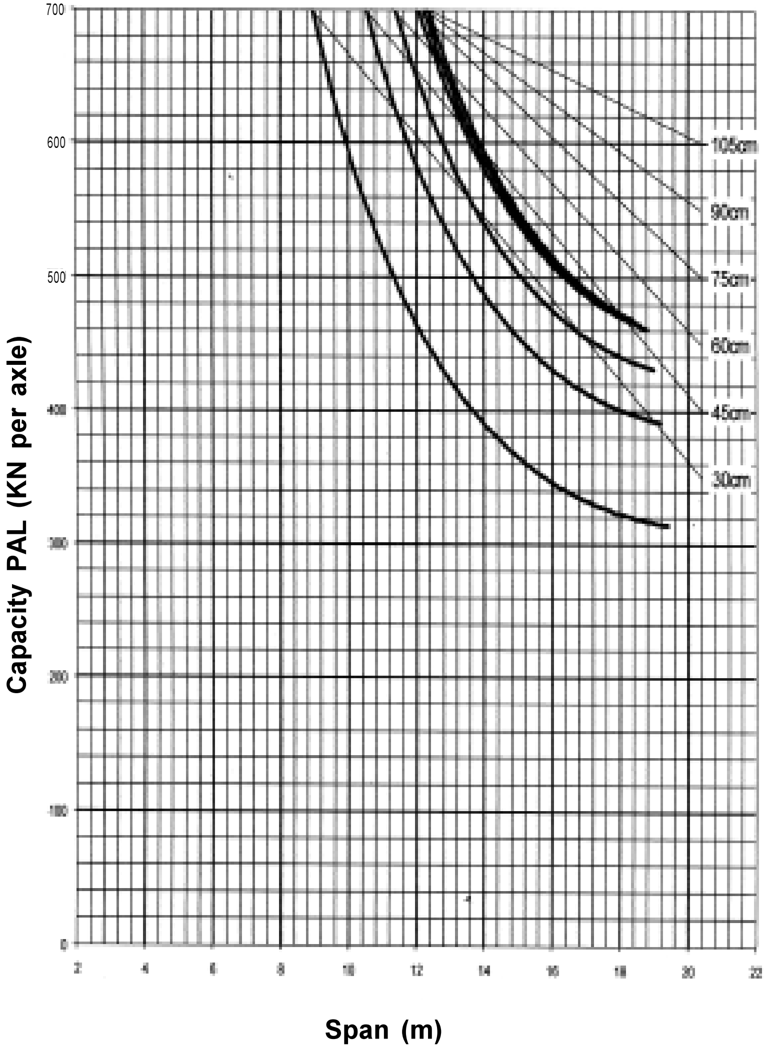


Fig. 3.8: For Ring thickness at crown=90 cm

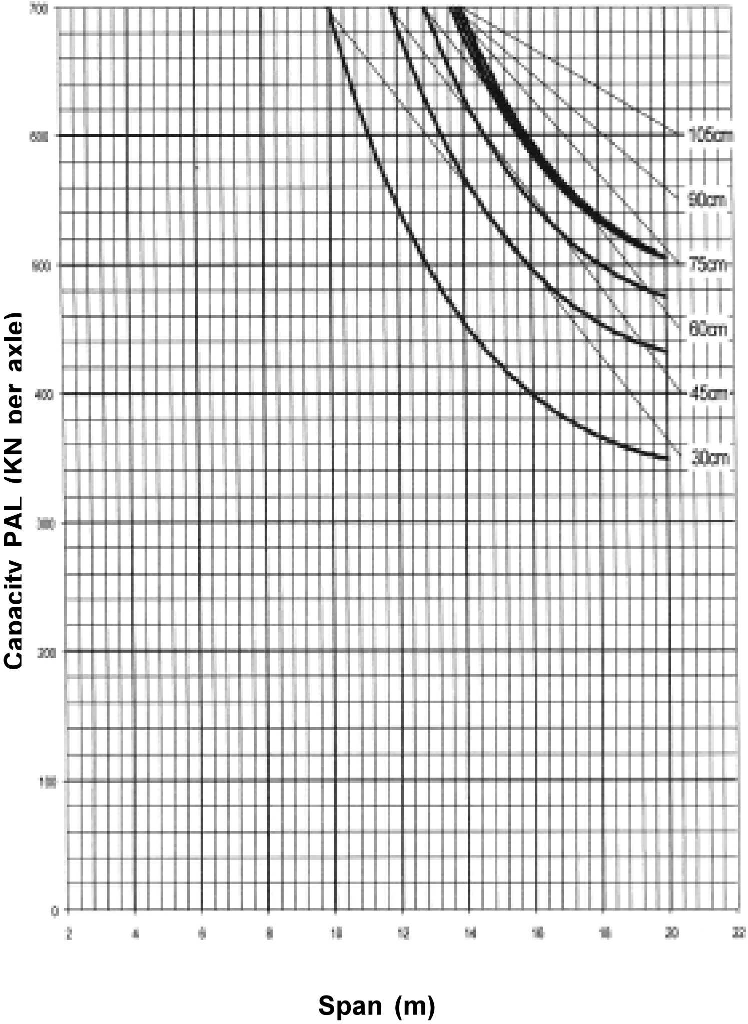


Fig. 3.9: For Ring thickness at crown=100 cm

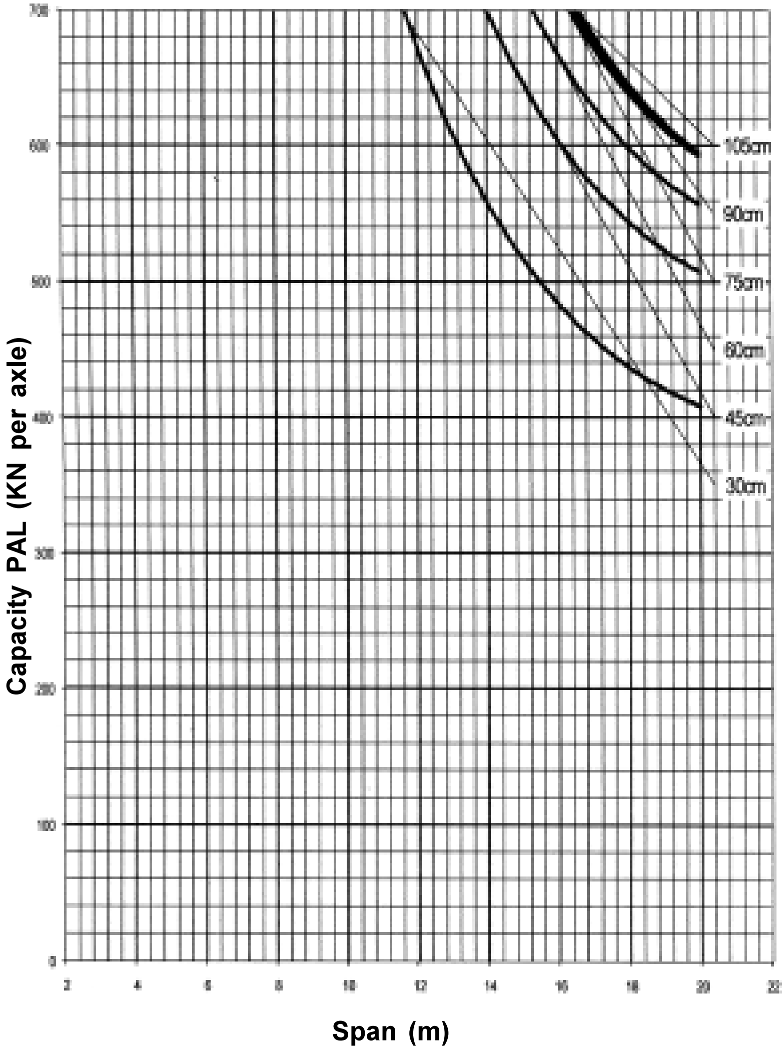


Fig. 3.10: For Ring thickness at crown=115 cm

(e) MODIFYING FACTORS

(i) Span/Rise Factor or shape factor (*Fs*): Flat arches are not as strong under a given loading as those of steeper profile, and the provisional assessment must, therefore, be adjusted. A span/rise ratio of 4 and less is assumed to give optimum strength and has a factor of 1. When the span/rise ratio is greater than 4, ***Fs*** should be obtained from graph in Fig. 3.11.

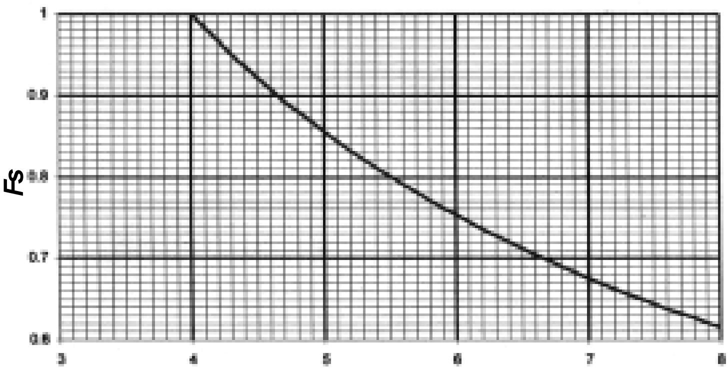


Fig. 3.11: Span/Rise Factor (*Fs*)

(ii) Profile Factor (F_p). The ideal profile has been taken to be parabolic and for this shape the rise at the quarter points, r_q = 0.75 r_c . The profile factor F_p for ratios of r_q / r_c shall be taken from Fig. 3.12

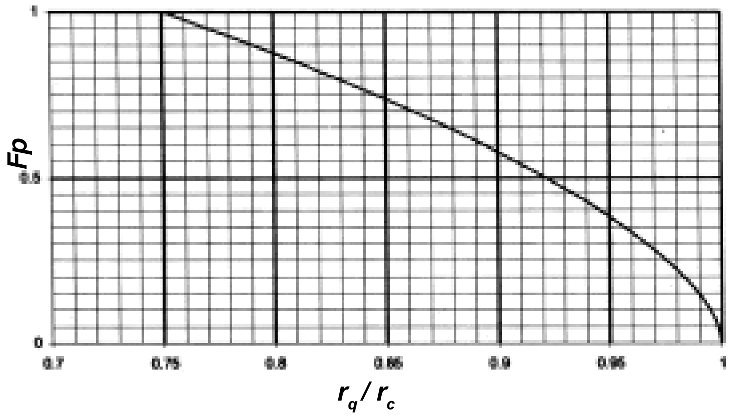


Fig. 3.12: Profile Factor (F_p)

(iii) Material Factor (F_m)

The material factor (F_m) is obtained from the following table:

Material	F_m
Soft brick and soft stone	1.0
Hard brick	1.2
Mass concrete	1.2
Hard stone	1.5

(iv) Condition Factor (*Fv*)

The estimation of the preceding factors is based on quantitative information obtainable from a close inspection of the structure, but the factor for the condition of the bridge depends much more on an objective assessment of the importance of the various cracks and deformations which may be present and how far they may be counter-balanced by indications of good material and workmanship. A quantitative estimate of the arch barrel condition factor should be made by the engineer, the value selected being between

0 and 1.0. A low factor should be taken for a bridge in poor condition while 1.0 may be taken for an arch barrel in good condition with no defects. It is important that the engineer dissociates the “condition factor” from the “material factor” and the “crack factor” as these are dealt with separately.

Ranges of condition factors for defects affecting the stability and load carrying capacity of the Arch Barrel are given below for crack patterns resulting from specific causes. The choice of factor is made from a critical examination of the size, shape and importance of the various defects. The overall figure representing several defects should be based on the relative importance of the worst type of defect present. It will not necessarily be derived by multiplying the factors for several separate defects together:

(a) **Brick arch**

Good condition - No spalling	1.0
Fair condition, slight spalling (between 0% and 25% of arch surface) and no bricks missing	0.9
Poor condition, significant spalling (over 25% of arch surface) and/or bricks missing	0.8

(b) **Stone arch**

Good condition - No spalling	1.0
Fair condition, slight spalling (between 0% and 25% of arch surface)	0.9
Poor condition, significant spalling (over 25% of arch surface) but no stones loose or missing	0.8
Some stones loose or missing, severe loss of jointing material in undressed stone arches	0.75

(v) Crack Factor (*Fc*)

i) **Longitudinal cracks due to differential settlement in the abutments**: large cracks, ie > 3mm, indicate that the barrel has broken up into independent sections.

(ii) **Lateral cracks or permanent deformation of the arch:** which may be caused by partial failure of the arch or movement at the abutments. These faults can be accompanied by a dip in the parapet which may be more easily observed.

(iii) **Diagonal cracks:** These normally start near the sides of the arch at the springings and spread up towards the centre of the barrel at the crown. They are probably due to subsidence at the sides of the abutment. Extensive diagonal cracks indicate that the barrel is in a dangerous state.

(iv) **Cracks in the spandrel walls near the quarter points**: These frequently indicate flexibility of the arch barrel over the centre half of the span.

The unfavorable defects which do not affect the stability of the arch barrel but may affect the stability of the track structure are indicated below, with a description of their significance:

(i) **Longitudinal cracks near the edge of the arch barrel** are signs of movement between the arch and spandrel or bulging of the spandrel, caused by the lateral spread of the fill exerting an outward force on the spandrels. This is a frequent source of weakness in old arch bridges and the proximity of the carriageway to the parapet should be taken into account.

(ii) Movement or cracking of the wing walls is another common source of weakness in old bridges and occurs for similar reasons to (i) above

Where the bridge consists of multi-span arches and the strength of intermediate piers is in doubt, the structure should be examined for cracks and deformation arising from any weakness in the piers.

Following values of F_c can be used normally.

a)	No cracks	1.0
b)	Longitudinal cracks:	
i)	Outside the centre third of the arch, less than one tenth of the span in length	0.95
ii)	Outside the centre third of the arch, longer than one tenth of the span in length	0.90
iii)	Within the centre third of the arch, less than one tenth of the span in length	0.90
iv)	Within the centre third of the arch longer than one tenth of the span in length	0.85
c)	Lateral and diagonal cracks:	
i)	Up to three small lateral or diagonal cracks less than 3 mm in width and less than one tenth of the arch width	0.90
ii)	Numerous small cracks as above in the centre third of the arch	0.60

(f) DETERMINATION OF MODIFIED AXLE LOAD (MAL)

Modified axle load = PAL . Fs . Fp . Fm . Fv . Fc

This is permissible axle load on bridge excluding dynamic effect. Dynamic effect should be estimated as per bridge rules. Bridge can be considered safe for multiple axles of above calculated MAL.

3.4.2 RING method

This method is based on limit analysis based on the static approach to obtain the lower bounds of the collapse load by fulfilling the equilibrium and yield criteria of the plastic theorems. This is the traditional 17th century approach, such as the middle-third rule, that had been assumed as the basis for several rules used for the construction of arch bridges. This approach to arch behavior is also the simplest. If line of thrust exists within the arch which can sustain the applied loads. Where the stiffness of the arch can be regarded as infinite, this approach will be sufficient. In fact, the question of realistic material strength can be overcome by ensuring that the thrust remains within the arch. This is effected by constraining the thrust line, or by creating a zone of thrust through the whole arch, which is just sufficient to transmit the forces.

Ring 2.0 is a rapid analysis tool for masonry arch bridges. The software is based on the 'mechanism' method of analysis. It can be used for single and multi-span bridges. It has special features like

- Automatic identification of the critical failure mode in multi-span bridges, even if this involves only a single span
- Failure modes involving sliding, if critical
- Multiple arch ring, presence of arch backing material
- Multiple load case facility
- Automatic detection of 'passive' pressures
- User-definable masonry compressive strength
- User-definable arch and backfill profiles

This is user friendly software and can be downloaded from www.masonryarch.com.

Data is filled in following steps;

Step 1 - General Project Settings

Firstly, specify the type of bridge to be analyzed.

Step 2 - Geometry

This includes data about the abutments, spans, piers and fill in several of the steps, there is an 'Advanced' button that will allow explicit modelling of the current bridge feature (It needs to be used only in special circumstances. Geometric detail once fed can be modified.

Step 3 - Partial Factors

Insert the partial factors of safety

Step 4 - Materials

Detailed properties for masonry, back fill are entered. If all properties are not known, default values given can be used with care.

Step 5 - Add vehicles

Using the vehicle database, specify the vehicles to be used in the current project. MBG loading is already available in database, any other loading like HM can be created easily:

Program can be then run. For details, help given with the program can be seen. One more advantage of this program is that user can analyse effect of various parameters like fill depth, crushing strength of masonry etc for better understanding of behavior of arch bridges.

3.4.3 ARCHIE- M method

This method is also based on line of thrust. Software program can be downloaded from www.obvis.com

Archie-M is graphical response which encourages users to explore potential behavior rather than expect to determine actual behavior.

The core working output is from the graphical screen where the user can immediately see the effect of loads and changes to the model. The program response is fast enough to allow a user to drag loads with a mouse to ascertain the critical position and load, and to observe the changes in the load path. Alternatively, an autorun facility runs a set of loads across the bridge and finds the worst load and position.

The basic minimum data required to be input into the program is span, rise, ring depth and surface level. The default arch assumed is a circular and with a strength of 5 N/mm^2. Default values are also provided for fill properties and material self weights. The model is less sensitive to material strength and fill angle of friction than it is to self weight. Low strengths, however, can reduce assessed capacity.

Five basic ring shapes: circular, three centered, semi-elliptic, pointed and flat, are provided within the model. To assess a distorted arch ring, it is possible to fit a curve to a set of measured points, and in such cases it is important that the data used is sufficient to define the shape effectively.

3.4.5 Elastic and Elasto-Plastic Analysis, 2-D, 3-D Finite Element Analysis

Commercial software are available for analysis by these methods. Modelling of an arch is ticklish and user should know how boundary conditions are to be applied. Guidance can be taken from “Brencich, A. Guide to High Level Assessment. International Union of Railways, January 2008”, copy of which is available in IRICEN.

These methods are not discussed in this book.

3.4.6 Assessment of Stability of Spandrels and Parapets

Currently there is no defined method for assessment of stability of spandrels and parapets to masonry arches nor any criteria or partial safety factors for serviceability against which such an assessment could be made.

3.4.7 Assessment of Damaged Arches

Damage in a masonry arch can affect the overall load carrying capacity of the bridge. Some types of damage are however not important when considering the short -term structural strength. The types of damage which can influence structural strength include:

- material degradation causing a reduction in material strength
- longitudinal fracture – dividing the arch barrel into separate arches
- transverse crack – as consequence of displacement of a pier or abutment
- delamination – ring separation in multi-ring brick arches
- loss of material – loss of mortar joint and/or masonry blocks from the arch barrel intrados
- deformation – change in the arch shape with respect to the original geometry
- displacement of a pier or abutment – horizontal and/or vertical movements of one support relative to another.

3.5 LOAD TEST

If none of the preceding analytical methods yields a sufficient result, consideration may be given to the use of an experimental approach to assess the load carrying capacity of masonry arch bridges. Load tests are expensive. Load test can be static or dynamic. A load test should be carefully designed, no "blind" load tests should ever be performed.

In load test, following observations are taken.

- compressive strain of masonry
- deformations/ residual deformations (important being vertical deflection at crown)
- increases in width or extent of load dependent cracks
- deflection of abutments (spread)

There are two approaches while conducting load tests.

First approach can be that arch is modeled with available FEM modeling tools; spread, strain in masonry, deformations at specified locations are estimated under applied loads and are than compared with actual value to validate the model. Once model is validated, ultimate load carrying capacity of arch can be estimated theoretically. Such type of approach can only be performed under expert advice.

Second approach can be to apply desired load and measure only deflection at crown and spread of abutments. If these two values are with in safe limit arch can be treated safe. This approach is not normally preferred, but being simple, has been used in Indian Railways extensively. Provisions for second simplified approach is given in "Code of practice for the design and construction of masonry arch bridges" issued by RDSO and are reproduced below.

For segmental and non-segmental arches of span 4.5 m to 15 m and span/rise ratio between 2 and 5, under the proposed load;

- The crown deflection should not exceed 1.25 mm
- The spread should not exceed 0.4 mm
- There should not be any residual deflection or spread after release of load

- There should not be any crack appearing on the intrados of bridge.

For load test, a proper assessment of target load, location of load and type of measuring devices is necessary. For planning a test load, reference should be made to 'Guide for load testing of masonry arch bridges' by UIC, copy of which is available in IRICEN library.

3.6 EFFECT OF VARIOUS PARAMERTES ON STRENGTH OF ARCH BRIDGE

Effect of fill depth, strength of masonry and backing & haunching on load carrying capacity of arch bridge has been covered in chapter 2. To make a better appreciation, one example has been solved below. The effect of fill, strength of masonry, backing and haunching etc. shall be different on different arches depending upon how a particular arch fails, but the following example shall give an idea of importance of these factors on load carrying capacity of a masonry arch bridge. Analysis has been made with the help of RING 1.5 software which is old version of RING 2.0.

3.6.1 Data of Arch analysed

(i) Brick masonry arch
(ii) Span – 3 x 6100 mm
(iii) Rise – 1510 mm
(iv) Height of piers – 4600 mm
(v) Width of pier at top – 1320 mm
(vi) Width of pier at bottom – 1570 mm
(vii) Ring thickness – 4 x 125 mm
(viii) No. of units in each ring – 93, 97, 101 & 105 respectively.

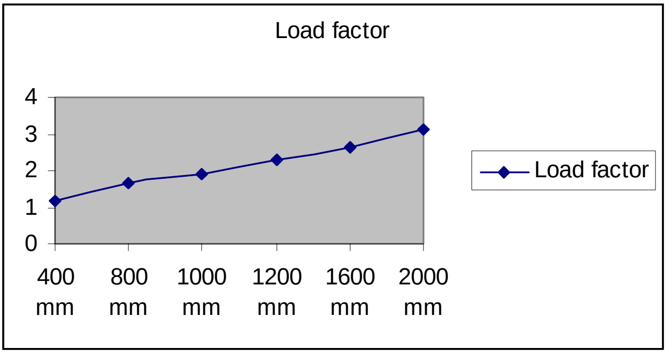
3.6.2 Effect of cushion i.e. fill depth

Effect of fill depth has been explained in para 2.2.3 in chapter 2.

Table 1 shows value of load factor v/s cushion for arch as per details in para 3.6.1 above.

Table 1

Fill Depth	400mm	800mm	1000mm	1200mm	1600mm	2000mm
Load factor	**1.15**	**1.66**	**1.90**	**2.29**	**2.63**	**3.13**



Crushing strength of masonry has been assumed as 5 N/mm^2 in above calculations. Load factors in the above table are without considering impact. If impact is also considered, difference in load factor shall be still higher.

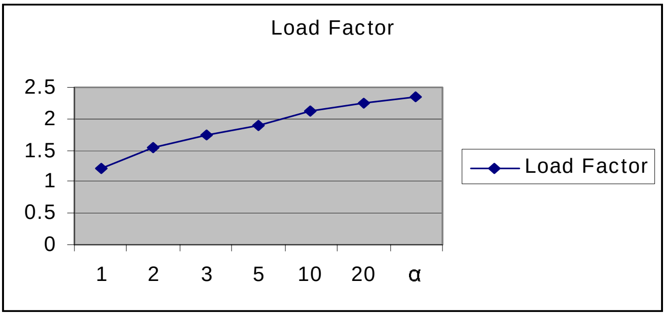
For this arch, load factor increases from 1.15 to 1.90 i.e. 65 % when cushion is raised from 400 mm to 1000 mm. The effect of cushion on different arches shall be different, but one conclusion can be safely drawn i.e. despite increase of dead load due to increase in earth fill over a masonry arch, capacity of arch to carry load increases substantially. This is true for all shapes and sizes of arches.

3.6.3 Effect of material strength

Effect of material strength on arch has been explained in para 2.3.12 in chapter 2. For analysis, arch as per details in para 3.6.1 has been assumed with fixed cushion of 1000 mm. Load Factor with various values of crushing strengths is given in Table 2.

Table 2

Crushing Strength N/mm²	1	2	3	5	10	20	Infinite
Load Factor	1.20	1.53	1.73	1.90	2.11	2.24	2.36



Normally a bad looking masonry shall not have crushing strength less than 3 N/mm^2. For the above analysed arch, load factor for bad masonry with crushing strength of 3 N/mm^2 is only 16% lower than very good masonry of crushing strength of 10 N/mm^2. Interestingly, even if masonry strength is made infinite, increase in load carrying capacity shall be marginal.

Therefore, it can be concluded that only look of an arch i.e. bad masonry or good masonry will not determine the health of an arch bridge. Existing arches with bad looking brick just cannot be assumed to have significantly less strength, a proper analysis is important.

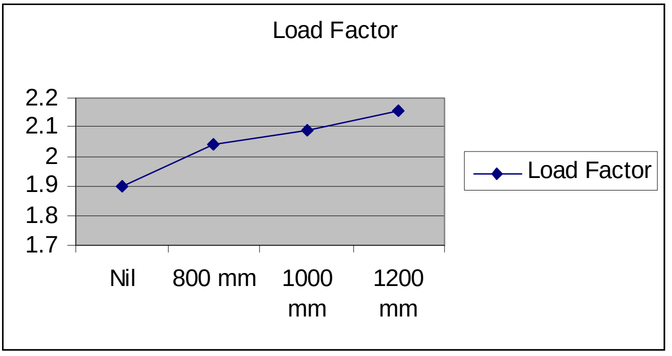
3.6.4 Effect of backing & haunching

Effect of backing and haunching on arch has been explained in para 2.2.9 in chapter 2.

Again the arch as per details in para 3.6.1 has been analysed for backing of 400 mm, 600 mm, 800 mm and 1000 mm. Crushing strength has been assumed as 5 N/mm^2 and cushion has been assumed as 1000 mm. Load Factor with various values of backing is given in Table 3.

Table 3

Backing	Nil	800 mm	1000 mm	1200 mm
Load Factor	1.90	2.04	2.09	2.16



Effect of backing in increasing load factor in this particular case is low because failure mechanism is such that advantage of backing is less. In some other cases, effect of backing can be too large. Take for example the following arch:

(i) Span – 1 x 4530 mm
(ii) Rise – 1500 mm
(iii) Material – Stone
(iv) Crushing Strength – 5 N/mm^2
(v) Depth of fill – 1000 mm
(vi) Thickness – 475 mm
(vii) No. of stones in ring = 20

Backing	Nil	600 mm	1000 mm
Load Factor	8.91	16.20	21.4

It can be seen that with a backing of 1 m, load factor increases from 8.91 to 21.4, i.e. increase of more than 100%.

Whatever may be the extent but definitely backing increases load carrying capacity of arch bridge. Therefore, it is important to explore availability of backing/haunching in an arch bridge and study its effect.

3.7 DETERMINATION OF MASONRY PROPERTIES

For analysis by various computer programs as discussed above or for modelling, masonry properties are required. Tests for compressive strength of brickwork should be performed on cylinders not smaller than 150 mm in diameter loaded on the lateral surface, Fig. 3.13. The cylinder should be drilled centred in the middle of a vertical joint, so that the test is performed in the same direction as the load acts; the test can be performed also on cut-stone masonry.

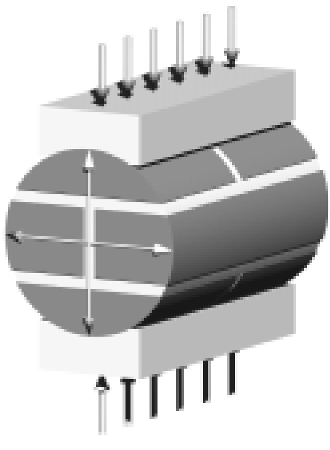


Fig. 3.13

The mechanical properties of stones can be deduced from compression tests on drilled cores. The core diameter depends on the dimension of the arch stones and on their quality taking into account that the core needs to be representative of the material, i.e. free of defects.

For existing mortars, the punching test of Fig. 3.14 can be used. A 10-to-25mm thick joint is embedded in-between two layers of gypsum (mortar joint needs to be wet when building the gypsum layer); the surfaces of the gypsum layers need to be plane. The punch is 20-to-30 mm in diameter; displacement-controlled tests (preferable) with the loading rate of 0.0075 mm/s; for force-controlled tests the loading rate should be close to 0.05 MPa/s.

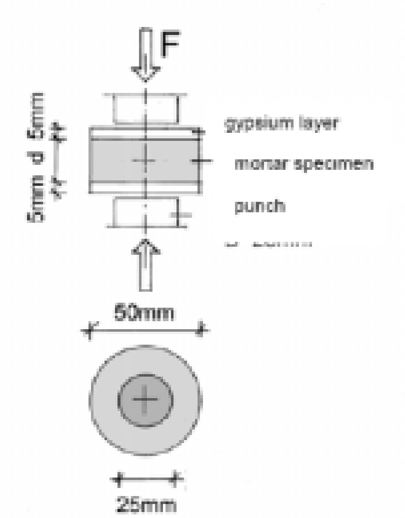


Fig. 3.14: Punching test on existing mortar

i) since friction is developed between the mortar joint and the gypsum layer, a correction factor of 0.7 should be introduced to derive the uniaxial compressive strength;
ii) 30 tests are suggested for statistics to be applied; not less than 10 tests are needed for an acceptable estimate on the material properties;

3.8 DETERMINATION OF SOIL AND BACKFILL MATERIAL PROPERTIES

Backfill is almost certain to be local material that was convenient to the site and may vary considerably in nature both vertically and laterally. The load carrying capacity of an arch bridge can be strongly influenced by the nature of the infill material extending over and beyond the arch and by the stability of the abutment and pier foundations.The most commonly required backfill parameters are self weight and shear strength. The possibility of saturation of the fill by for example flooding should be considered. The soil-arch system response can be significantly altered due to changes in the backfill effective weight and/or detrimental water pressure conditions in the fill. To assess such scenarios the saturated unit weight and permeability of the backfill should be determined. Tests can be performed as per relevant IS codes.

Values of parameters of soil which can be roughly used in an assessment of a masonry arch are given below:

3.8.1 Properties of non-cohesive soil

Type of soil	Density (kN/mm³)		Internal angle of friction (°)	Stiffness modulus E_S (MN/m²)
	Damp soil	Submerged soil		
Loose sand round	18	10	30	20 – 50
Loose sand angular	18	10	32.5	40 – 80
Medium density sand, round	19	11	32.5	50 – 100
Medium density sand, angular	19	11	35	80 – 150
Gravel without sand	16	10	37.5	100 – 200
Natural ballast, angular	18	11	40	150 – 300
Dense sand, angular	19	11	37.5	150 - 250

3.8.2 Properties for cohesive soil

Type of soil	Density (kN/mm³)		Ultimate strength		Initial strength		Stiffness modulus E_S(MN/m²)
	Damp soil	Submerged soil	Internal angle of friction (°)	Cohesion of undrained soil kN/mm³	Internal angle of friction (°)	Cohesion of undrained soil kN/mm³	
Clay, difficult to model, stiff	19	9	25	25	0	50 – 100	5 – 10
Clay, medium stiffness	18	8	20	20	0	25 – 50	2.5 – 5
Clay, easy to model, soft	17	7	17.5	10	0	10 – 25	1 – 2.5
Loam, medium stiffness	21	11	27.5	10	0	50 – 100	5 – 20
Loam, soft	19	9	27.5	-	0	10 – 25	4 – 8
Silts	18	8	27.5	-	0	10 - 50	3 - 10

3.9 ASSESSMENT REVIEW

The conclusion from the assessment should be subjected to a plausibility review. In particular, discrepancies between the results of the structural analysis, indicating inadequacy, and the observed structural condition, for example sign of distress or failure should be explained.

- If there is an arch, which looks very sound under loads, It is likely to have capacity to carry more load (how much ?)
- If an arch had not required excessive maintenance during it’s recent history, is likely to be have more strength than that which requires frequent

maintenance under similar loading conditions; irrespective of theoretical results

- Drainage poses great health hazard for arch. An arch with poor drainage should always be seen with suspicion; irrespective of theoretical results.

Despite use of various assessment methods available, there is no substitute of engineering judgment. All the methods discussed above are only inputs to mind, final decision has to be based on sound engineering judgment.

CHAPTER 4

INSPECTION OF ARCH BRIDGES

4.1 GENERAL RULES

Few general rules, which are not only true for inspection of arch bridges but are true in general, should be followed during inspection.

(a) Read the past observations/inspection reports of the bridge, which will help in understanding the history of damages in the bridge. Monitoring of the damage for a reasonably long period is of great help as it shows whether cause of damage is (i) still active or (ii) exhausted, i.e. took place in some specific moment of bridge life and then stopped (examples can be damages due to foundation settlement due to floods, landslides, earthquakes etc.). Such a historical study could allow cause of damage to be identified and the historical damage which had taken place long time before is not active now is a lesser cause for concern.

(b) Take time to inspect; it is common practice to rush through the inspection and delegating inspection work to lower category staff. Even if no damage is seen in the bridge, look at it for some time, take measurements of shape, take few photographs covering all faces.

(c) Most of the arch bridges were constructed more than 100 years back; it is a common observation that quality of workmanship of construction was exceptionally good in those days. All string course lines should be horizontal, geometry of bridge should be of some specific shape (no deformity) and dimensions should be some whole number in feet and inches. Measure geometry, measure string course levels; if there is any thing abnormal, study it in detail, it may be construction defect but it is most likely to be deformation of structure.

(d) Check verticality of spandrel walls

(e) Take large number of photographs with digital camera covering all faces, soffit, each face of pier and cover all defects. Since only ADEN goes for field inspections normally, other officers can visualize the structure better if photographs are available. Photographs can also be used for comparing condition of structure in successive inspections.

(f) Check for special features like haunching, backing, intermediate spandrel walls, cross spandrel walls etc. if required. This is important only in case of arches where assessment of load carrying capacity is to be carried out. Boroscope can be used for this purpose.

4.1.1 Likely causes of deterioration

Recognising the cause of deterioration is not always direct and straight forward. It is particularly necessary that the inspecting official should avoid a process of pattern matching. A crack, wet patch or dropped stone may look similar to the one seen previously, but the cause may be quite different. Deterioration in most cases is caused by;

(a) Water penetration

Most of the damage that occurs to arch bridges is caused in some way by water. Water washing down through the bridge may erode mortar and saturate porous masonry, dramatically reducing its strength. In regions where deep frosts develop, the water in the masonry may freeze and do untold damage. Stone, brick and mortar softened by saturation becomes susceptible to wind erosion.

Water usually finds its way through the arch at points where it is forced to gather. This might be at the top of solid masonry backing. If drains are not provided, the water will make its own way and do damage at will. Lime runs are common. Severe erosion of joints and masonry units are less, but still frequent. Dropped stones may result where loss of mortar is severe. Dropped bricks can also occur, but the contorted water paths through the ring usually lead to more distributed erosion.

Spandrel walls also suffer water damage and again it may concentrate at the level of solid backing masonry. The walls are more exposed than the arch barrel and the damp wall masonry weathers markedly more.

Highest level at which waterproofing can be applied is at the top of the masonry. Many bridges were built with no effective waterproof layer and no designed drainage. If water is allowed to collect, the damage it can do is considerable.

(b) Mechanical effects

Damage to arches caused solely by loading is very rare. The damage is usually a result of compounding other forms of deterioration. Live load from traffic is inherently dynamic, though it may be possible to treat it as pseudo static. That is to use a magnifying factor to account for dynamic effects. In general, the result of load movement is greater than the effect of a single application of a load in the critical position.

Modest overloads which are not frequently repeated typically result in modest cracks which may heal with time. There is no doubt that bridges respond less well to faster and the more frequently repeated over loads. It is reasonable to assume that there is a threshold below which no damage accrues, but some railway bridges may be at or near that threshold if modern loads are applied at high frequency. The mechanisms of deterioration are not well understood. If a crack is repeatedly opened and closed, it will grow through jacking effects of loosened particles and through steady deterioration of the material.

Impact on arch bridges gets magnified with imperfections in rail or train wheels, therefore track on arch bridges should be maintained in best condition. Joints in track increase impact. As per para 1107(7)(h) of Indian Railway Bridge Manual, joints on bridges, if unavoidable, should be provided at 1/3 of span. This is due to the fact that at this location both bending moment and shear force are less. But above statement is not true for arch bridges; in case of arch bridges, joints if unavoidable should be provided just above the pier for the simple reason that cushion at this place is maximum which absorbs impact and saves the arch.

4.2 STEPS FOR DETERMINING CAUSE OF DAMAGE

If arch bridge has no visible damage and there is no proposal of increasing load carrying capacity of the arch bridge, arch can be certified as good and minor repairs as may be required can be carried out. In case, damage is noticed in arch bridge, it can be studied in following steps.

4.2.1 Look for any obvious cause like severe settlement/ rotation of pier, spreading of arch etc.

4.2.2 Calculate load carrying capacity of the bridge with empirical methods like MEXE method as described in Chapter 3. If load carrying capacity of bridge works out to be adequate with handsome safety margin, then it is O.K. If strength of bridge does not work out to be adequate with handsome safety margin, recalculate the strength using methods like Thrust Line Analysis, RING-2 software as described in Chapter 3.

4.2.3 If load carrying capacity as calculated in para 4.2.2 of the bridge is low, retrofitting methods as described in Chapter 5 can be used after ascertaining cause of damage.

4.2.4 If theoretical load carrying capacity is adequate, but still there is damage, it can be concluded that damage in the bridge is due to failure of material, settlement etc.

4.2.5 To investigate further, take observations of all cracks, note the dimensions, i.e. length, width of cracks in the arch. Prepare 3D diagrams of the arch and mark these cracks on the diagram with exaggerated scale as shown in Fig. 4.2 below. The cracks have been marked in reference to observed cracks as per photo 4.1. Note that crack has been marked in such a way so as to clearly show its width.

By drawing crack pattern at exaggerated scale, it can be clearly visualized that damage on arch vaults and spandrels is due to rotation of abutment against the backfill and formation of three hinges.

Photo 4.1

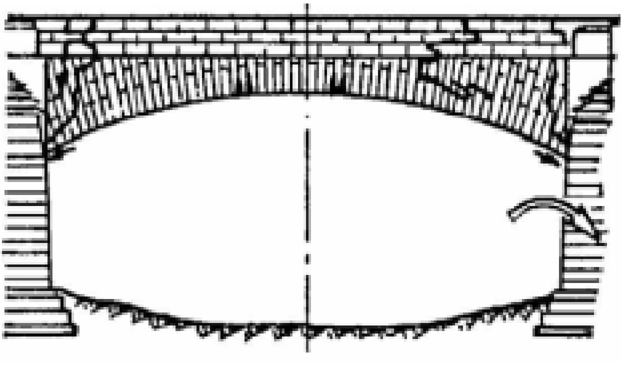


Fig. 4.2

In all cases crack patterns may not be due to a single cause and it is sharp observation and experience of an engineer that is important. All types of such combinations can not be thought of and covered in the book. Some common types of crack patterns normally observed along with likely cause and action required to be taken is given in the following pages.

4.3 DAMAGE CASES

4.3.1 **Local erosion of foundation elements (Photo 4.3) or General undermining of foundation (Photo 4.4)**

Photo 4.3 : Local erosion of foundation

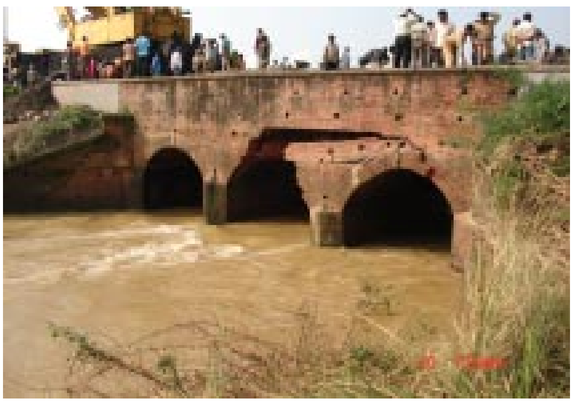


Photo 4.4 : Bridge no. 169 (Span 3 X 10'), Howarh Div., showing general undermining of pier

Cause :

- Increase of hydraulic speed due to decrease of river cross section or changes on the longitudinal profile of the river.
- Disappearance of protection elements.

Structural Importance :

Local erosion does not affect bridge integrity within a short time however it may turn into serious damage in long run. Where as undermining of foundation may affect bridge integrity within a short time.

Observations required :

- Detailed visual inspection of the foundation.
- Determination of the typology and dimensions of the foundation.
- Determination of soil type.
- Determination of the longitudinal profile of the river bed.
- Determination of the cross section of the river bed down and upstream

4.3.2 Mechanical failure of masonry (Photo 4.5)

Photo 4.5 : Mechanical failure of masonry

Usual locations

Localized near to the springings and haunches of the vaults. It may appear in any kind of pattern, when caused by a foundation problem, but it is more usual in very slender ($d/L<1/20$) and shallow vaults ($R/L<1/6$) with bad quality or deteriorated masonry. Where d is thickness of ring, R is rise at centrepoint and L is span.

Cause :

Internal forces are caused by the live loads or indirect actions due to foundation problems. There are three collapse criteria:

- Collapse failure due to masonry collapse under compression.
- Collapse failure due to masonry collapse under interaction between axial bending and shear.
- Collapse failure due to masonry collapse under compression orthogonal to the joints.

Structural Importance :

Such damages affects the strength behavior of the bridge. It announces high risk of collapse.

Observations required :

- Estimation of masonry's mechanical properties
- Estimation of masonry's deterioration level
- Detailed visual inspection of the foundation
- Determination of the typology and dimensions of the foundation
- Determination of soil type and of the river bed
- Determination of the bearing capacity of the vault.

Long term effects :

- Transversal cracking on vault. Three hinges / Mono-arch mechanism / Shear mechanism / Multi-arch mechanism

4.3.3 Longitudinal cracking on Vault (Centre) (Fig. 4.6)

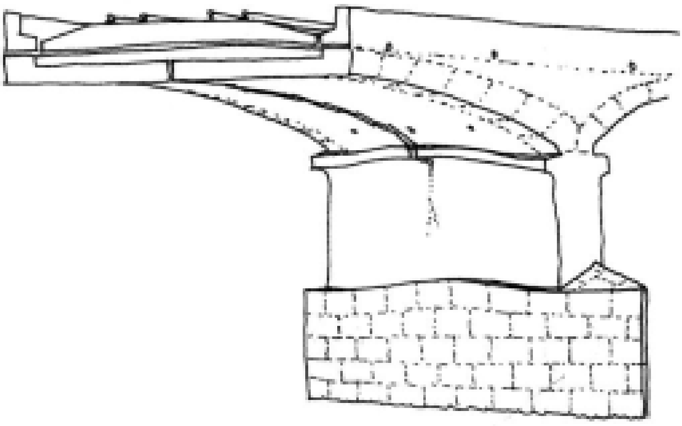


Fig. 4.6: Longitudinal cracking on Vault (Centre)

Usual locations

- Located on central part of the vault. It may appear in all kinds of typologies. However, it is more usual on bridges built for two tracks, and on bridges with pies on the bed of the river where undermining and foundation failures may occur.

Cause :

- Differential settlement of the pier or abutment (due to a stiff point on the centre of the pier)
- Asymmetrical load on the vault (asymmetrical traffic on the bridge)

Structural Importance :

Damage that affects the strength behaviour of the bridge. It affects bridge integrity within a short time if damage is not stabilized. Small longitudinal cracks are not serious and can be stabilised by grouting.

Observations required :

- Detailed visual inspection of the foundation
- Determination of the typology and dimensions of the foundation
- Determination of soil type
- Measurement of the depth of fill over the crown

Long term effects :

- Diagonal cracking on vault / loss or dislocation of pieces
- Mechanical failure of masonry (micro cracking failure)
- Vertical cracking on pier
- Stair cracking on pier
- Vertical cracking on abutments
- Horizontal cracking on abutment

4.3.4 Longitudinal cracking between Ring & Vault (Photo 4.7 and Fig. 4.8)

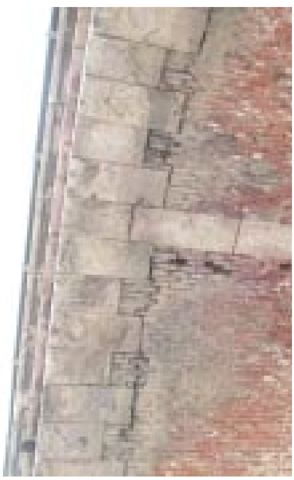


Photo 4.7 : Longitudinal cracking between Ring & Vault

Usual locations

Located on the joint between ring course and vault. It usually appears on deep bridges with saturated backing.

Effect can be aggrevated if the traffic runs near the spandrel.

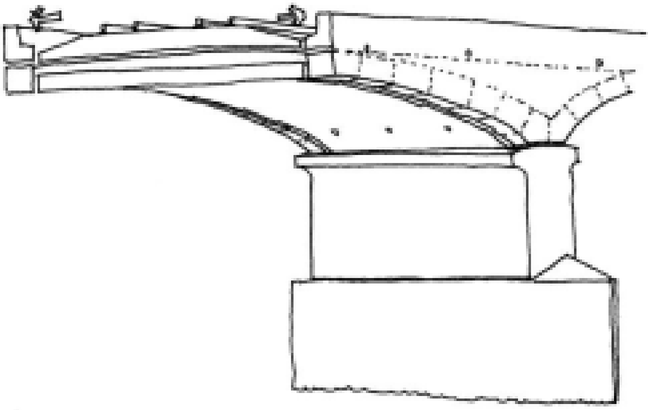


Fig. 4.8 : Longitudinal cracking between Ring & Vault

Cause :

- Horizontal pressure on the backing on spandrels
- Pressure due to the water on the backing (defective drainage)
- Damage of the joint between ring course and spandrels
- Horizontal pressure due to traffic loads near to the spandrels

Structural Importance :

Damage that affects the strength behavior of the bridge. It affects bridge integrity within a short time in case damage is not stabilized.

Observations required :

- On site layout of the track on the structure
- Estimation of effectiveness of the vault drainage
- Estimation of the depth of the backing at crown
- Estimation of the horizontal pressure of the backing on the spandrels

Long term effects :

- Bulging of spandrels
- Sliding of spandrels
- Overturn of the spandrel

4.3.5 Diagonal cracking on Vault (Fig. 4.9)

Usual locations

Located on the central part of the vault, it may appear in all kind of typologies. However, it is more usual on skewed bridges with straight bond, and on bridges with piers on the bed of the river where undermining and foundation failures may occur.

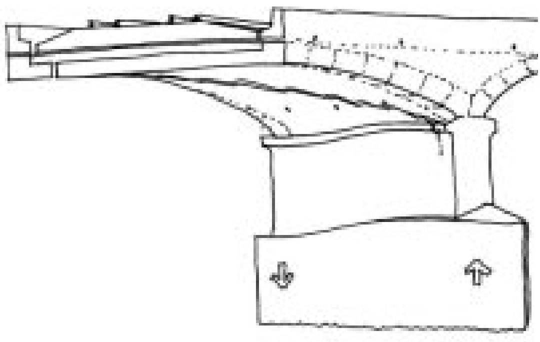


Fig. 4.9 Diagonal cracking on Vault)

Cause :

- Foundation failure of piers and abutments, related with rotation
- Inappropriate bond for the skew and the asymmetric load of the vault (asymmetric traffic)

Structural Importance :

Damage that affects the strength behaviour of the bridge. It affects bridge integrity within a short time in case damage is not stabilized. High risk of failure if other damages due to mechanical failure of masonry appear.

Observations required :

- Detailed visual inspection of the foundation
- Determination of soil type
- On site layout of the track on the structure and sketch or picture

Long term effects :

- Longitudinal cracking on vault
- Mechanical failure of masonry (micro cracking failure)
- Vertical cracking on pier/Stair cracking on pier
- Vertical cracking on abutments
- Horizontal cracking on abutments/Loss and dislocation of pieces

4.3.6 Transversal cracking on Vault, Three-Hinges (Fig. 4.10)

Usual locations

Appearance of open joints and cracks in the intrados near to the springing, as well as open cracks in the extrados near to the crown. It may appear in any kind of typologies. In case of bridges with slender piers (base of pier/span<1/5) and shallow vaults (rise/span<1/6), the cracks will be of large width.

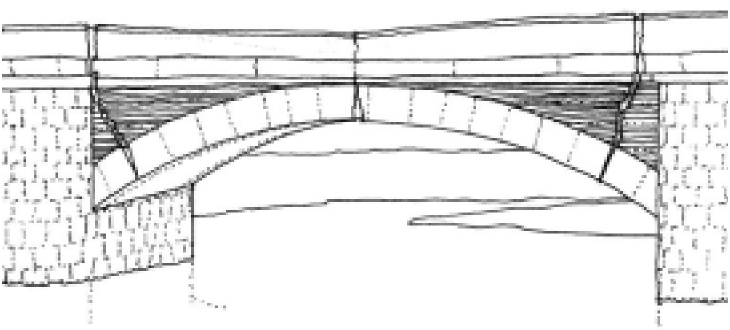


Fig. 4.10 :Transversal cracking on Vault, Three-Hinges

Cause :

- Failure on the foundation of the piers (rotation on piers and abutments) due to undermining problems.
- Failure and overturn of abutment due to horizontal pressure of the vaults

Structural Importance :

Damage that affects the strength behaviour of the bridge. It affects bridge integrity within a short time in case damage is not stabilized.

Observations required :

- Detailed visual inspection of the foundation
- Determination of soil type
- Estimation of the state of the backing at the abutment

Long term effects :

- General undermining/Local undermining of foundation
- Local undermining on abutments/Loss or dislocation of pieces
- Mechanical failure of masonry (micro cracking failure)
- Stair cracking on spandrel/Stair cracking on abutments

4.3.7 Transversal cracking on Vault, Mono-Arch Mechanism (Fig. 4.11)

Usual locations:

Transversal cracking pattern characterized by cracks that open alternatively to intrados and extrados, four times. It may appear in any kind of typologies, but it is more usual on slender (thickness of barrel/span<1/20) and cut down vaults (rise/span<1/6) with bad quality or deteriorated masonry.

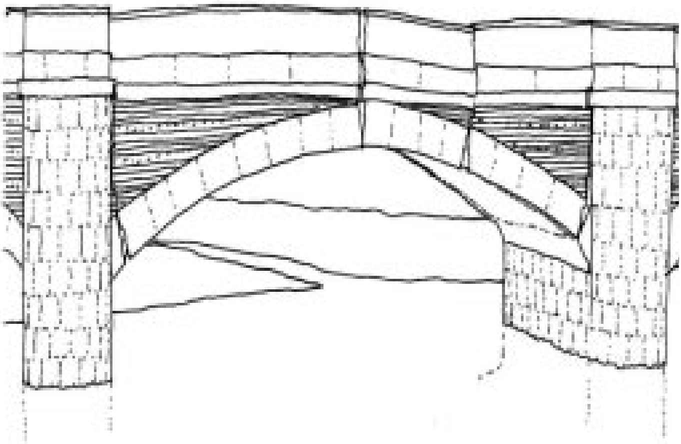


Fig. 4.11 : Transversal cracking on Vault, Mono-Arch Mechanism

Cause :

Failure due to lack of bearing capacity of the vault under application of live load

Structural Importance :

Damage that affects the strength behaviour of the bridge. It announces the imminent collapse of the structure.

Observations required :

- Estimation of masonry´s mechanical properties
- Sketch or picture of the bond used for the vault
- Estimation of masonry´s deterioration level

Long terms effects:

- Mechanical failure of masonry (micro cracking failure)
- Stair cracking on spandrel
- Stair cracking on abutments

4.3.8 Loss or dislocation of pieces (Photo 4.12 and Fig. 4.13)

Usual locations:

Photo 4.12 :Loss or dislocation of pieces

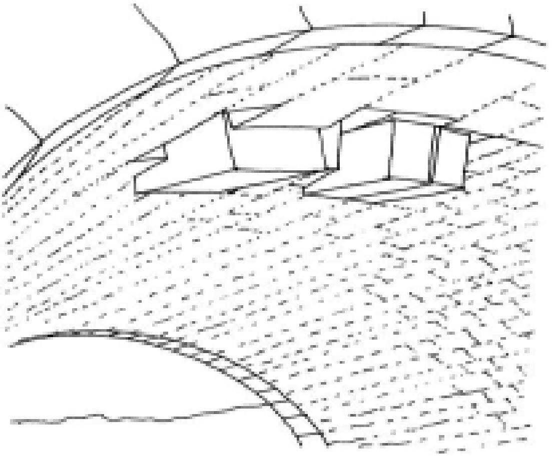


Fig. 4.13 : Loss or dislocation of pieces

It usually appears on the crown of those vaults where the depth of fill is small (< 40 cm). Where track on it has poorly maintained joints. The arch becomes specially vulnerable if drainage is poor.

Cause :

- Impact problems of the live loads near to the crown of the vault, where cushion is less.

- Foundation failure (rotations and settlements on piers and abutments) producing hinges that cause local decompressions.
- Very poor drainage

Structural Importance :

Isolated dislocation of elements can be re-fixed without much long term effect. Dislocation at number of places affects the strength behavior of the bridge. It announces long term collapse of the structure.

Observations required :

- Detailed visual inspection of the foundation
- Estimation of the trains' speed over the bridge and detection of any track defects located near to the crown of the vault.
- Drainage

Long terms effects:

- Transversal cracking on vault. Three-hinges may form
- Stair cracking on spandrel

4.3.9 Vertical cracking on pier (Fig. 4.14) or stair cracking of pier (Fig. 4.15)

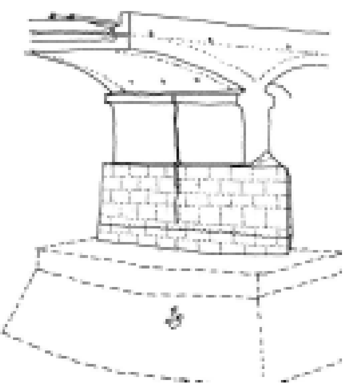


Fig. 4.14 : Vertical cracking on pier

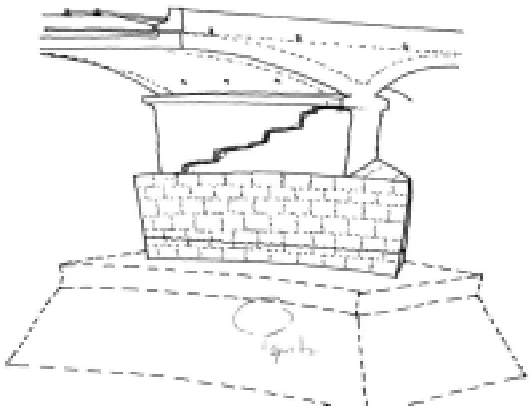


Fig. 4.15 : Stair cracking on pier

Usual locations:

Localized on the central zone of the piers. It is common on bridges with undermining problems that produce differential settlements on the pier.

Cause :

- Local failure on the foundation due to differential settlement between the central part of the pier and its extremes.

Structural Importance :

Damage that affects the strength behavior of the bridge. It affects bridge integrity within a short time in case damage is not stabilized.

Observations required :

- Detailed visual inspection of the foundation
- Determination of the typology and dimensions of the foundation
- Determination of soil type
- Determination of the longitudinal profile of the river bed
- Determination of the cross section of the river bed down and upstream

Long terms effects:

- General undermining/Local undermining of foundation
- Local undermining on abutments
- Longitudinal cracking on vault.
- Diagonal cracking on vault
- Stair cracking on pier
- Vertical cracking on abutment
- Horizontal cracking on abutment

4.3.10 Horizontal cracking on abutments, wing walls and side walls (Fig. 4.16)

Usual locations:

Localized on central part of the abutments. It is common on very shallow bridges (rise/span<1/6) where abutments are not able to resist the horizontal reaction from the vault.

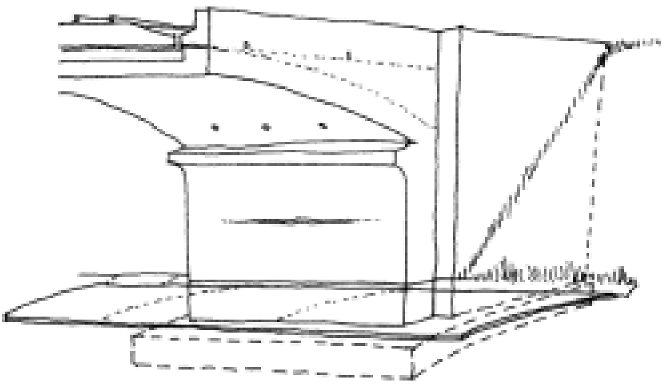


Fig. 4.16 : Horizontal cracking on abutments, wing walls and side walls

Cause :

- Abutment inability to resist bending effects due to the horizontal thrust of the vault against the abutment.

Structural Importance :

Damage that affects the strength behaviour of the bridge. It affects bridge integrity within a short time in case damage is not stabilized.

Observations required :

- Determination of state of the backing at abutment
- Determination of the horizontal thrust from the vault to the abutment for the worst envelope.

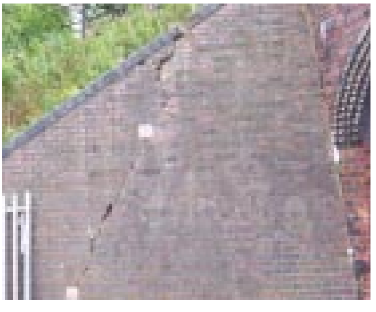
Long terms effects:

- Transversal cracking on vault. Three-hinges may form.

4.3.11 Overturn and bulging of wing walls and side walls (Photo 4.17 and Fig. 4.18)

Usual locations:

Localized on the wing walls and side walls. It may appear on any bridge typology with wing walls of a certain height.



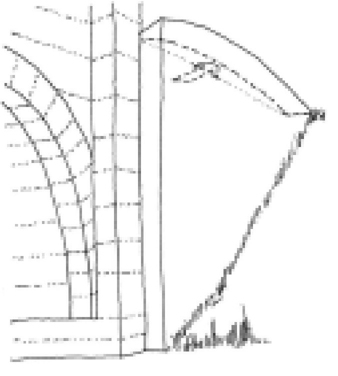


Photo 4.17 and Fig. 4.18
Overturning and bulging of wing walls and side walls

Cause :

- Lack of bearing capacity of wing and side walls to resist the thrust due to the backfill
- Thrust due to the water contained on the backfill caused by a deficient performance of the drainage system
- Growth of vegetation on the wing and sidewalls. Its roots deteriorate the bearing capacity of the masonry

Structural Importance:

Damage that affects the strength behaviour of the bridge. It affects bridge integrity within a short time in case damage is not stabilized.

Observations required:

- Determination of the constitution and state of the backfill at the abutment, wing and side wall
- Determination of the performance state of the drainage element at the abutment
- Bond between wing or side wall and abutment

Long terms effects:

- Horizontal cracking on abutments
- Vertical cracking on abutments

4.3.12 Bulging of spandrels (Fig. 4.19(a)) or sliding of spandrel (Fig. 4.19(b)). Photo 4.20 (a) and (b) showing collapse of spandrels

Usual locations:

Localized on the spandrels over the pier, where the spandrel reaches its greatest height. It is common on bridges with deep and not very wide vaults.

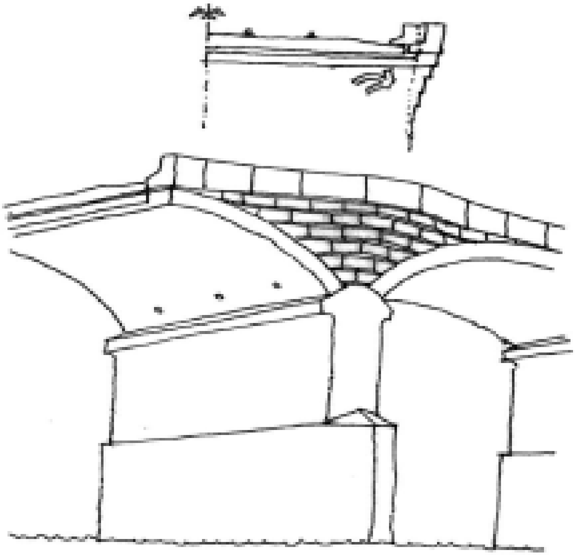


Fig. 4.19(a) : Bulging of spandrels

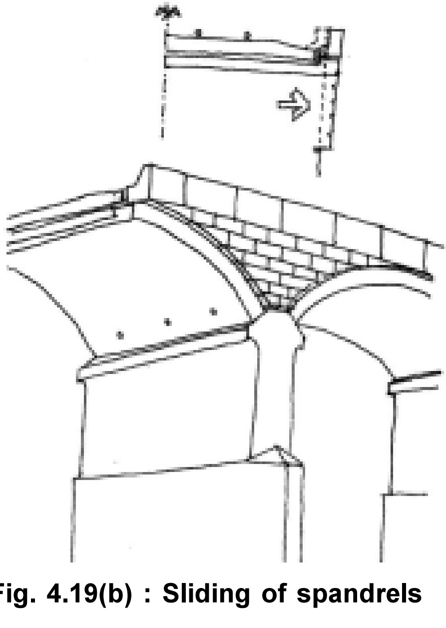


Fig. 4.19(b) : Sliding of spandrels

Photo 4.20 (a) and (b) of Bridge no. 129 (Span 45 x 15'), Howarh Division, Showing failure of Spandrel wall

Cause :

- Thrust from the backfill to the spandrel
- Pressure due to the water on the backing (defective drainage)
- Damage to the joint between arch ring course and spandrels
- Horizontal pressure due to traffic loads near to the spandrels
- All these effects cause the bending of the spandrel working as a cantilever beam of a variable height

Structural Importance :

Damage that affects the strength behaviour of the bridge. It affects bridge integrity within a short time in case damage is not stabilized.

Observations required :

- On site layout of the track
- Determination of the state of drainage elements at the vault
- Bond used on the spandrel and its bond with the vault
- Determination of the depth fill over the crown

Long terms effects:

- Longitudinal cracking between ring course and vault
- Sliding of spandrels
- Overturn of spandrels

4.3.13 Efflorescence

Usual locations:

Generally located along the intrados of the vault and on the front elevation of the piers. They usually concentrate around elements with the highest level of humidity (nearby the drainage pipes at spandrels and intrados of vaults, etc.).

Cause :

These are the manifestations of salt crystallization. This phenomenon is unleashed when soluble salts, present in dissolution in the porous system of the masonry, crystallize. If the evaporation occurs on the surface, efflorescence is generated. However, if the evaporation occurs before the salt reaches the surface, crypto efflorescence is formed. The presence of efflorescence and crypto efflorescence warn that, on the one hand, a chemical degradation process is happening (not very dangerous generally) and on the other hand, important internal mechanical stresses can be generated due to salt crystallization that can damage the material seriously.

Structural Importance :

Damage that affects durability. It does not affect bridge integrity within a short time. However, it may lead to serious long term damage.

Observations required :

- Detection of possible waterways (drainage, capillarity, etc.)
- Detection of possible salt sources (coming from the masonry itself or from external agents or pollutants, or from materials from previous interventions, etc.)

Long terms effects:

- Stains
- Crusts and superficial sediments
- Alveolus or vesiculation

4.3.14 Failures due to vegetation (Photo 4.21)

Cause :

Mechanical deterioration due to action of the roots and vegetation on the masonry.

Structural Importance :

Damage that affects durability. Therefore, it does not affect bridge integrity within a short time. If it is of great extension and of advanced scope (the roots cause the breaking of pieces and their loss) it must be repaired.

Photo 4.21

Long terms effects:

May damage bridge in long term

4.3.15 Previous interventions (Photo 4.22)

Cause :

Interventions made due to the upgrading of the railway (electrification, overexploitation, etc.) or due to need of repair or enlargement, where incompatible or aggressive materials than the previous ones have been used. Sometimes the bearing behavior of the bridge have been altered without studying its consequences.

Structural Importance :

These damages may affect durability or the strength behavior of the bridge depending on their importance on the structural safety.

Photo 4.22

Observations required :

Sketch of the damage location.

4.3.16 Impacts

Usual locations:

Generally located on the crown of the vault or piers of structures with traffic underneath, and therefore exposed to impacts of vehicles and accidents.

Cause :

Scratches and abrasions caused by the impact of vehicles on masonry arch bridges, normally due to accidents where clearance is insufficient.

Structural Importance :

Damage that affects durability. It does not affect bridge integrity within a short time, except in case that the impact affects an important structural element severly.

Observations required :

- Research on vault clearance and width of the road.
- Research of the traffic underneath the structure, speed and type.

4.3.17 Drainage Problems (Fig. 4.23 and Photo 4.24)

Usual locations:

Through vault or spandrel walls.

Cause :

- Poor drainage through ballast
- Side holes in spandrel wall blocked.

Structural Importance :

It affects durability in long term. Causes loss of mortar.

Observations required:

- Drainage of ballast
- Clearing drainage holes in spandrel wall
- Clearing weep holes in abutments

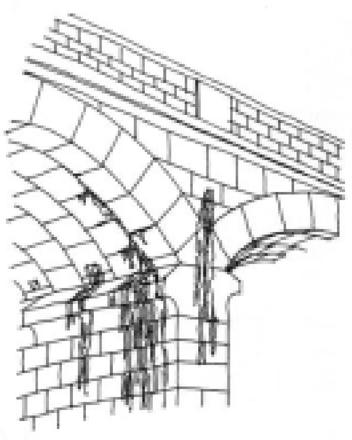


Fig. 4.23

Photo 4.24

Long terms effects:

- Leaching of mortar
- Erosion of stones

4.4 GUIDELINES TO JUDGE LOSS OF STRENGTH OF ARCH DUE TO CRACKS

What effect a crack will have on strength of a arch or for that matter on any other structure is hard to define. It is primarily the experience and objectivity of inspecting official, which can lead to some conclusions. In case of masnory arch bridges, some guidelines are available but these should be used only for understanding relative importance of various cracks rather than actually judging percentage loss of strength of a arch bridge.

	Type of crack	% loss
a)	Longitudinal cracks:	
i)	Outside the centre third of the arch, less than one tenth of the span in length	5-10
ii)	Outside the centre third of the arch, longer than one tenth of the span in length	10-15
iii)	Within the centre third of the arch, less than one tenth of the span in length	10-15
iv)	Within the centre third of the arch longer than one tenth of the span in length	15-20
c)	Lateral and diagonal cracks:	
i)	Up to three small lateral or diagonal cracks less than 3 mm in width and less than one tenth of the arch width	10-20
ii)	Numerous small cracks as above in the centre third of the arch	40-60

In a arch there may be more than one type of cracks, inspecting official should use his judgement and experience.

4.5 CONCLUDING REMARKS

Objective of an inspection is to assess condition of existing arch, defects in the arch and their effect. A fruitful inspection should result in:

a) Whether any repairs/retrofitting is required or not?

b) If repairs/retrofitting is necessary, what type?

c) How required type of repairs/retrofitting is to be carried out? (See chapter 5 for details)

CHAPTER 5

REPAIR, RETROFITTING AND DISMANTLING

Whenever strength of an arch is to be assessed, a through investigation of its features and their effect on load carrying capacity needs to be studied. Arches in most cases will have much more strength than what is normally assumed. Whenever strengthening becomes necessary, damage on the bridge and its causes need to be identified along with the effect on the bridge safety. If no cause is recognized, and the retrofitting works are not adequately and very carefully evaluated, the strengthening works may result in unjustified expensive repairs with reduced effect on the safety of the arch.

Once the cause of damage has been identified, the choice of the repair or retrofitting works is the second crucial step for repairing a bridge. The two steps are not necessarily connected one to the other: if damage did not reduce significantly the safety factor of the structure towards the service loads, then the second step is not necessary. Strengthening is needed when damages are so severe that the safety margin remains too low even after repairing of the damages. Assuming that damage means necessarily that the bridge must be strengthened leads to unjustified retrofitting works, economic costs that may be extraordinary high while the bridge is not necessarily strengthened at the end of the works. The common approach to retrofitting and strengthening of a masonry bridge assumes that any work will somehow enhance the bridge safety, which is obviously not true.

The repair and retrofitting can be broadly divided into following groups

1. Basic Repairs

- o Salt removal, vegetation removal, cleaning of arch

2. Reconstruction of lost of damaged pieces
    - o Reconstruction of elements
    - o Replacement of ellements
3. Pointing
4. Grouting
5. Concrete saddling
6. Near surface reinforcement
7. Under ringing
8. Shotcreting
9. Sustaining walls
10. Transverse ties

Items 1 to 4 above can be called repair and from 4 to 10 retrofitting.

Details of each of above techniques and situations where these can be used is described in following paragraphs.

5.1 SALT REMOVAL, VEGETATION REMOVAL AND CLEANING OF ARCH

Soluble salts are one of the important causes of deterioration of railway masonry bridges. The position of its crystals inside the porous of mortars and pieces contributes to its cracking. The process progresses from the evaporation surface (exterior), towards the interior. The repair work is carried out with the application of paper pulp dressing or any other absorbent material with clay. This paste is soaked with distilled water and it is left, at least 48 hours, on the surface, so as to let the salts be dissolved and pass through. Content of salts falls asymptotically, with every new extraction, and the most efficient are the first ones. Normally, one or two extractions may be required.

Similarly any vegetation on the structure should be carefully removed along with roots. Cleaning should also be done at appropriate intervals, it can be manual or with water jet. Chemicals such as $CuSO_4$ may also be required to be used for certain type of deposits. More details can be seen from cleaning manual of UIC (refer para 1.5(9) in chapter 1)

5.2 RECONSTRUCTION OF ELEMENTS

These techniques allow the recovering of the shape and outlines that masonry elements originally had. Problem is generally more in brick masonry than on stone masonry. The reconstruction will be done when the losses and missing section are less than 5 cm. in brick masonry (7-10 cm. in stone masonry) as general criteria. When bigger than that, the action shall be as per para 5.3. Photo 5.1 shows brick masonry with lost mass of all elements.

Photo 5.1: Showing losses in brick elements

The properties, stiffness and resistance of the mortar to be used for reconstruction depend on the material compatibility with the base material. The re-composition must preserve the properties that the brick had before. Primarily porosity/permeability of original material should match with reconstruction material. Stones used on masonry structures include a large range of granites and sand stones, their porosities vary from 0.5% in granites with fine grain, reaching 11% for some sand stones.

Lime is suitable as depending on the dosage lime/grains/water, it is possible to obtain different properties of the reconstruction mortar. Lime mortars are the most elastic and do not add soluble risky components. It has similar features that of masonry materials (like lime stones, sand stones and bricks). Portland cement types are not suitable, because they tend to release salt and are also not permeable enough.

Before application of mortar, drill to form case (properly shaped depression), clean, eliminate the parts that came off the masonry and clean the dust in the surfaces by means of high pressure air steams. Once surface is clean, moisten it and then apply re-composition mortar on the missing area by a manual spread of the mortar.

5.3 REPLACEMENT OF PIECES

This technique is used when the loss of section in the pieces is more than 5 cm. in brick masonry (7-10 cm. in stone masonry) as general criteria since it is better and efficient to substitute the element rather than its reconstruction. Photo 5.2 show a brick masonry where more than 5 cm. depths have been lost.

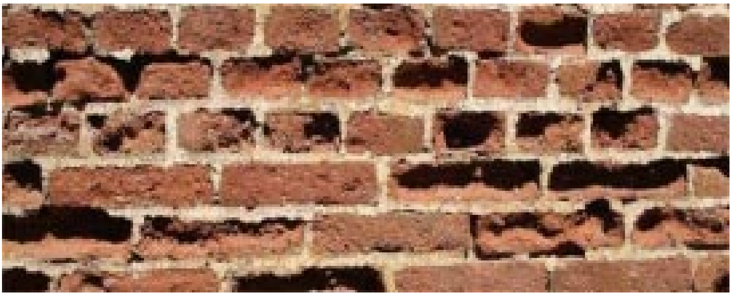


Photo 5.2: Heavy loss of mass of bricks

Firstly a case forming must be executed by drilling the affected elements, removing the parts that come off from the masonry and afterwards cleaning the surface of dust by means of pressured air steams. It is very important to eliminate rest of the mortar in the sides of the case formed.

Suitable bricks/masonry should be selected which is compatible (mechanical and hydraulic properties) with original and dust etc. should be removed by washing. Bricks/masonry pieces are placed in their final position and pointing is done with lime mortar.

5.4 POINTING

Environmental degradation or/and water leakage often makes the mortar joints to be severely damaged. Under these conditions, the degraded mortar of the joints reduces the actual dimension of the load-bearing brickwork. For example, a 50cm thick arch and mortar joints that are degraded till 4cm in depth at intrados and 3 cm in depth at extrados; the actual thickness of the arch (residual arch) is reduced to 43cm. Since the load carrying capacity of an arch-type structure depends approximately linearly on the arch thickness, we can roughly estimate the load reduction as 7/50 = 14% Thickness reduction is not constant throughout the arch, but is more pronounced in the parts of the arch where water leakage is more severe.

Repointing consists of: i) cleaning of the mortar joints from the degraded mortar; ii) re-filling of the joints (exactly: re-pointing) with compatible mortar. The joint should be clean and wet; if the joint is dry, it should be wetted. A wrong choice of re-pointing mortar, such as cement-based mortars (cement based mortars produce thaumasite if sulphates are present in the original brickwork), may increase the degradation speed. Cement mortar also seals the joints which in turns forces the percolating water through the masonry blocks causing it's deterioration. For the same reasons use of epoxy is dangerous. In case it is not possible to use lime, some exception can be made in by using weak cement mortar which is adequately porous. Shrinkage of re-pointing mortar can vanish part of the re-pointing work therefore mortar should be made non-shrink by additives.

5.5 GROUTING

Procedure of grouting is explained in detail in para 209 of Indian Railways Bridge Manual.

IR Bridge Manual allows use of cement mortar or epoxy grouting for arches, however during interaction with experts UIC group, it was again and again emphasized that old arch bridges should not be grouted with cement mortar as they seal the joints which in turns forces the percolating water through the masonry blocks causing it's deterioration. Secondly it changes the stiffness of arch barrel at certain locations where grout percolates and load distribution gets affected. These more stiff areas may attract more force and may collapse. For similar reasons, grouting with epoxy must be prohibited.

Draft leaflet of UIC on arch bridges states **"Repairs, whether in reconstruction, pointing, or for surface repairs, should use lime based mortars. Lime mortar with properties similar to the original construction allows the masonry to breathe and for water to pass through it rather than the masonry. Portland cement based mortars should not be used unless the structure was originally constructed with this material as further damage can be caused."**

If at all use of cement cannot be avoided, grouting/ pointing may be done with weak (1:3, 1:4) cement sand mortar. Epoxy grouting shall never be done.

5.6 CONCRETE SADDLING

The concrete saddle on top of arch is difficult to provide under normal working conditions in Railway. However procedure is explained below for use in places where line can be blocked/traffic can be diverted for sufficient period. It can be done in four different situations:

- The fill is completely substituted by a block of concrete, normally un-reinforced (Fig. 5.3), with an horizontal extrados of the concrete saddle; concrete is connected to the arch extrados by means of a rough surface only or/also by means of mechanical connectors. Such a technique is usually applied to very shallow arches (rise to span ratio < 0.2) in order to limit the thickness of the concrete block.

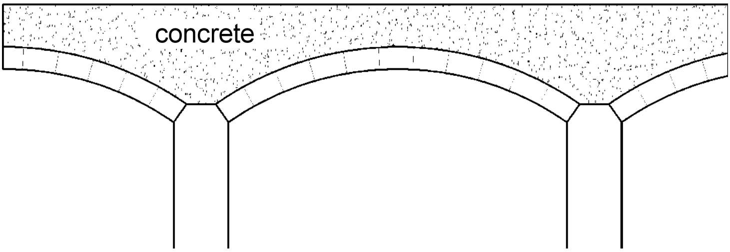


Fig. 5.3

- Over-ringing is done by a new reinforced arch on the extrados of the original masonry arch (Photo 5.4). Typically this is suitable for deep arches (rise to span ratio > 0.2 and < 0.5), the r.c. reinforcement is usually (but not necessarily) of constant thickness and is connected to the masonry arch either by means of a rough surface only or more frequently by means of mechanical connectors also.

- For long span arches and/or when the haunching is relatively limited on the top of the piers, the concrete saddle may be limited to the top of the piers (Fig. 5.5)

- A continuous r.c. concrete slab directly on the fill can be provided as a saddle. This kind of saddle does not directly act on the arch but rather restrains the vertical displacement of the fill. In fact, surface concrete saddles (or concrete slabs) over the fill increase the fill resistance by postponing the shear failure of the fill material. Besides, the high stiffness of the r.c. slab in comparison with the original fill surface improves the load distribution in the fill. For this reason, this technique should be better addressed as "retaining slab" rather than concrete saddle.

Photo 5.4 Concrete saddling

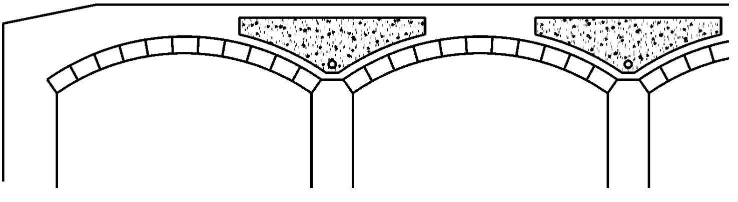


Fig. 5.5 Showing saddling over pier only

Related issues

Effects of concrete saddles, under the service loads is that of increasing the arch thickness. If the arch thickness is increased, its load carrying capacity would be enhanced; besides, the stress state in the masonry arch would be relieved by the new c.c./r.c. arch. Apart from the dead loads of the masonry arch, that are supported by the old arch only, the weight of the fill and the live loads would be sustained by a mixed masonry/r.c. arch in which the main role is played by the new arch due to the higher stiffness of the r.c. arch.

Effects of concrete saddles, under the ultimate load are that arch behavior at collapse is rather different. The over ringing r.c. arch, being able of sustaining tensile forces and being connected to the old arch by a large rough surface, behaves as a hinge-locking device, preventing one side of the hinge from opening. In this way collapse is caused by compressive crushing in some sections but a four hinge mechanism is no longer active. The retrofitted bridge is expected to exhibit highly increased natural frequencies; this may increase the dynamic effects on the bridge.

Steps for execution include fill removal, cleaning of the arch extrados, making connection between old and new works, placement of reinforcement, connection to springing (the concrete saddle needs to be adequately connected to the springing of the original bridge), casting, water proofing and refilling.

5.7 NEAR SURFACE REINFORCEMENT

The collapse mechanism of an arch assumes, in the classical approach, four hinges to be needed for a single span arch to collapse, opening alternatively at the extrados and at the intrados (para 2.3). Bearing in mind this approach, the main idea is that providing tensile resistant reinforcement to the arch, either at the extrados or at the intrados, locks the activation of such a mechanism (Photo 5.6). Few firms like “Helifix” are doing this type of work. Steps for execution of works are;

- **Brickwork grouting:** The bridge brickwork should be grouted (if needed) and the mortar joints re-pointed before any other work is performed. Then rebates on arch intrados are made to place reinforcement. This work needs to be performed on brickwork in the best possible conditions.

- **Transverse reinforcement:** Transverse reinforcement is placed on the arch before longitudinal reinforcement. Careful attention in filling the rebates is required, looking at the best possible connection between reinforcing bars and arch.

- **Longitudinal reinforcement**: Longitudinal bars are placed on the arch intrados and the rebates are all filled with mortar of resigns.

- **Abutment and spandrel reinforcement**: The bridge retrofitting and strengthening is completed by insertion of steel bars in the mortar joints of abutments and spandrels.

Photo 5.6 Near surface reinforcement

Related issues

(i) Some objections to the efficiency of this technique arise from some experimental data

ii) Connection of the bar to the arch is provided by the resins and/or by mechanical connections to the arch. Recent experimental research shows that the bond provided by the epoxy resins to the bars and to the brickwork is enough for preventing the bars from detaching.

iii) Re-filling of the rebates should be done very carefully since the presence of voids could activate corrosion of the reinforcement and degradation of the resin.

iv) The technique does not require long time for execution since the epoxy resins reach reasonable performances within 24 hours.

v) No dead weight is added to the arch.

vi) The effect of this technique is to be expected mainly at collapse.

5.8 UNDER-RINGING

A quick and relatively cheap strengthening technique is known as “under-ringing” and consists of steel (Photo 5.7) or reinforced concrete (Photo 5.8) located below the masonry arch or below the parts of the arch that are believed to be seriously damaged.

In this strengthening technique, the steel/concrete arch is placed below the masonry arch, which already sustains the dead loads. Therefore, the new arches are effective only with respect to the live loads. Since both the original masonry arch and the new one are active when the trains pass on the bridge, the live loads are supported by both the arches. This is a quick and widely used method. It should be ensured that there is no gap between old and new addition. Under ringing can be done in complete barrel or partly. Problem arises in case of concrete under ringing if there is problem of water leakage in original arch. Water leakage problem is original arch should be addressed before going in for concrete under ringing. Water leakage problem can be tolerated in steel under ringing. Also grouting should be done if there is any sign of hollowness in original arch.

Photo 5.7 showing under ringing with steel

Photo 5.8 showing under ringing with concrete

5.8.1 Issues related with steel under ringing

Usually under-rings are connected to the original piers of the bridge, since new supports would make under-rings much more expensive. The supports may be: i) directly built in the original piers, with small steel cantilevers used as an additional device distributing the load on the pier or ii) connected to the original pier by means of strong steel structures. The latter solution is preferable since it does not need holes to be perforated in the original pier. The efficiency of the supports is crucial for the effectiveness of under-ringing.

The second, and probably the main crucial issue for under-ringing is the connection of the new steel ring with the masonry structure. If the material filling the gap in-between the two arches is deformable or the gap is not properly filled, the two arches do not cooperate and the original masonry arch remains the only load bearing structure.

5.8.2 Issues related with RCC under-ringing

An alternative to discontinuous steel under-ringing, is continuous under-ringing by means of r.c. arches. Problem in this method is proper filling of gap between new r.c. and old masonry. Use of shotcreting may overcome the difficulty in making a perfect and stiff connection between the two arches. The continuous under-ringing is not suitable if there is water leakage problem in original arch.

r.c. under ringing can be designed either for additional loads or for the complete load. If r.c. under ringing is designed for complete load, old arch is treated as non existant. In this design filling of gap between old and new elements is not important. This alternative is being used widely in Indian Railways.

5.9 SHOTCRETING

Shotcreting consists of spraying a layer of concrete (shotcrete) on to the intrados of the arch (Photo 5.9). *Shotcrete* is a mixture of cement, aggregate and water projected pneumatically at high velocity onto the surface. It undergoes placement and compaction at the same time due to the force with which it is projected from the nozzle. It can be impacted onto any type or shape of surface, including vertical or overhead areas. This solution undoubtedly improves the load carrying capacity of the bridge but, however, a number of questions arise:

- What is the optimal thickness for the composite ring of concrete and masonry?
- What is the most effective form of detail at the springings so as to ensure the transmission of horizontal thrust to the abutments?,
- How to provide proper bond between the shotcrete lining and the masonry barrel?
- How shrinkage of shotcrete affects composite behaviour?

Two systems of strengthening can be adopted with regard to the thickness and rigidity of lining: i) rigid shotcrete lining; ii) flexible shotcrete lining.

5.9.1 Rigid shotcrete lining

A 200-300mm thick shotcrete layer is sprayed on the intrados of the arch usually with double reinforcement. The connection between the existing arch and the shotcrete layer is provided by steel bolts. This solution intends to substitute the load-carrying system of the existing arch and does not rely on existing structural capacity. As the interaction of the rigid shotcrete lining and the existing arch cannot be effectively solved the new lining requires a new foundation or needs to be properly connected to the existing foundation.

5.9.2 Flexible shotcrete lining

The method uses a thin (30-50mm) shotcrete layer with engineered properties. The shotcrete material is often combined with high ductility welded steel mesh reinforcement. The method is applied when the arch surface only is seriously degraded and the arch itself needs

Photo 5.9 Showing reinforcement on top and shotcrete operation at bottom

stabilization and only a moderate increase in load carrying capacity. Interaction between the existing arch and the shotcrete layer is provided by stainless steel stitching and the interfacial bond between the new lining and the existing surface.

One of the crucial issues of shotcreting, whatever flexible or rigid, is related to draining of water: the shotcrete layer is substantially waterproof. Waterproofing or proper drainage should be provided or restored before shotcreting the arch. Drainage problems in original arch can cause durability problems (Photo 5.10).

Method of shortcreting has been explained in para 209 (3) of Indian Railways Bridge Manual, however the details given are not sufficient. For details of method of execution and quality control, reference be made to IS 9012.

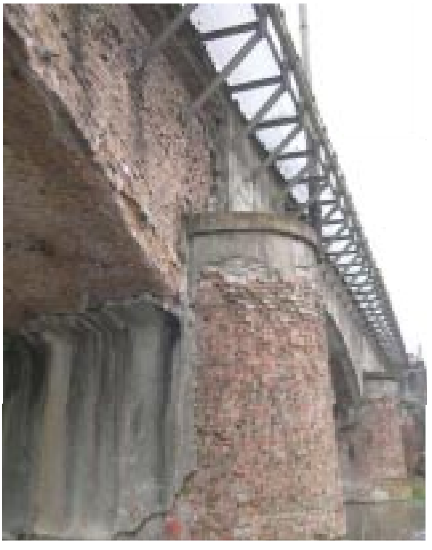


Photo 5.10 Damaged shotcrete due to water seepage through masonry

5.10 SUSTAINING WALLS

In some specific case, sustaining walls can be built below the arch barrel. This technique, which is simple and undoubtedly efficient, can be used only in cases where water way or flow of traffic are not the issues. Photo 5.11 shows such a technique used for increasing the stiffness of a span in the specific case. This technique can be adopted temporarily in damaged arches till proper solution is found.

Photo 5.11 Sustaining Wall

5.11 TRANSVERSE TIES

Transverse ties have been used to join the barrel which has longitudinal cracks. Use of transverse ties affects architectural looks. Fig. 5.12 shows schematic diagram of this technique. Ties can be internal or external as shown in Photo 5.13.

Based on same principal, ties can be provided from one side of spandrel wall to the other for holding them together and not giving way under lateral loads.

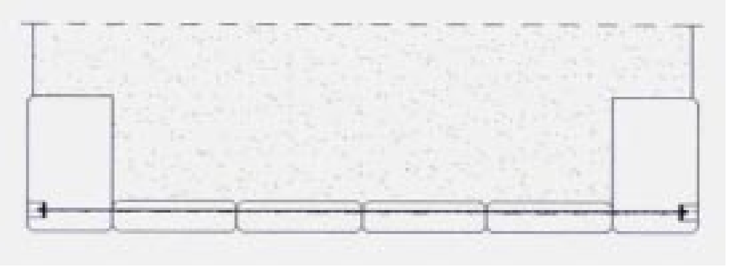


Fig. 5.12

Photo 5.13 Showing internal and external ties

5.12 CLOSING REMARKS FOR RETROFITTING

The techniques discussed above, if applied, sometimes produce irreversible changes in behavior of arch. Design of some of techniques can be done by 3D FEM non linear models. The theoretical (numerical) evaluation of their effectiveness is very uncertain since 3D FEM non linear models, both the damages and their causes need to be modeled. The complexity of the resulting FEM model makes the results not easy to be understood and too dependant on the assumed constants. For these reasons, retrofitting of a masonry bridge should always be supported by experimental verification of the works.

5.13 DISMANTLING OF ARCH BRIDGES

Dismantling of an arch bridges is as typical as their construction. On 02/12/06, bridge no. 153, 3X30 feet arch was being dismantled at Bhagalpur Railway station. It collapsed and fell on Howrah- Jamalpur exp. killing 36 passengers and grievously injuring 12. Some portion of the same bridge, but other span, fell on 30/11/06 before this accident. A photograph of bridge one day before collapse is shown in Photo 5.14 and a photo after catastrophe is shown in Photo 5.15. Photo 5.14 clearly shows thin pier supporting huge structure and bearing horizontal thrust.

Similar accident had taken place at Tundla station of Northern Railway few years back. Both accidents took place because force flow in arch was not understood or appreciated by field staff.

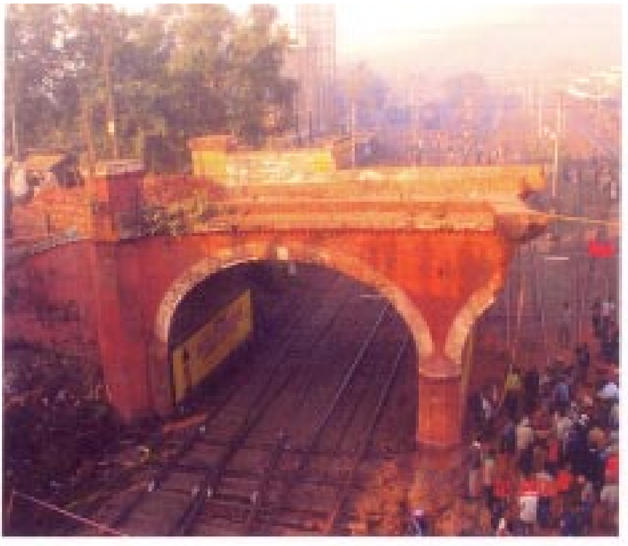


Photo 5.14 Showing partially dismantled arch prior to collapse on 01-12-2006

Arch is a structure, which transmits heavy horizontal thrust to abutments and piers. In case of abutments, this load is resisted by heavy section of abutment and soil fill behind it. At piers, in case of multi span arches, horizontal thrust due to dead load is balanced. If both spans are loaded, horizontal thrust due to live load also gets balanced, but, in case of only single span being loaded, pier has to bear some unbalanced horizontal thrust. Piers are, therefore, designed to take up only unbalanced horizontal thrust which is quite less as compared to the total thrust at abutment.

Photo 5.15 Showing collapsed arch on train

Whenever in multi span arches, if one span is dismantled, large unbalanced horizontal thrust comes on pier and there can be collapse of pier along with other spans. Following procedure can be followed to safely dismantle arch bridges, these methods are suitable for both single and multi span arches.

5.13.1 Dismantling with explosives

Explosives can be used to bring down all spans of an arch bridge at one go. This will require cordoning off the area likely to be affected by the explosion and long time to remove the debris thereafter. This method can only be used if the arch is not near habited area and experts can be engaged to take up such work.

5.13.2 Dismantling with machinery

Special type of machinery with long jib can be used to dismantle one span of arch in one go. As unbalanced horizontal thrust may cause collapse of all or few other spans of the bridge, whole work should be planned in a single block and all the spans should be dismantled in one block. It must be ensured that work is completed in the block and no portion of the arch is left without dismantling in the block. This procedure will require cordoning off the whole area and engaging suitable machinery.

5.13.3 Part-by-part dismantling

The above two methods though, are safe, may not be possible under many circumstances. In part-by-part dismantling method, dismantling is done in such a systematic manner that at no point, there is excessive unbalanced horizontal thrust on piers.

Step by step procedure shall be as under in reference to figure 5.15 (a) to (c)

1. Divide the depth of soil into two part i.e. **Part 'A'** from top of soil to the depth up to the level of Crown of arch. **Part 'B'** is from Crown level to the top of Abutment / Pier. As shown in Fig. (a).

2. Divide the width of Bridge into equal parts each about 50cm wide for the length of each span as shown in Fig. (b). (Fig (b) shows bridge divided into seven parts, it will be more for wider bridges). No. of division should be odd number.
3. Engage four parties to remove soil. First party will start removing soil from the **Section 'A1'**. It means start removing soil in the **section- 1** from top level and depth upto the level of crown of arch i.e. **Part 'A'** as shown in the sketch. Second party will simultaneously remove the soil from **Section– 2**, **Part 'A'** i.e **A2**. It means soil from top level to the depth up to the crown. Third and four parties shall work in section A3 & A4.

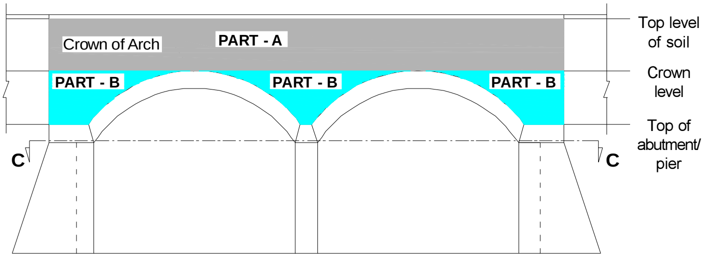


Fig.5.15 (a) Sectional Elevation

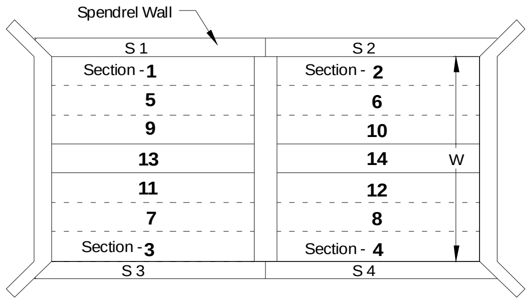


Fig.5.15 (b) Plan

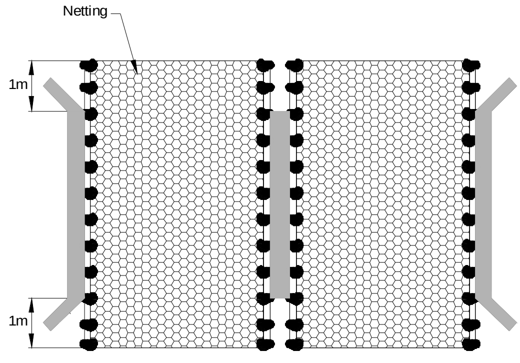


Fig.5.15 (c) Section C C

4. After completing A1, A2, A3 & A4, follow the sequence Section – A5, A6, A7, A8, and then A9, A10, A11 & A12 and then A13 & A14. After this exercise Section A is cleared. This procedure ensures that there are no unbalanced lateral forces.

5. Similarly follow the same sequence for removing soil of Section – B.

6. Provide thick nylon netting supported on piers so as to arrest any falling debris as shown in fig (c)

7. Now each of four parties should break spandrel wall S1, S2, S3 & S4 simultaneously under block, as some debris can fall on track.

8. After breaking spandrel wall, arch barrel of section 1, 2, 3 & 4 shall be broken under block protection by each of four parties. In next block, section 5, 6, 7 & 8 shall be broken and so on.

9. At the end, last middle section 13 and 14 will remain (since arch has been divided into odd numbers of parts), which should be dismantled by pulling it down with the help of ropes or some long jib machinery. While dismantling last section, no person should be on top of the arch.

10. Afterwards piers can be dismantled in systematic manner from top to bottom

In case of 3 span arch, no. of parties required shall be 6, in case of 4 span arch, no. of parties required shall be 8 and so on.

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