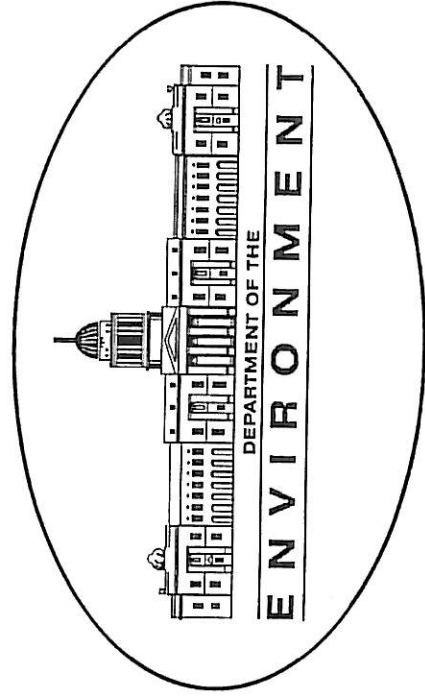


# REPORT ON INSPECTION, ASSESSMENT AND REHABILITATION OF MASONRY ARCH BRIDGES



**REPORT ON  
INSPECTION,  
ASSESSMENT AND  
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OF MASONRY ARCH  
BRIDGES**



# BACKGROUND

Council Directive 85/3/EEC of 19th December, 1984 fixed the maximum weights and dimensions of 5 and 6 axle combined vehicles circulating internationally, including a gross vehicle weight limit of 40t and a triaxle limit of 24t. Ireland and the UK have a derogation to retain national limits of 38t and 22.5t. National regulations were made in 1986 implementing Directive 85/3/EEC - apart from the EC limits from which Ireland has a derogation.

Council Directive 86/360/EEC of 24th July, 1986 fixed a drive axle limit of 11.5t for 5 and 6 axle combined vehicles circulating internationally by 1st January, 1992. Ireland the UK have a derogation to retain a national limit of 10.5t.

The EC Council of Transport Ministers will determine the duration of Ireland's derogations from EC lorry weight limits - on the basis of proposals made by the EC Commission, following a detailed submission by Ireland. The relevant submission will be made following publication of the proposed blueprint for road development.

The purpose of this report is to provide advice to road authorities on the strengthening of masonry arch bridges to the levels required by the Directives.

It was commissioned by the Road Works Section of the Department of the Environment and prepared by Mr. E. Molloy, Consulting Engineer under the direction of a Technical Steering Group chaired by Mr. V. Brennan, County Engineer, Roscommon and comprising nominations of the County and City Engineers Association and the Department of the Environment.

# SUMMARY

Advice is given on inspection, assessments and strengthening of Masonry Arch Bridges.

The importance of full inspection is stressed together with diagnosis of the faults encountered. Notes are included on the MEXE method of assessment and recommendations are given on the interpretation of the results.

Attention is drawn to the inspection of early scour and how this defect may be endemic to particular areas and particular rivers or sections of them.

Inspection and early repair is seen as far more important than assessments.

Advice is offered on assessments for Abnormal Loads.

Reference is made to proposed increases in certain axle and bogie loads of vehicles arising from directives of the EEC. Where upgrading of strength of bridges is being undertaken such future increases need consideration.

Advice is given on the effect of such increases on assessment.

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# SECTION 1

## INTRODUCTION

- 1.1 Of the estimated stock of 20,000 bridges in the country some 80% or 16,000 are masonry arches. Many of these bridges are close to 200 years old and some of them far older than that.
- 1.2 The bridges exhibit a wide variety of shapes and of quality of construction.
- 1.3 Shapes vary from rare cases of gothic to parabolic to segmental to elliptical. The most common shape is segmental, which although superior to elliptical is not as good as the parabolic shape which is the ideal. In very many cases, due to deformations, the original shape is not always definable.
- 1.4 Construction varies from cut stone masonry in courses with fine well defined joints to the crudest random masonry. In the latter case voussoirs can be of mixed and poor quality with large joints occupying a large proportion of the surface area.
- 1.5 Quality of construction can, with exception, vary with locality. In a general way bridges in the eastern part of the country are superior to those in the west. Many bridges west of the Shannon are of the poorest construction.
- 1.6 Distinction can also be seen between important or prestigious bridges in towns and the more rural bridges. Such bridges as Carrick-on-Shannon, Athlone, Sarsfield Bridge (Limerick), Patrick's Bridge (Cork) and Salmon Weir Bridge (Galway) are examples of prestigious bridges.
- On the other hand Drumsna Bridge (Leitrim), Shannon Bridge (Offaly), Croom and Adare Bridges (Limerick) are examples of a much poorer quality of construction.
- 1.7 Such factors as time of construction, funds available, traffic type and volumes and the personality of the Engineer presumably all contributed to these distinctions.
- The service given to the nation by masonry arches is phenomenal. It is particularly so when we consider that these bridges were built without the benefit of modern machinery and without our present day understanding, if incomplete, of the behaviour of arches. In one area, however, the lack of adequate pumping facilities caused many river bridges to be built on shallow erodible foundations, a fact which is now being highlighted by severe damage due to scour.
- 1.8 The admirable skills of the masons of previous centuries is evident in the more prestigious bridges some of which are mentioned above. One cannot say the same however for a very large number of river bridges, large and small. The fact that they have survived over hundreds of years is often not a tribute to the skill of the masons but to the extraordinary inherent strength of arch construction.

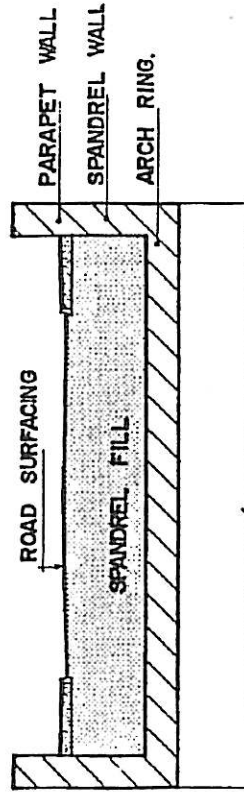
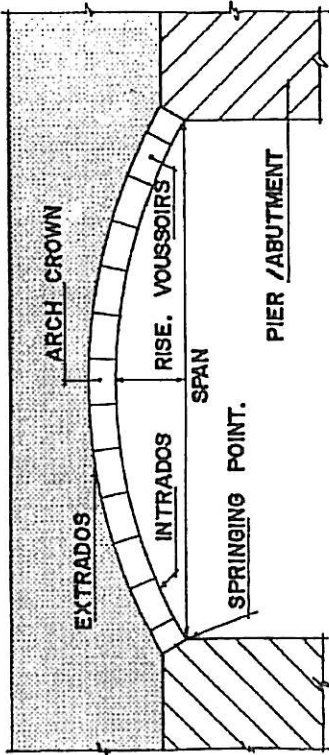


Figure 1

# SECTION 2

## SCOPE OF REPORT

2.1 The report discusses the following:

1. *Inspection;*
2. *Common faults in masonry arches;*
3. *Assessment of strength of masonry arches;*
4. *Rehabilitation of masonry arches.*

2.2. Engineers involved in inspection should be familiar with the MEXE method of assessment as outlined in Departmental Standard BD 21/84 with Advice Note BA 16/84 (Department of Transport London).

The significance of the factors used in the assessment can, irrespective of the value of the assessment itself, alert the engineer to examine certain aspects more carefully, e.g. the simple observation that "h + d" is low will prompt a careful examination since the allowable axle load is probably low.

2.3 In the matter of remedial works, it is desirable that engineers be familiar with methods of repair and strengthening, pressure grouting, guniting, saddling, tie bars etc.

# SECTION 3

## INSPECTION OF MASONRY ARCHES

3.1 The reader is referred to "Bridge Inspection Guide" 1983 issued by UK Department of Transport.

This is an excellent booklet and outlines among others the following factors which seem particularly relevant to masonry arches.

1. Bridge inspection is an important and onerous duty, not to be undertaken in a casual manner.
2. The inspector should either be specially trained or preferably be the Engineer involved in the maintenance and repair of the bridge. He should, at least, have a broad understanding of the design of bridges and be able to spot unusual defects.
3. A general preliminary inspection is often advisable prior to a principal inspection. Such information as heights, ease of access, depth of water, tides, necessity for a boat, is useful in making the prior preparations.
4. Plentiful photographs can prove very useful afterwards in discussion or to supplement notes.
5. Time should be spent in examination of the bridge. If inspected by the Engineer, he will use much of this time in diagnosing the cause of the defects and determining whether the defects are critical or otherwise.
6. This is also the most appropriate time for the Engineer to determine the Condition Factor  $F_c$  if an Assessment of Strength is intended.
7. A visual inspection of the roadway over the bridge may give preliminary warning of trouble in the barrel of the arch. Broken or depressed pavement is often an indication of distortion of arch ring, or at least of poor fill material. Similarly depressions in the line of the parapet walls or outward bowing may indicate weakness in the arch ring or outward movement of the spandrels.
8. Preliminary visual inspection of the elevations from both sides can similarly give advance warning of possible trouble. Low "h + d" values can often be observed giving warning of possible low strength and also of the need for careful measurements. Similarly deformation of the profile, and of the parapets, can often be detected. Again high span/rise ratios can be observed visually, flat arches have lower strength than the more curved ones.
9. If possible inspection in a river bridge should be carried out at low water. In very many of the smaller arch bridges, and even some of the larger bridges, inspection is possible using thigh boots. Such conditions as scour of the footings, leaching of the infill or cavities in the supports can be detected at low water.
10. In general, it is advised that prior to any detailed inspection a good general look at the quality of the pavement overhead, at the longitudinal and vertical lines of the parapet walls and at the general elevation of the bridge will reveal much.
11. Where a general examination of all bridges in a county is being organised and assessments of strength are included in this examination it is suggested that measurements such as  $L$ ,  $r_c$ ,  $r_p$ ,  $h$ , and  $d$  should be carried out separately either by junior engineering staff or trained technicians. The examination of the bridge in regard to condition of pointing, extent of cracking, deformations and their significance, scouring of foundations and the general condition of the bridge should be carried out by a more senior engineer doing all the assessments.
12. The time involved in taking measurements distracts the Engineer from the more vital examination of the defects and the critical nature of them.
13. The number of bridges to be examined or the size of the county and indeed the size of any one bridge itself may alter the above recommendation, but in long multispan bridges, at least, the taking of measurements simultaneously with structural examination is very time consuming. It is emphasised that it is highly desirable that all assessments should be carried out by one Engineer within a county or divisions of a county. In this way experience in dealing with the various factors particularly the Condition Factor  $F_c$  will lead to more consistent assessments. Furthermore, determination of priority of schemes for improvement is more effective.
14. The strength of a masonry bridge is influenced by a number of conditions not easily determined by a very detailed examination. Such conditions are:
  - (a) the nature of the fill not alone within the spandrels but behind the abutments.
  - (b) spandrel walls contributing to the strength especially in narrow bridges.
  - (c) internal spandrels add greatly to the strength and it may be necessary to explore for them.

# SECTION 4

## COMMON DEFECTS IN MASONRY ARCHES

- (d) The strength of the masonry itself and of the jointing material is a factor. Ultimate failure of an arch barrel often follows the crushing of the masonry. Portions of voussoirs dropping out would indicate such crushing and may indicate undue flexibility in the ring.
- (e) The barrel may have been saddled in the past.
- (f) Pipes through the barrel are a source of weakness and are easily observed. Occasionally large pipes have been placed by partly excavating into the extrados. Such a condition may not be obvious and should be explored.
- (g) The nature of fill within piers cannot readily be determined by probing. Some probing through joints may reveal voids behind the outer facing of stone.
- (h) The inspection of spandrels and wingwalls is often impossible due to growths of all types and clearly such growths should be removed, prior to inspection.
- (i) In important or long span bridges, it may be desirable or even necessary to probe the infill material and at the same time probe for internal spandrels. This would be indicated where low assessments were expected or found and where because of the nature of the bridge and its location gunning to the soffit would not be tolerable.
- (j) Similarly where movement of the supports e.g. piers or underscours is occurring it is desirable to assess the suitability of the infill or the foundations for grouting.
- (k) Probing into the fill, either in spandrels or supports, is obviously a costly and time consuming operation and would only be undertaken where the information was considered critical. In spandrel filling the best method is to open a number of holes through the road surface. Probing into piers and abutments is best achieved by wagon drill, relying on the skill of the operator to judge the nature and groutability of the material.

4.1 These are divided into groups as follows:

- (1) *Defects in the Arch Barrel*
  - (a) Longitudinal cracking or separation of barrel
  - (b) Transverse cracking of barrel
  - (c) Deformations in barrel
  - (d) Diagonal cracking
  - (e) Shearing movement of voussoirs relative to each other
  - (f) Punching of voussoirs
  - (g) Crushing and spalling of voussoirs
  - (h) Cracking of spandrel walls
  - (i) Longitudinal cracking under spandrels
  - (j) Loss of mortar in joints

- (2) *Defects in supports*
  - (a) Underscour of supports
  - (b) Movements and settlement of support

(3) *Bulging, listing or cracking of wing walls*

(4) *Listing or movement or parapet walls.*

4.2 **Group 1 - Defects in Arch Barrel**

All of these defects with the exception of (j) which deals with joints in the voussoirs, affect the Condition Factor  $F_c$ . These defects are outlined in 5.13. The defective mortar in joints is included under the Joint Factor  $F_j$  in the assessments and as such is not considered in assessing  $F_c$ .

4.3 **Group 2 - Defects in Supports**

1. *Underscour of Supports*

Underscour of supports has by itself, caused more damage to arch bridges than any increase in loading over the years. Because it is not readily visible there is a tendency to ignore it. Even where full access to supports at low water is possible scour action may not be obvious. Deep holes near a support can be filled in a short time by translocation of bed materials. Such local and hidden scour leaves the bridge highly vulnerable to increased and more damaging scour in a later flood.

Coupled with scour action is the leeching effect of water flow under and within a masonry support. Probes into river piers frequently reveal loose rubble fill and cavities inside the outer masonry skins. The finer materials have been leached out through joints and holes in the masonry. Such leeching weakens the support and exposes it to serious damage in times of increased scour action.

Visual inspection alone is not always sufficient to indicate the seriousness or extent of scour. Where mechanical probing is undertaken, scour and leeching effects are often found to be of greater extent than visual inspection indicates. Probing by a pointed tool under and into supports will often reveal more than anticipated. The presence of open joints and water seepage will confirm porous infill and the almost certain presence of cavities.

In the most general way scour may be divided into two categories:

- (1) *General or chronic scour caused by everyday flow with average floods.*
- (2) *Acute and sudden local scour caused by exceptional and fast flows of a catastrophic nature.*

Occasionally these overlap, the scour neglected over the years paving the way for the heavy damage resulting from the exceptional type. Local scour of a serious nature can occur during normal floods due to a local obstruction.



These matters are dealt with admirably in a Guide to Bridge Hydraulics (C.R. Neill) and in a recent paper by Dr. G. J. Farrell of OPW and it is not proposed to comment in detail.

**The following points are made:**

- (1) In all river bridges some scour action is possible and routine inspection for scour should be made.
- (2) Because of the great difficulty of dewatering, many old bridges were built on shallow foundations. Frequently the bed of the river was protected by shallow stone pitching, which over the years has been washed away or damaged, leaving the bridge prone to scour damage.
- (3) Shallow foundations on gravel beds are frequently seen in mountain and hill catchments. Bridges in these catchments are vulnerable to scour even in moderate floods and are particularly vulnerable to catastrophic floods, as experienced during August, 1986.
- (4) A feature of mountain and hill catchments, particularly in gravel beds and banks, is the translocation downstream of enormous quantities of gravel, stones, boulders, trees etc. which choke the eyes of bridges and cause severe damage. Little can be done to change the nature of a river channel over the whole catchment but the following may at least reduce the severity of damage.

- (a) Vegetation and debris trapped by a pier should be removed immediately. Obviously such bridges need inspection after each flood.
- (b) Accumulations of deposits of gravel in and in the vicinity of a bridge should be removed.
- (c) Any form of fencing across the eyes of a bridge to prevent animal passage should be removed.
- (d) Flows in channels should be normal to the bridge cross section. Deposits which tend to concentrate flows into one side of a bridge or divert flows from their principal direction should be removed. Channel flows at an angle to a support are far more damaging than parallel flows.
- (e) Pitched beds are often the only protection against scour in may bridges. These should be examined regularly and repaired where necessary. Deposits can often hide considerable damage to the pitching. Pitched floors should be carried at least 5 metres downstream and 3 metres upstream and protected by deep curtain walls.
- (f) The leading edges or cutwaters of piers are particularly vulnerable to scour and deep underpinning or rip-rap protection is indicated. In both of these cases pressure grouting before and after the repair is desirable with a view to:
  - (1) protection of the piers during excavations;
  - (2) filling of voids within and under the supports after the repair is effected.
- (g) Protection works, such as pitched floors, underpinning, aprons, and rip-rap around leading edges should not project above the normal bed and should preferably be below normal scour level.
- (h) Where piers are wide, as they frequently are, long narrow cutwaters may be helpful in training the flow and in translating any damage to a more forward and less damaging area upstream of the bridge.

The keynote to the scour problem is regular and thorough inspection of all bridges and particularly those in vulnerable catchments. Even what appears to be mild scour damage may leave a foundation exposed to severe damage by the next flood.

**2. Movement and Settlement of Supports**

1. As with many of the defects in the arch barrel movements of the abutments are often almost impossible to determine with any degree of confidence except in the "inner" or cut stone arches. The irregularity of the masonry and the lack of coursed lines makes it difficult to determine whether apparent movement was in the original construction, or occurred early in the life of the bridge or occurred in recent years. In other cases the movements are so large that there is no doubt, at least, that movement has occurred.
2. Abutments and piers, however massive they may appear, tend to move longitudinally in either direction. These movements are complex and no general rule can be given and this applies particularly to multispans arches. In single spans crown loading usually causes spread of abutments or increase in span, while loading at 1/4 span or behind the abutment causes a reduction in span. In some cases, the barrel moves relative to the abutment, usually inwards involving reduction of span. This movement is usually local rather than widespread. It is probably due to poor fill material over and behind the abutment exerting excessive surcharge pressures near the springing. In general spans tend to decrease rather than increase except where there is reduced pressure behind the abutment when the abutment tends to lean backward.

The shortening of the span is probably assisted by "ageing" of the transverse joints particularly in the presence of water. The mortar is softened and squeezed and joints become smaller. Large movements are a fairly serious condition and are accompanied by distortions of the barrel, crushing of voussoirs, punching and sometimes shearing of the voussoirs relative to each other. Improvement of the fill material in the spandrels and behind the abutments is indicated. Where forward translation of the abutment is detected some propping between abutments below bed level would be indicated.

3. Settlement is similarly not always easy to detect in rough masonry, unless it is of large dimension or accompanied by obvious cracking. Moderate settlement of a support of a uniform nature is rarely of a serious nature, the flexible nature of the barrel being able to adapt to it. Differential settlement within a support is much more serious. This is characterised by a crack of small dimension in the abutments increasing in width as it spreads into the barrel and finishing in the crown area under the parapet. This is obviously a serious condition since portion of the arch barrel is in tension due to cantilever action. Settlements can be prevented or arrested by underpinning or grouting of the foundations. Large settlements unfortunately result in serious damage to other parts of the structure, as e.g. to parapet walls, spandrel walls and wing walls, in addition to the barrel itself.

**Group 3 - Bulging, listing or cracking of wing walls**

1. Wing Walls are almost invariably parallel to the road.
2. They suffer from all the defects associated with masonry retaining walls, listing, bulging, cracking open joints, growths etc.
3. Evidence of the general weakness of construction in wing walls is the large number of such walls to which buttresses of all types were added later.
4. Wing walls, and spandrel walls, were often built of single stone depth. It is surprising that time and increasing traffic surcharge loads have not caused more serious damage than is evident. This is, to some extent at least, due to the flexibility of such walls and their ability to sustain large movement.
5. Cracking or movement of wing walls is common at the joint with spandrel walls. The stiffer and shallower spandrels are less flexible than the wingwalls which tend to greater deflection outwards. Occasionally the wing wall tends to pull the spandrel and the arch ring outwards at and even below the springings. Treatment usually consists of grouting, or provision of tie roads, or guniting or even a combination of these.
6. Movement or cracking may also be due to differential settlement of abutments and wing walls. The nature of the movement will usually indicate which is causing the defect.

**Group 4 - Listing or Movement or Parapet Walls**

1. As with spandrel walls, parapet walls tend to flex upwards and downwards with the arch barrel. Where the movement is excessive parapets may crack or display open joints particularly in the area of the crown.
2. Occasionally the arch barrel and spandrel walls settle relative to the parapets leaving them "suspended". This condition is seen in a dramatic way when an arch bridge collapses leaving the parapet walls still partially intact. In these cases a void between spandrels and parapets may be suspected.
3. In flexible barrels where rib shortening is occurring, parapets are sometimes called to bear large compressive or "column" loads. In these circumstances buckling occurs and this phenomenon is evident in many bridges in the "in and out" appearance of the parapet walls.
4. Vertical deflections in parapets again indicate permanent deflection in the arch barrel and is usually accompanied by flattening of the ring.
5. Occasionally parapet walls are exposed to vehicular impact varying from slight sideways contact to head-on or near head-on collision with varying results. In the latter case, usually occurring where the bridge is on a sharp bend, severe damage can occur not alone to the parapet walls but to spandrel walls and even portions of the outer arch barrel. The replacement and repair in masonry can be quite costly. The Engineer may consider three different types of repair:

- (a) replace masonry to existing conditions and be prepared to repeat the repair should similar collisions occur;
- (b) replace damaged arch barrel and spandrel walls in reinforced concrete providing ties to the far spandrel walls. These ties should be as strong and plentiful as possible and capable of resisting a force greater than that to cause failure of the over head parapet wall, in overturning or shear. There should be no structural connection between parapet and spandrel walls, the joint between them being a simple mortar bedding joint. The purpose in this case is to allow the parapet wall to fail before any damage is done to spandrels or arch barrel. Normally this will be achieved by providing a tie of 20 tonnes for each two metres of wall.

(c)

in this option the Engineer may consider the need to contain vehicles fully. It should be remembered that we are considering the rare case of head on or near head-on collision with a parapet wall. The requirement here is similar to those for a P6 parapet wall. The strength requirements in this option are very onerous being about 10 times those required for a normal P2 parapet wall.

In this rare case, reliance on normal transverse ties would be insufficient. The parapet wall would need to be a reinforced wall at least 400 mm thick, exclusive of masonry facing and stability would be achieved by connection to a wide transverse slab, saddle or massive concrete blocks over the springings of the arch. Needless to say the cost in this case would be quite high and the need for such high containment should be examined carefully.

6. It is not always clear which of these three options is appropriate. Where damage is minor and collisions are frequent the Engineer must decide between options (b) and (c).

In most cases option (b) will be the more appropriate solution. A near head-on collision with a high containment wall is probably more "fatal" than permitting yield of the wall. The provision of a high containment wall should be reserved for exceptional cases, e.g. over railways or where the bridge is very high.

7. The use of steel guardrail as added protection should also be considered. Unfortunately in these cases, narrow width often precludes their use.

# SECTION 5

## ASSESSMENT OF STRENGTH OF ARCH BARREL

5.1 Assessment of Arch Barrels are carried out for three reasons:

- (a) an aid to the Engineer in his assessment of the overall condition of a bridge and as a guide to the type of improvement or strengthening that may be required.
- (b) to determine whether a weight restriction should be applied and the type of weight restriction.
- (c) to determine whether a bridge is capable of carrying heavy abnormal loads.

5.2 The most common method of assessment is the MEXE method as outlined in BD 21/84 and more particularly in Advice Note BA 16/84. The method is simple and easily used, but can only give a conservative and sometimes very conservative, estimate of the strength of the arch barrel.

5.3 Other methods have been explored and are still being explored with a view to giving a more exact and less conservative estimate of strength. These include:

- (1) Elastic analysis based on limiting the compressive stresses and eccentricity of thrust;
- (2) Non linear analysis using Limit State principles. The main concentration in this type of analysis is on "mechanism analysis", the assessment of the minimum load at a critical position to cause the development of four hinges in the arch barrel.

There is particular concentration on the development of computer programmes for "mechanism analysis" in the UK. It is probable that better estimates of the strength of an arch barrel can be realised by these methods. However these methods are as yet, not sufficiently developed to suggest that they give results more reliable than the MEXE method, or that they are suitable for everyday use in dealing with a large number of bridges. It is proposed then, for the present at least that the MEXE method be retained as the general method of assessment.

5.4 The assessment by the MEXE method gives an estimate of the strength of the Arch Barrel only. The strength of the supports, wing walls, spandrel walls and even parapet walls must be examined separately and a judgement made of the effect of any defects in them on the overall strength of the bridge.

5.5 Section 5 of the Advice Note BA 16/84 outlines the method of assessment and this need not be restated in full here. For those with no experience of the method it will be noted that the examination may be divided into two parts:

- (a) *Determination of the dimensions,  $L$ ,  $r_c$ ,  $r_p$ ,  $d$  &  $h$  from which the Provisional Axle Load PAL may be deduced from the formula*

$$PAL = \frac{740(d+h)^2}{L1.3}$$

or from the Nomogram on Page 19 of BA 16/24

- (b) *Determination of the modifying factors*

Span Rise Factor	$F_{sr}$
Profile Factor	$F_p$
Material Factor	$F_m$
Joint Factor	$F_j$
Condition Factor	$F_c$



These factors are multiplied by PAL to give the Modified Axle Load (MAL).  
Part (a) lends itself to survey by Technicians or junior engineers. Part (b) it is suggested should be carried out by an Engineer with some experience.

5.6 A third part of the examination, separate from assessment of the strength of the arch barrel, is the examination of the abutments or piers for scour, movements, cracks etc. together with wing walls and parapets. This examination is equally as important, some would say more important, as that of the arch barrel. There is no ready method of applying factors to these aspects of the bridge and the Engineer must rely solely on his judgement of the significance of the defects and to what extent they place the bridge at risk of damage or even collapse.

5.7 The MEXE method is strictly not applicable to arches which are badly deformed. The line of thrust is now at best very indefinite and is certainly of irregular shape. Voussoirs are bearing on each other over small or reduced depths.

5.8 If this is so, then the value of these assessments is extremely limited since a large proportion of our arches are deformed. The very arches that need assessment most would be excluded.

5.9 It is proposed that the method should be used even for deformed arches. The Engineer will clearly take the extent of deformation into account in arriving at the Condition Factor, and where an arch barrel is so badly deformed and distorted as to threaten collapse a Condition Factor below .4 will be imposed. In such cases remedial action should be undertaken immediately.

5.10 In "rough" arches with deformations and bulging and movement of the springings, the taking of measurements  $L$ ,  $r_c$ , and  $r_q$  will pose difficulty. Even the crown may be ill defined by distortion.

One method of dealing with such cases is to plot a number of points on the intrados of the arch and to fit an approximate segmental or parabolic curve to those points. This curve could then be used to give  $L$ ,  $r_c$ , and  $r_q$ . Unfortunately this operation may need repeating a number of times across the width of the arch. It is doubtful if such a tedious operation is warranted considering the approximate and conservative nature of the assessment. Any overestimation to give favourable factors are probably well taken care of by the low condition factor applicable in such cases. In distorted arches the measurements should be taken at a number of points across the width and the mean taken.

Where the profile factor derived from  $r_q$  is enhanced by flattening of the barrel over the  $1/4$  points,

$r_q$  should be calculated using the appropriate formula for a segmental arch, deriving  $r_q$  from  $L$  and  $r_c$ .

5.11 In calculating the Material Factor  $F_m$  it is more accurate to measure the actual thickness of the barrel internally in the width rather than the thickness of the outer ring. In this however, due to the rough nature of the voussoirs, such measurements have often little meaning and in any case require openings in the pavement.

It is suggested that since the critical thickness is "h + d" rather than h or d separately it is sufficiently accurate to take the value d from the outer ring, or that value reduced by say 10%. If a reduced value is taken h will obviously be correspondingly increased since "h + d" remains unchanged.

5.12 In "rough" arches the joint factor alone will often be a very low factor, influencing the strength to a large degree and perhaps will be the single largest factor. In most of these cases joints can far exceed the 12.5 mm giving a width of joint factor 0.8. Mortar in the joint is not easily judged. However dampness, salicalities, discolouration will indicate that this is only a surface treatment over poor joint material, or none, internally. The factor will be 0.9.

The depth will often be far more than  $1/10$  of the arch thickness, resulting in a factor as low as 0.6. The combination of these three factors is  $.8 \times .9 \times .6 (= .432)$  or less than  $1/2$  the ideal strength of the arch.

It should not be assumed from the latter statement that good pointing will double the strength. It can be said, however, that since the depth factor of 0.6 will increase to 1 the strength will be increased by at least 66.6%.

5.13 The Condition Factor  $F_c$  is an objective assessment of the overall condition of the arch barrel. Engineers often find difficulty in arriving at the Condition Factor and it may be helpful to understand the steps by which the strength of the barrel is derived.

(1) The arch is initially idealised as a well built arch in "perfect" condition with ideal span/rise proportions and ideal profile factor. It is free of all cracks, deformations and abutment movement. The stonework is of a high standard and joints are thin and fully pointed.

(2) The PAL is derived considering span and "h + d" only for this ideal arch.

(3) The factors  $F_{sr}$ ,  $F_p$ ,  $F_m$ , and  $F_j$  are applied reducing the PAL where the span/rise, profile, materials and joints are below a very good acceptable standard.

(4) The remaining factor  $F_c$  takes account of all other defects in the arch ring under a single global factor. These defects are all forms of cracks and deformation in the barrel. It will not include defects already included in the other four factors. In making his assessment of this factor the Engineer will consider the following:

(a) *Longitudinal Cracks* spreading from a support or supports, usually resulting from differential settlement in one or both supports. He will judge whether these cracks are long standing or recent and whether they are sufficiently wide to indicate that the barrel has broken up into separate load bearing sections. The pavement overhead will sometimes indicate by cracking or depressions that such separation is serious. Very many of the older bridges were built in two portions, not always equal portions. In most cases the abutments or vertical piers seem to have been in continuous construction while the barrels were fully separate. The purpose is obscure but the dual use of centering may be one reason and in some cases later widenings occurred. The longitudinal and fairly uniform joint is sometimes confused with longitudinal cracking, which would imply differential settlement of the supports. The lack of bonding between the portions, the uniformity of the joint particularly in multi-span bridges and occasionally the differing stonework will indicate separate construction.

(b) *Transverse Cracks*, mainly due to outward horizontal movement of the abutments, or of the barrel relative to the abutment. Such cracking is not usually observed as the arch ring will tend to accommodate itself to the resulting strain by deforming sufficiently.

(c) *Deformations, bulges and all forms of distortions.* In some cases these can be so extensive in the cruder arches that the arch form bears no relation to the original form. At first sight the deformations transversely and longitudinally are so large that imminent collapse may be expected. Such large deformations are indicative of a weak arch barrel, with poor jointing material and low or poor fill material.

The engineer will again discern whether these deformations are long standing or recent and whether the barrel thickness can accommodate such deformations. Where they are obviously excessive, some action is necessary to prevent possible collapse.

(d) *Diagonal Cracking.* Usually extends from the abutment at one side diagonally to crown area of the barrel on the other side. The cause is usually differential settlement of portion of one abutment. This can be a very serious condition as portions of the barrel tend to be suspended as cantilevers.

(e) *Shearing of voussoirs relative to each other.* In "crude" arches this is not easily observed since the rough masonry often projects in a haphazard way. It may, however, be observed in the outer ring and is often accompanied by other deformations.

In "fine" arches of cut stone built in courses the defect is more easily observed, the courses becoming "stepped" relative to each other.

This condition may be due to a number of causes or a combination of them. It is perhaps not a coincidence that masonry arches are weakest at about  $1/4$  point, and flattening of the arch barrel and the shearing effect under discussion is more often than not observed in this area.

The weakness at the  $1/4 - 1/3$  point in an arch judged from the influence line for a two hinged arch is usually balanced by the fact that superimposed loads in this area are spread transversely through the spandrel filling. If the fill material is of poor quality and softened by ingress of water, load distribution is poor and moments in this area may be high. Hinges tend to develop in the extrados in these areas i.e. the line of thrust passes through or is close to the extrados. This condition resulting in tension in the intrados predisposes to slippage of the voussoirs.

Another cause is the spread of the abutments. This is more frequently observed in multi-span arches where fill over the abutment is often reduced by approach gradients. The reduced abutment thrust permitting of the spread again tends to form hinges between the springing and crown resulting in slippage.

It is noted that punching of voussoirs as described in the following paragraph is more prevalent in the abutment span of a multi-span arch bridge, presumably for the same reasons. Whatever the cause, the condition is serious. Any undue spread of the abutments must be arrested if serious damage or even collapse is to be avoided.

Obviously any upgrading will depend on the Engineers assessment of the general condition of the arch barrel, and so will be directly related to the factor  $F_c$ . It is proposed that the MAL be upgraded by a factor of 2  $F_c$ .

5.16 The MAL may be rewritten as follows:

$$\begin{aligned} \text{MAL} &= 2F_c (F_{sr} F_p F_m F_j F_c \text{ PAL}) \text{ or} \\ \text{MAL} &= 2F_{sr} F_p F_m F_j F_c^2 \text{ PAL} \quad (F_c > .5) \end{aligned}$$

The above MAL applies only where the factor  $F_c$  is greater than .5. Where the factor  $F_c$  is .5 or less the original assessment would hold i.e.  $\text{MAL} = F_{sr} F_p F_m F_j F_c \text{ PAL}$ .

5.17 Where e.g.  $F_c = .7$ , the original MAL is upgraded by 40%.

5.18 The formula in 5.16 represents the final allowable axle load if the following additional factors are not be applied.

- (a) *Curvature Factor*  
(b) *Axle Factor, without or with lift-off*

These factors are dealt with in 5.19, 5.20 and 5.21.

5.19 Curved Carriageways

Where an arch is on a horizontal curve particularly of sharp radius, an allowance for the increase in vertical loading caused by centrifugal effects may also be necessary. This factor  $F_A$  is derived from the formula:

$$F_A = 1 + \frac{.20v^2}{r} \text{ where } v \text{ is in m/s and } r = \text{radius of curvature.}$$

This may also be written

$$F_A = 1 + \frac{.01543v^2}{r} \text{ where } v \text{ is the more used form of km/hour.}$$

This factor may be ignored in radii exceeding 600 m and it is suggested also where the factor is less than 1.25 as in 7.6.3 of BD 21/84.

It will be noted that this can be a very severe factor as the reader will confirm by doing a few examples e.g. take the case of a curve of 100 m radius. The appropriate  $v$  is derived as:

$$\begin{aligned} v &= 3.6 \sqrt{\frac{1000r}{r+150}} \text{ km/hour (Page 36 of BD 21/84)} \\ &= 72 \text{ Km/hour (20m/s)} \end{aligned}$$

$$F_A = 1 + \frac{.01543v^2}{r} = 1.80$$

The Modified Axle Load derived in 5.16 should then be divided by  $F_A$ .

In the above case the Modified Axle Load would almost be halved by this factor. The application of Centrifugal Effect Factor  $F_A$  should be examined with some care because of its relatively large effect, particularly on sharp curves. It may well be that because of the general nature of the road, its width and substandard alignment approaching the bridge, that speeds assigned from the formula  $v = 3.6 \sqrt{\frac{1000r}{r+150}}$  km/hour will not be reached

(f) *Puncturing of Voussoirs*

This condition is common where cover over the arch ring is small. Long voussoirs protruding upwards near the road surface can be punched downwards by road traffic. This is often followed by dropping out of neighbouring voussoirs. As mentioned previously this occurrence could indicate lengthening of the span by outward movement of the abutment. Since even local punching leaves surrounding voussoirs vulnerable, immediate repair is necessary by filling the void with masonry or concrete.

(g) *Crushing and Spalling or Voussoirs*

Arch barrels behave elastically in a general way and deform under load. The extrados or intrados of a joint between voussoirs may be experiencing continual variation between compression and tension depending on the location of a heavy load relative to its location in the span. Upward deflections may occur in portions of a span removed from the load. In some old arches built of thin barrels with poor and ageing masonry, voussoirs crack under heavy loads and eventually fall out. This condition in a less dramatic way may be observed in some of the better bridges where edges of voussoirs on the outer rings are crushed or spalled off.

The condition would generally indicate a flexible barrel. Thorough pointing and pressure grouting of masonry and filling may stiffen the barrel, but if the condition is widespread, saddling or thick grouting would be indicated.

(h) *Cracking of spandrel walls within the span*

Considering the downward and upward movements of a flexible arch, it is not surprising that spandrel walls crack. Cracking is sometimes seen in areas near the crown (between quarter points) again indicating as in (g) a flexible arch. The same remedies may be applied. In serious cases the provision of concrete backing to the spandrels might be considered.

(i) *Longitudinal Cracks under the Spandrel Walls*

This is probably the most common defect in masonry arches and will be seen even in the better types. The word "crack" may imply small width. This type of defect is more often characterised by a large separation of the outer ring from the main barrel. The cause is probably the outward thrust from fill and traffic surcharge forcing the spandrels outwards and separating the outer ring from the main barrel. The stiffening effect of spandrel walls and even parapets may also contribute to the development of this type of cracking, the inner arch being more flexible. Another possible cause is the effect of percolation of water which leeches out the mortar in the joints. Such leeching is greater in longitudinal joints than in transverse joints since the latter tend to close up under arch action. Surface water from the road is usually concentrated near the parapets.

Whatever the cause, the cracking or separation must be seen a fault to be corrected. It is not always seen as a major fault and is not generally included as a factor in the general assessment. Where such cracks occur in a narrow bridge with vehicles plying near the parapets, or where they occur on both sides of the bridge, it is suggested that they should be included for consideration in arriving at the Condition Factor. This would be particularly true if the separation(s) were occurring some distance in from the parapet.

Considering the defects he finds in the arch barrel the Engineer will arrive at a Condition Factor  $F_c$  between 0 and 1. Little guidance can be given as to the factor to be applied. Much would depend on the severity of the defects, his diagnosis of the causes and his judgement as to whether the bridge is in a dangerous condition. It is emphasised that a high condition factor of say .8 does not mean that the bridge does not need repair.

5.14 Having determined all the factors the Modified Axle Load is as follows:

$$\text{MAL} = F_{sr} F_p F_m F_j F_c \text{ PAL}$$

5.15

As outlined earlier the MEXE method is a very crude empirical method of assessing the strength of the Arch Barrel. Recent tests to destruction of a number of Arch Bridges in UK suggest that the ultimate strength of these arches varied between 3 and 12 times the strength by the assessment. In a less direct way, we ourselves have noted how frequently assessments yield strengths as low as 5 or 6 tonnes and yet the bridge is carrying normal traffic loading with no apparent distress. There seems no doubt that the method is highly conservative. What is not clear is to what degree. It would seem desirable that the MEXE method, if it is to be of any real value, should be modified upwards. Further testing of bridges in UK is proceeding which may yield firmer conclusions in this regard, although experimental work seems to be directed more towards analytical methods of assessment rather than modifications of the MEXE method. Pending further information it is proposed that the MAL derived above should be upgraded by an additional factor between 1 and 2. Because of limited information this upgrading factor must be conservative.

**Table 5.6A**

**Axle and Gross Vehicle Weight Restrictions for Masonry Arches**

Span m	AAL		Restriction
	Double t	Single t	
< 4	10.5	10.5	All C & U vehicles permitted All C & U vehicles permitted Single Axle Restriction only = value of A.A.L.
	10.0	10.0	
	<10.0	<10.0	
> 4 < 6	10	10.5	All C & U vehicles All C & U vehicles All C & U vehicles 15 12 10 8 7
	9	10.5	
	8	10.0	
	7	8.5	
	6	7.5	
	5	6	
	4	5	
3	4		
> 6	10	10.5	All C & U vehicles All C & U vehicles 20 16 14 12 9 7
	9	10.5	
	8	10.5	
	7	10.0	
	6	-	
	5	-	
4	-		
3	-		

**5.20 Axle Factor - No Lift-Off Fig 5.5a BA 16/84**  
The MAL derived from 5.16 represents the effect of a two axle bogie in spans over 4 m. In spans below 4m the single axle effect is dominant and in these spans the MAL represents the single axle. A 9t assessment at 4m is equivalent to a 9t single axle and a tandem bogie of 18t. The same assessment at 6m yields a single axle of 9 x 1.28 = 11.52t and a tandem bogie of 18t. The graphs in Fig. 5.5a are drawn from the envelopes of the maximum bending moments for all C & U vehicles.

For most spans the critical two axle bogie is the 9.15t with a spacing of 1.2m. An assessment of this value would be adequate for many of the 3 axle bogies. Unfortunately it would be inadequate in the shorter spans for the closely spaced triple axle bogies and in the longer spans for the wider spaced bogies. To cover these deficiencies the 2 axle bogie is increased to 10t per axle. A rating of 10t for the 2 axle bogie is deemed adequate to cover all C & U vehicles. Below 4m span a rating of 10.5t is required to cover the allowable single axle load of 10.5t. Fig. 5.5a is used principally for weight restriction purposes. It is necessary only to derive the single axle load corresponding to the 2 axle bogie assessment. The 3 axle bogie values need not be derived since weight restrictions are evaluated from the single and double axle values only. Weight restrictions are dealt with in 5.23.

**5.21 Axle Factor with Axle Lift-Off**  
Because of a severe hump-back bridge or pronounced irregularity in the road surface, axle lift-off may occur in two and three axle bogies even where compensating mechanisms are provided. Since the effect has a pronounced influence on the assessment the Engineer should satisfy himself that the condition warrants the introduction of the factor  $A_f$  for lift-off. The general factor for the two-axle bogie is .8 for spans up to 11m and varies linearly from .8 at 11m span to .95 at 20m span. As with the Axle Factor with no lift-off, there will be no need, normally, to derive the factors for the triple axle. For a satisfactory result up to 11m span an assessment of 10/.8 = 12.5t is required. If the 2 axle bogie is satisfied, all other axle configurations will also be satisfied.

**5.22 The final Allowable Axle Load may now be stated.**

$$1. AAL = \frac{2 F_{sr} F_p F_m F_j F_c^2 PAL A_f}{F_A} \quad F_c > .5$$

$$2. AAL = \frac{F_{sr} F_p F_m F_j F_c PAL A_f}{F_A} \quad F_c \leq .5$$

In the general case where Axle Lift-off is not considered and severe curvature is not involved the factors  $A_f$  &  $F_A$  will be unity in each case.

**5.23 Weight Restrictions**  
Weight restrictions are difficult to enforce and should be avoided wherever possible. A large proportion of masonry arch bridges are of short span and usually can be upgraded at relatively small cost. Where axle lift-off is a factor regrading and surfacing of the roadway should be undertaken where possible. Some regrading and surfacing may not alone remove the effects of axle lift-off but greatly increase the AAL where additional thickness of pavement is added at the crown. Where the assessment falls below 10.5t in the single axle and 10t for the double axle, upgrading of the bridge or the imposition of a weight restriction may be considered necessary. In the exceptional case where a weight restriction cannot be avoided, guidance would normally be sought from Table 5.6 of BA 16/84. The regulations regarding weight restriction here vis a vis those in the UK are different. In the UK vehicles are 'plated' and there is considerable concentration on the type of vehicle and its potential GVW. In this country we have no plating system and controls are concentrated on actual axle load and GVW of the vehicle. Because of these considerations Table 5.6 is not suitable for conditions here. An alternative table, labelled 5.6A is drawn up offering guidelines in cases where weight restrictions become necessary. Table 5.6A will be seen only as a guide to the Engineer. He may well decide from his detailed inspection of the bridge and from its general condition that assessments lower than those shown in 5.6A would not require a weight restriction.

**5.24 Further Requirements**  
To comply with recent directives of the EEC the regulations in regard to certain vehicle loading will require amendment. The changes will come into effect gradually. The principal changes are as follows:

1. Single Drive Axle - 11.5t replacing 10.5t currently.
2. Triple Axle Bogie, 2.7m spread - 24t replacing 22.5t currently.

Where strengthening works are being carried out these future requirements need consideration. In regard to the single axle, upgrading to a value of 11.5t is required in spans up to 5m. In spans over 5m upgrading to the double axle value of 10t will automatically yield 11.5t for the single axle. In regard to the 24t triple axle bogie, examination of Fig. 5.5a will show that a 10t assessment is adequate for 3 axles of 8t at 2.7m spread up to 11.5 span. Even over 11.5m the effect of 3 x 8t axles is only marginally greater than that of the 10t tandem axle. In the light of this it is proposed that the standard of 10t will continue to be adequate for all C & U vehicles in spans over 5m.

# SECTION 6

## GENERAL COMMENT ON ASSESSMENTS

- 6.1 It will have been gathered from the discussion on assessments that the MEXE method is, at best, a crude method. It can only yield an approximate notion of the strength of the arch barrel.
- 6.2 Some bridges in good condition may show low assessments and contrariwise some very poor bridges may show high assessments. That the defects are found and assessed and corrective measures taken is far more important than the accuracy of the actual assessment.
- 6.3 The real value of assessments may be as follows:
- (1) The bridge is inspected in some detail.
  - (2) Faults are observed and the causes assessed.
  - (3) The build up of factors leading e.g. to low AAL will often indicate concentration on certain forms of repair rather than others. The large increase in strength from good pointing or spray gunning has been mentioned in 5.12. A low "h + d" value may be one of the significant factors giving an initial low PAL. Increasing the value of "h + d" alone may yield a substantial increase in AAL. The value of (h + d) may be increased as follows:
    - (a) Where the arch barrel is in good condition and pointing is good and where the vertical profile permits, a surfacing of 100mm or more will greatly improve the strength.
    - (b) Where pointing is poor or longitudinal cracking in the barrel is present, then thick gunning up to 150mm thick is indicated.

- 6.4 In general, any assessment is seen only as an aid to the Engineer in forming his opinions. In this matter the judgement of an experienced Engineer, having examined the bridge in detail, will be superior to any assessment.

# SECTION 7

## ASSESSMENT FOR ABNORMAL LOADING

- 7.1 Depending on the district or area of the country, the Engineer is occasionally faced with the task of assessing the suitability of a bridge or a number of bridges for the passage of Abnormal vehicles. These are usually ESB trailers and typical details are shown in Figures 2 and 3.
- 7.2 The MEXE method is strictly not applicable for vehicles outside the range of normal C and U vehicles. However, in the absence of any other more reliable and readily usable method, there is no option but to use it. It will at least be of some assistance to the Engineer in arriving at a decision.
- 7.3 The task is usually an unenviable one for the following reasons:
- (1) It is desirable in the national interest that the ESB should be accommodated.
  - (2) The axle loads of the laden trailer may be in the order of 20t i.e. about twice the allowable axle load on a normal two-axle bogie. In addition there are often 6 axles in a bogie carrying a total load of 120t or more.
  - (3) A normal MEXE assessment is often far below the axle loads of the trailer.
  - (4) The vehicle, or a similar one, may have been permitted passage previously, apparently causing no damage.
  - (5) There may be no other suitable alternative route or the bridges on the alternative route may have no greater capacity.

- 7.4 An assessment as follows is recommended:

1. A MEXE assessment should be carried out in the normal way using the formula as in 5.16:

$$MAL = 2F_{sr} F_p F_m F_j F_c^2 PAL \quad (F_c > .5)$$

- to the exclusion of the factors  $A_r$  and  $F_A$
2. Since the operating speed of these vehicles is very low and since the vehicle can negotiate sharp bends only at extremely low speeds, the curvature factor  $F_A$  may be taken as unity.
  3. Considering also the number of axles in the bogies, 4 or 6, and also the slow operating speed Axle Lift-Off may also be ignored. Obviously however care should be taken that the very low trailer does not foul a hog-backed bridge.
  4. Using the half-span rule, comparison is made between the bogies of two ESB trailers and a two axle bogie of 10t per axle with a 1m spacing. The latter is chosen as it represents the two axle bogie in Fig. 5.5a of the Advice Note i.e. with an Axle Factor of Unity. The loads and dimensions of the two ESB heavy vehicles are tabulated below. Outlines of the vehicles are shown in Figures 2 and 3.

Total Load	Load/Bogie	Number of Axles	Spacing of Axles	Load/Axle	C/C of Bogie
140t	70t	4	1.3m	17.5t	15.5m
240t	120t	6	1.8m	20t	22.8m

In general the Abnormal Load Effect is about twice that of the 20t bogie. A factor then relating the effect of the Abnormal Loads to the accepted standard i.e. the 20t bogie would be in the order of .5. This factor is referred to as  $F_{ab}$ .

- 7.5 Other considerations tend to reduce the effect of the Abnormal Load. These are:

- (a) The Abnormal Load is a rarely occurring load.
- (b) Bridges carrying these loads will normally be two lane or wider. An assessment of 10t would imply two bogies of 20t each in line transversely i.e. a total load of 40 tons spread over at









# SECTION 8

## THE REHABILITATION OF MASONRY ARCHES

8.1

### General

The Engineer will be familiar with all the tools available to him in tackling the strengthening of arch bridges. These would include:

- (1) The contractors specialising in this work. They can be of considerable help to the Engineer in deciding a course of action which is often influenced by budgetary considerations.
- (2) Such applications, as pressure grouting, with cement or combinations of cement and bentonite, grouting of arch soffits, saddling of arch rings, that are commonly used to strengthen and refurbish arch bridges.
- (3) Chemical grouting is also used, although this is not as yet much used in bridges in this country. Increase in its use in future is anticipated.
- (4) The rods with or without end plates are used transversely to tie separating arch rings, spandrel walls and listing wingwalls.
- (5) Small diameter piling of 150 - 200mm diameter inserted through abutments and piers are occasionally indicated.

8.2

### Pointing

As outlined in 5.12 the joint factor alone can reduce the strength of an arch barrel considerably, to a figure like 50% of its ideal strength. The raking out of joints and removal of loose mortar followed by adequate pointing can increase the strength as assessed considerably in certain cases. Opinions are divided about the nature of the mortar to be used. It is clear however that filling of the joints with almost any mortar is the first requirement. Traditionally lime mortars were favoured because although less strong than cement mortars, their resilience seemed to be better suited to load transfer through the joints. Mixes of 1 cement to 1 lime to 6 sand are used with a view to getting the best of both worlds and this is recommended for manual applications. However for shallow work 1 cement to 3 sand is quite suitable. Non shrink additives are frequently used and when they contain a plasticiser, are particularly useful. It is emphasised that cleaning out of joints, the removal of sandy inert mortar and growths of all kinds is necessary prior to any pointing. In the case of soffits of arch barrels and river faces of abutments and piers another method of pointing may be the use of spray gunning. Compressed air and water jet are used to clean down the joints and stone work prior to application of gunite. Repeated application will fill the deeper joints. This method is far more effective than manual pointing. However, it has the disadvantage of spoiling the masonry appearance, although undersides of arches are not usually clearly visible.

8.3

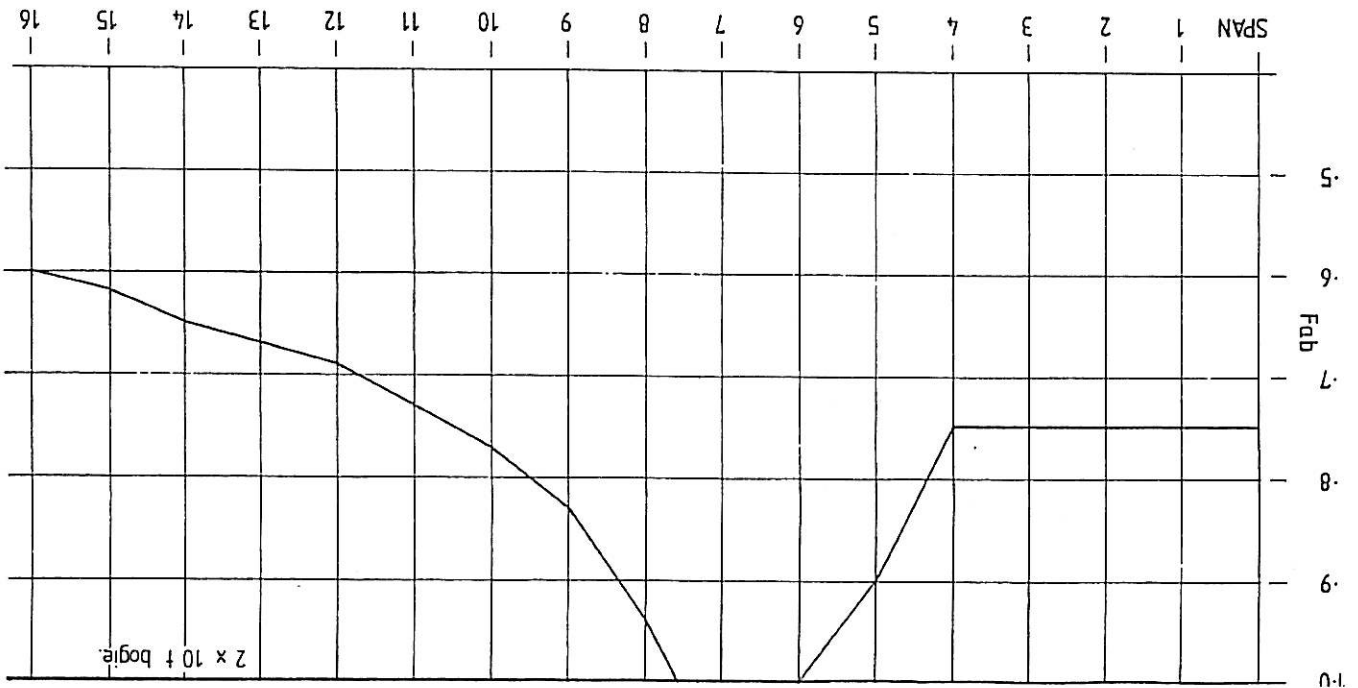
### Pressure Grouting

Pressure grouting may be applied to almost all parts of an arch and can if properly applied achieve many good results. Typical uses are:

- (1) The stiffening of subsoils under supports, and the filling of voids within supports. It is particularly useful with granular or stoney materials. Where curtain walls or aprons have been applied to remedy or prevent scour, pressure grouting subsequently is highly beneficial as the grout is better contained. Even where the subsoil under a foundation is not readily groutable, clays or silty clays, pressure grouting will compress the material, squeeze out water and improve its bearing capacity. Pressure grouting of masonry with cement is usually highly successful, provided joints are sealed prior to application. Pressure grouting of arch barrels themselves usually has good results, the grout filling the voids and at times placing a layer of grout concrete over the extrados. The grouting of the spandrel filling is usually less successful depending on the nature of the fill. The same remarks apply to vertical walls, e.g. wing walls or retaining walls. The fill behind walls if amenable to grouting, will exert less outward pressure and even act in a composite manner with the wall.
- (2)

Figure 5  
Fab for 20t axles at 1.8m spacing

SPAN m	Fab	Reqd. MAL
1	.75	13.3t
2	.75	13.3t
3	.75	13.3t
4	.75	13.3t
5	.9	11.1t
6	1.0	10.0t
7	1.0	10.0t
7.6	1.0	10.0t
8	.94	10.6t
9	.83	12.0t
10	.77	13.0t
11	.73	13.7t
12	.69	14.5t
13	.67	14.9t
14	.65	15.4t
15	.62	16.1t
16	.60	16.7t



- (3) Opening of the spandrel filling or probing for samples is usually recommended before embarking on pressure grouting. This is certainly indicated in large works where the stiffening of the spandrel filling is seen to be advantageous. For the average single span arch it is somewhat doubtful if the information is of great value, particularly if costly sampling is undertaken. The Engineer is usually satisfied to pressure grout the arch ring and to see the grouting of the over head filling, at little extra cost, as a bonus.
- (4) One of the difficulties with spandrel filling, or any filling behind walls, may be the reluctance of grout to penetrate fill of small particle size like clays and silty clays. Cement - Bentonite mixtures while reducing the grout strength, penetrate better than pure cement grouts. There is no good reason why Bentonite should not be used for spandrel filling, cement only being used for the arch barrel.
- (5) Chemicals are now being used increasingly for pressure grouting. Their use in bridge works is somewhat limited due to high cost, toxicity and extreme care required in handling. Detailed knowledge of the materials to be grouted including grading of the finest particles, is required before attempting their use. The use of chemicals would be confined to special problems and they are not indicated for routine bridge works.
- (6) Some bridges, usually the more prestigious long bridges, contain internal spandrel walls often forming boxes and covered by stone flags. The presence of such boxes should be confirmed before pressure grouting, otherwise the volume of grout used could be embarrassing. At other times they are filled with clay or fine material.
- (7) Water/cement ratios in grouting mixes vary from 0.5 to 3 depending on the penetration and "lake" of the materials. A common mix in bridge works is 1:1. The seepage of grout from observation holes of joints will give some idea of "travel" of the grout. Loss of grout into river beds sometimes encourages Engineers to increase the thickness of grout or to use sand. This may be indicated when the loss is excessive, but such action will usually reduce penetration and may be more of a disadvantage rather than an advantage. A few bags of cement can cause high discoloration in a river flow and must be expected.
- (8) Where it is desired to contain grout in loose granular materials e.g. under abutments or piers, vertical diaphragm walls in thick sandy ground around the perimeter can be effective.
- (9) Pressures in grouting vary with the mix, purpose, amount of overburden and possible damage. Normally pressures rarely exceed  $35 \text{ N/mm}^2$  (50 psi). In pressure grouting spandrel filling care must be taken not to damage road pavement. Bulging of the pavement can occur where the depth of fill is low.
- (10) Services of all kinds need to be located in bridge decks. They can be damaged by drilling or by the ingress of grout. Sewers are particularly vulnerable.

#### 8.4.0

##### Guniting

Guniting is the term applied to sprayed concrete. Normally gunitite is a mixture of cement and sand projected on to a surface in layers up to 50mm thick. The reader is referred to the Draft Code of Practice issued by the Association of Gunitite Contractors. When aggregates in excess of 10mm are used the sprayed concrete is called "Shotcrete". Guniting may be applied to any surface, masonry concrete, rock brick or steel. We are concerned here almost totally with the application of gunitite to soffits of arch barrels and river sides of supports of masonry arches. Its use has been greatly accelerated in recent years because of its versatility and relative cheapness.

#### 8.4.1

Normally guniting is applied in two forms:

- (i) **Spray gunitite.** A thin application akin to rendering. It may have the following purposes.

- As a means of pointing, particularly where masonry is rough and joints are wide and deep, already described under 8.2 pointing.
- As an initial adhesive and smoothing coat in preparation for a normal thicker or strengthening layer.

- (ii) **Gunitite or Thick Gunitite**

This is applied in layers of 10mm to final depths of 50mm to 150mm and on occasions or locally to much larger depths. Gunitite under this heading will usually have the following purposes:

- To protect masonry from undue weathering or attack from reactive chemicals.
- In most cases, thick gunitite is applied with a view to strengthening the arch barrel.

#### 8.4.2

##### Conditions for which thick gunitite is indicated

- Where  $(t + d)$  is too low to give the required M.A.L. an increase in "d" of 150mm will generally increase PAL by up to 100% particularly in the shorter spans.
- Where longitudinal or transverse or even diagonal cracks are judged to be a serious nature, it is clear that an application of 150mm of reinforced concrete will tie separated sections of the arch barrel together both longitudinally or transversely.
- Where there is punching of voussoirs. Again thick gunitite will span across any weakness which is causing punching or resulting from punching.
- Where large deformation of the arch barrel is occurring. The increased thickness will ensure that the line of thrust is restored within the barrel.
- Where outward movement of spandrels is causing separation of the arch barrel under the spandrels.

#### 8.4.3

##### Thick gunitite for strengthening purposes is carried out as follows:

- Surface is sand blasted to remove poor mortar, scale and flaky stone growths etc. Prior to guniting it is cleaned down with air jet or water jet.
- Spray gunitite is applied, normally 10mm thick or less, which fills holes, wide and deep joints.
- A layer of  $50 \times 50 \times 10$  gauge mesh is fixed using Hilti fasteners and with an inside cover of at least 15mm.
- Gunitite is applied up to 50 to 75 mm thick depending on final thickness.
- If 150mm or more is intended another layer of mesh steel is fixed, again with at least 15mm inside cover.
- The final spraying is brought to final thickness.
- It is suggested that at least 30mm of cover to outer steel be applied. In marine locations this cover should be increased to 50mm to prevent possibility of chloride attack.
- Where a good finish is required, and the gunitite is to be worked by trowelling, a final spray coat should be applied for this purpose, as working reduces the strength of the main layers.

#### 8.4.4

##### Mixes for Gunitite

Gunitite is mixed at 1:3 1/2 by weight. Allowing for rebound of the predominantly larger particles the final in situ gunitite is a very rich mix of 1:2 1/2 or 1:2. Its strength is usually well over 40 N/mm<sup>2</sup>.

#### 8.4.5

To hide the appearance of the application in elevation the edge of the gunitite is often tapered to a thin section at the spandrel edges of the ring. Where it is required to tie the spandrels to the arch as indicated by separation under the spandrel, such tapering obviously reduces the strength of the tie and should not be done. In general it is advised not to taper the gunitite, in this way. The effect of the full depth gunitite on the elevation of the bridge is minimal.

#### 8.4.6

Ideally gunitite to full thickness should be carried to a springing string course, or if possible where indicated to a curtain wall or to the foundations. Where no springing course is present, the gunitite course is abuted onto a transverse layer of projecting dowels and then tapered at about 4:50. However the adhesion of gunitite to the arch ring is so strong that the thrust in the gunitite ring is probably adequately transferred into the masonry arch at the ends. Obviously, however, the dowels are on added safety precaution and it is recommended that they be generally provided.

#### 8.5.

##### Saddling

- Saddling involves the stripping of the spandrel fill and the provision of a concrete ring over the masonry barrel.
- Where it can be achieved the results are excellent since in effect a new arch barrel is provided.
- Thickness is usually 200 - 300mm with 12 mm  $\beta$  bars at 150mm centres top and bottom in both directions. If the span is relatively long, say in excess of 9m, a short length of shear key may be inserted at the crown to reduce shrinkage effects in the concrete being transferred into the old barrel. Reinforcement should preferably be lapped in this area rather than carried through.
- A weakness with simple saddles is the lack of abutment at the springing ends to take the horizontal thrust. Without this the thrusts are transferred into the old arch through friction. It is desirable to develop abutment action by thickening of the springing area or by a sizeable mass concrete block, or by a vertical reinforced wall to develop resistance form the fill behind it. In small arches the simple solution is to fill the spandrel area completely with concrete although this is somewhat uneconomical.
- Strengthening by saddling has many secondary benefits.

- The barrel factor and fill factor are improved, it is assumed that suitable fill material superior to the old will be provided. The PAL may not increase unless the road level is raised. If full concrete infill is provided the material factor  $F_m$  will be improved.

# SECTION 9

## GENERAL COMMENTS ON REHABILITATION

- (b) Saddling ties together all cracks which tend to separate the ring into units whether longitudinal or transversely.
- (c) Where keyed into the spandrels, it ties the spandrels together.
- (d) It relieves the spandrels of outward thrust due to traffic surcharge. Further relief may be obtained by strengthening the spandrels by a vertical backing wall.

5. It fills any voids in the extrados of the arch ring. The extrados should be cleaned down thoroughly to remove loose mortar, clay etc, so that there is a good key between concrete and masonry unless separation is intended.
6. Saddling is very economical and much less costly than guniting. No falsework or formwork is needed, special skills are not required and it can be speedily executed.
7. Its major drawback is that the roadway needs to be totally or partially closed during the operation. A further disadvantage is that it does not improve the intrados of the old arch and such defects as punching voussours, open joints and deformations remain. For these reasons strengthening by guniting is often preferred to saddling.

### 8.6 Transverse Tie Bars

1. Transverse tie-bars, usually with end plates, are indicated for all types of longitudinal cracking and for listing spandrels and wing walls. The method is very old as it conformed by the large numbers of them provided in the past.
2. The rods usually of 22 - 28 mm diameter are threaded through holes drilled transversely through pregrouted masonry and filling. Normally they are spanner torqued onto buttress plates at each end and occasionally, but rarely, they are stressed by jacks. The holes are subsequently filled with grout.
3. Where corrosion or chloride attack is expected, the use of thicker bars or galvanising should be considered. The use of larger bars can increase the cost considerably unless single length bars, usually 6m long, can be used, when couplers can be avoided.
4. Tie bars would be used for longitudinal cracking in an arch where this was the main or only defect. They would always be accompanied by pressure grouting. While not as effective as saddling or guniting, they are far cheaper, a single tie bar with end plates costing about £300 - £400 depending on length. Tie bars are particularly useful in listing or bulging wing walls.

### 8.7 Anchor Bars

1. Short or even long bars are used to stretch across local opens or cracks. The holes bored are grouted or filled with chemical bond. They are little different from tie bars except they are usually short and are rarely if ever fitted with plates.
  2. Such bars are usually used to tie concrete to masonry. Their use in arch rings is fairly rare. They are useful in tying curtain walls around supports to the masonry abutments or piers. It is noticed occasionally that concrete curtain walls as described may move or settle away from the masonry and loose some of their value in protecting against leaching, scour and load dispersal.
- For this purpose, it is usual to place tie bars at 600mm centres with right angled bends into the new concrete.

### 8.8 Small Diameter Piles

1. Their application is rare and would be confined to bridges in the following conditions.
  - (a) Fairly large bridges where large settlements of the foundations are occurring and where such settlements are not otherwise easily arrested e.g. foundations on clays or silty clays.
  - (b) Foundations in deep water e.g. tidal inlets where the cost of underpinning would be excessive or very costly.
  - (c) Where increased loading is anticipated above the capacity of the foundations. This may be due to widening of the bridge without widening of the supports, or the use e.g. of quay walls as abutments to a new bridge.
2. The piles are inserted similarly to bored piles except that the holes are bored vertically or near vertically through the masonry (or concrete) supports.
3. Because of their relatively high cost their application is limited.

- 9.1 It is clear that the cause of defects should be ascertained before remedies are applied. There is little value e.g. in repairing a cracked arch barrel resulting from settlement of a support, if the settlement is progressive and allowed to continue.

- 9.2 The above is quite understandable in certain cases but in certain cases only. There will be many cases and perhaps the bulk of these cases where an arch bridge suffers from almost every defect. There are cases where defects are predominantly due to weakness of the arch barrel, others due to weakness of the abutment and others where it is not clear which is the cause, or where both are defective.

- 9.3 While it is prudent not to undertake unnecessary work it should also be borne in mind unless indications are to the contrary that in many old arches it may well be stated that everything is somewhat substandard. In this consideration also is the cost of the repairs. It is unthinkable almost that an arch barrel be grouted without grouting the supports. This is stated for three reasons:

- (1) The condition of the supports, or the internal filling of the supports may not be fully known.
- (2) The additional cost of grouting the supports at this stage is small, since on-site charges, scaffolding costs etc will already have been incurred.
- (3) If only minimal grout is taken, then the cost is low, since the large cost is the cost of the grout itself.

- 9.4 In many cases of poor arches, and there are very many of them, virtually the full treatment is demanded including pressure grouting of foundations, supports, arch ring and spandrel filling followed by thick guniting.

- 9.5 However, there are also cases where e.g. a good pointing will make the bridge satisfactory, or a curtain wall to protect scour, or where a saddle alone will suffice.

- 9.6 It occasionally occurs that a bridge in very good condition yields a low assessment. Here the judgement of the Engineer is supreme and his judgement will be superior to any assessment.

- 9.7 In the case quoted in 9.6, it may well be that "h + d" is low for the span. Instead of increasing "h + d" by guniting consideration should be given to the possibility of increasing "h" by the addition of 100mm - 150mm of macadam, if the longitudinal profile allows.

- 9.8 Road surfaces should not be neglected in the overall assessment. Axle lift-off may be prevented by small regrading and pavements over a bridge particularly where filling over the crown is low, should always be kept free of irregularity to prevent sudden impact and reduce the risk of punching.

- 9.9 In very small single span arches of 2m span or less in extremely poor condition, or even in larger spans, it may be that repairs can cost as much or virtually as much as renewal. The Engineer must be alive to this possibility and while repairs may be less troublesome than renewal, a newly built bridge will almost always be superior to a repaired one. In this matter the use of heavy duty concrete pipes or even multiples of them can be a satisfactory solution. It must be stated, however, that the bridge needs to be in an extremely poor condition to warrant renewal.

# SECTION 10

## ADDITIONAL COMMENTS

- 10.1 Contract documents for the strengthening of masonry arches often fall far short of the desirable. This may lead to faulty tendering, underestimation of the total work required and undue onus on the contractor.
- 10.2 The documents should be as full and complete as in any other civil works contract, including specification of the work and proper bill of quantities.
- 10.3 In grouting e.g. the number of holes bored, their depth and the estimated quantity of cement per hole should be itemised in the bill together with basic cost of cement.
- 10.4 A highly skilled operative will often inject more grout than a less skilled one. Contrariwise a poor operative can disperse grout uselessly under a foundation in filling large cavities. It should be noted that the estimated quantities of cement to be injected are estimates only and they can often be exceeded. It is an error to inject only the specified amount unless redrilling or secondary injection is to follow.
- 10.5 Assessments of strength of Arch Barrel should be seen as aids to the Engineer in his overall assessment and not as complete in themselves. A thorough inspection followed by corrective measures is far more important than detailed assessments.

# REFERENCES

1. THE ASSESSMENT OF HIGHWAY BRIDGES AND STRUCTURES  
DEPARTMENTAL STANDARD BD 21/84  
DEPARTMENT OF TRANSPORT UK.
2. THE ASSESSMENT OF HIGHWAY BRIDGES AND STRUCTURES  
ADVICE NOTE BA 16/84  
DEPARTMENT OF TRANSPORT UK.
3. BRIDGE INSPECTION GUIDE  
DEPARTMENT OF TRANSPORT UK.
4. MASONRY ARCH BRIDGES; A STUDY BY CS CHETTOE AND WILLIAM HENDERSON  
STRUCTURAL PAPER NO. 53 21 MAY 1957  
INSTITUTION OF CIVIL ENGINEERS (LONDON).
5. THE ESTIMATION OF THE STRENGTH OF MASONRY ARCHES BY J. HEYMAN  
NO. 8364 PROC. INSTITUTION OF CIVIL ENGINEERS (LONDON)  
PART 2 1980, 69, DEC.
6. LOAD CARRYING CAPACITY OF MASONRY ARCH BRIDGES  
J. PAGE, BRIDGES DIVISION  
TRANSPORT AND ROAD RESEARCH LABORATORY, CROWTHORNE, BERKSHIRE,  
ENGLAND.
7. LOAD TESTING TO COLLAPSE OF MASONRY ARCH BRIDGES  
J. PAGE, BRIDGES DIVISION  
TRANSPORT AND ROAD RESEARCH LABORATORY, CROWTHORNE, BERKSHIRE,  
ENGLAND.
8. ASSESSMENT OF OLD BRIDGES BY S. CHATTERJEE  
HIGHWAYS AND TRANSPORTATION FEBRUARY, 1985.
9. GUIDE TO BRIDGE HYDRAULICS.  
EDITED BY CR NEILL  
ROADS AND TRANSPORTATION ASSOCIATION OF CANADA.
10. LOCAL SCOUR AT BRIDGE PIERS BY DR. G. J. FARRELL  
THE ENGINEERS JOURNAL VOL. 40 NO. 11 - NOVEMBER, 1987.
11. DRAFT CODE OF PRACTICE FOR THE SPRAYING OF CONCRETE OTHERWISE  
KNOWN AS GUNITE OR SHOTCRETE  
THE ASSOCIATION OF GUNITE CONTRACTORS  
THE ORMOND ROAD, RICHMOND, SURREY TW 10 6 TH.
12. EEC  
COUNCIL DIRECTIVE OF 24 JULY, 1986  
OFFICIAL JOURNAL OF EUROPEAN COMMUNITIES NO. L 217/19.